A. No I don't believe it is. I think, let's take this section 3 for example. I believe it's 3.3 that is referring, that is referred to in the diagram. So we've got under each section in the standard, this happens to be the general one but any other section is the same. We have a subsection .3 which covers general principles and then we have .4 which covers principles for members not designed for seismic loading and .5 for principles for design for members and structures designed for seismic loading. So that's, that's what's referred to in that diagram. So it refers to the last digit on those, the 3.3, 3.4, 3.5 and the "X" indicates whichever section of the standard you're referring to.
Q. Well if you go back to page 12 –
A. Oh, the "C", sorry, yeah okay. I used "X" in my statement.
Q. Well going back to page 12, the diagram doesn't make the distinction you've just referred to does it?
A. Well it does. It's, it's taking the, can we pull this diagram up again, page 12? So we talked to the second paragraph. "Generally the design requirements of each section of the code are presented under five clauses in the following order," and it gives those one to five and those two to five are referred to in that diagram. So it's just the way it's set out. So you can look at any section in the standard and it would have those same, those same five subsections.

HEARING ADJOURNS: 1.03 PM

HEARING RESUMES: 2.16 PM

CROSS-EXAMINATION CONTINUES: MR REID

Q. Mr Smith, before lunch we were discussing issues about ductility and the additional seismic requirements of 3101 an interpretation. Do I understand it that your view about ductility is based on an interpretation of clause 3.2.1 of 4203. That's the general requirement that elements of

CROSS-EXAMINATION CONTINUES: MR RENNIE

Q. And in doing that you reject the alternative which some other engineers have adopted in their evidence in this matter of following the concrete code on this point?
A. I think those other interpretations are inconsistent even without considering 4203.
Q. Well there will be an opportunity to review that in due course. Thank you Mr Smith.

CROSS-EXAMINATION: MR ELLIOTT

Q. Mr Smith, Mr Reid and Mr Rennie asked you some questions about the fact that the NZS 3101 does have provisions for non-ductile columns in it. I just want to refer you to a section in the Bylaw, ENG.CCC.044A.87 and if we could highlight 11.2.5.2 subsection A. That provision talks about the building as a whole and its elements et cetera being designed to possess ductility but it goes on to say, "That shall not apply to small buildings having a total floor area not exceeding 140 metres square and having a total height not exceeding 9 metres." Now firstly the CTV building did not fall into that latter category did it?
A. No it didn't.
Q. And would you say that these non-ductile columns which are in the code would find a place within that type of building described in the latter part of that clause?
A. They could do, yeah.

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Q. In paragraph 10 of your statement you refer to "capacity design" and I want to ask you some questions about the applicability of that to the design of this building. Firstly, am I correct in saying that the equivalent static force analysis in NZS 4203 sets out a formula which requires the selection of a structural type factor?
A. Yes.
Q. And I'll show that to you ENG.STA.0018.47.

WITNESS REFERRED TO SLIDE

A. Ah...

Q. Or perhaps the latter?

A. Certainly the latter. We found a level – quite a low level of damage in that north core which wasn't consistent with severe hinging at the base of that wall, yeah.

Q. It did not yield in the likely plastic hinge region?

A. It may, there may have been a yielding to a minor extent in some regions of that wall but –

Q. On the 22nd of February?

A. Yes.

Q. But not on the 4th of –

A. And the 4th of September.

Q. Now, I just want you to consider given this difference between the structural type factors of the north core and the south wall –

A. Well can I, before you go on, I would say that it would be inconsistent to assume different structural type factors for different parts of the structure so you would, by choosing the S factor you are choosing the overall level of seismic load that you are going to design that building for, it would seem to be inconsistent to choose a different factor for one end of the building as compared to the other end, so...

Q. So there is a problem there potentially?

A. I think so yeah.

Q. Well given that there appears to be that difference in the selection of structural type factor for different parts of the building and as I say Mr Harding can comment on this, if we take that as the case, if we were to assume that the diaphragm connections that remained intact, it would follow from that disparity that the south wall would be likely to yield before the north core, wouldn't it?

A. Yes, yes.

Q. So that the south core would be behaving plastically while the north core could be continuing to be behaving elastically?

A. Yes that is right.

Q. Would that type of disparity in yielding have impacted upon inter-storey drift?
A. Yes it would have.
Q. In what way?
A. It would increase the torsional displacements which add to the translational displacements so it would increase the, potentially increase the drifts on columns.
Q. And if one was designing a building and contemplating that possibility of different yielding points, would that make it more difficult or less difficult to predict the level of inter-storey drift?
A. More difficult.
Q. There has been some evidence about whether the beam column joints were designed as pin ended and I am just going to ask you some questions about that. I am going to start by showing you some graphs from the most recent Compusoft report BUI.MAD249.0552.55. Obviously we need to accept that this is a model?
A. Yes.
Q. With all that goes with the model?
A. Yes.
Q. Assumptions and so on but we have two figures here, 21 which refers to base shear components in an east-west direction and 22 which is base shear components in a north-south direction and I am just going to ask you to explain one or two things about those figures. Firstly, the graph is referring to base shear. Is that in a way a measure of earthquake load?
A. Yes it is, yeah.
Q. So could I refer to parts of the building which were exposed to earthquake loads according to that model for the purpose of our discussion?
A. Yeah these curves are actually not – these are from a model that was designed to assist in interpreting the actual performance on the day of the earthquakes as opposed to a design situation so –
Q. I am asking about actual performance as opposed to design?

A. Oh, okay.
Q. So that is fine?
A. Okay.
Q. Now if we just talk about the top one first. We are talking about the east-west direction and the blue line at the top represents the total load, is that right?
A. Yes.
Q. And the red line is a full line and a broken line, the red line refers to loads affecting the north core, the westward direction and the broken red line the north core in an eastward direction, is that right?
A. That is correct.
Q. And then the same case with the purple broken and unbroken lines which relate to the south wall?
A. Yes, yes.
Q. And the green lines broken, unbroken relate to the columns, is that right?
A. That is correct.
Q. So what that is telling us is that according to the model the north core, south wall and the columns were all exposed to earthquake loads?
A. Yes.
Q. And the extent of that exposure could be determined just by looking at, by reading the numbers off the graph given particular levels of base shear.
A. Yeah.
Q. Is that right?
A. It wouldn't be a – it is an indication. It is not an actual measure of –
Q. Yes?
A. – what they did experience.
Q. Yes. If we move to the bottom one, what that tells us is that in relation to the north-south direction in terms of the, going from largest to smaller, in terms of the extent of earthquake load exposure, it was the north core followed by the columns, followed by the south wall, is that right?

A. Yes, um, yeah I am not so sure about exposure where you are going with that.
Q. No further.
A. Okay, yep.
Q. Now some questions about the beam column joint in particular in the context of Mr Harding's comments about pin ended joints and so on. His calculations firstly BUI.MAD249.0273.9. This here, I will refer you to the heading there so you can see, we are looking at page G8 of his calculations and he has the heading, "Floor beams," which suggests that he is designing floor beams, agreed?
A. Yes.
Q. Can we then go to 0273.10, and do you see some words just towards the top of the page, "Assume columns have no stiffness."?
A. Yes.
Q. So he has adopted that assumption?
A. Yes.
Q. Then go to 0273.12, and do you see the heading there, "Check effect of columns."
A. Okay, yes.

1451
Q. And is it right that what he appears to be doing there is to be checking the effect of columns with stiffness included from that point?
A. Yeah.
Q. Then go to 0273.13?
A. I interpret all these calculations to be designing for gravity load.
Q. Yes. There are two bending moment diagrams there, is that right?
WITNESS REFERRED TO SLIDE
A. Yes.
Q. And in those bending moment diagrams the horizontal line represents the floor beam, is that right?

A. Oh, it represents a line of zero moment I understand but, ah, so we're talking about anything above the line is tension on top of the beam and anything below the line is tension on the bottom face of the beam.

Q. So are the columns represented above and below that line?

A. Sorry the columns? Oh, sorry, okay, the middle, they're quite small scale but there are some moments in the columns.

Q. We can enlarge those if you like?

A. Is that what you're saying?

Q. Well let's just enlarge the diagrams, both of them please. Can I ask you about the moments in a moment, but just asking you about what the diagram seems to represent and is it right that they seem to represent column height both above and below floor level in those diagrams?

A. Yes they do, yeah.

Q. And according to those diagrams for gravity loading as you say, the columns are picking up a moment at the beam column intersection. Is that right?

A. Yes.

Q. Now look at the particular detailing of the beam column joint that emerged and Commissioner Fenwick has represented this in a diagram, BUI.MAD249.0493.3.

WITNESS REFERRED TO SLIDE

Q. Would you agree there we have column vertical bars passing all the way through the joint?

A. Yes.

Q. We have beam top bars passing all the way through the joint?

A. Yes.

Q. And we have beam bottom bars lapped with a hook into the joint?

A. That's correct.

Q. So do you agree that that is detailed as a moment resisting joint?

A. It, the both, the method of construction did join the beams to the columns in a monolithic way so that when they deform they will pick up moments, both the beams and the columns yeah. I, is that answering your question or not really?

Q. Do you agree that those details indicate the detailing of the moment resisting joint, or perhaps you're saying it is a consequence of the detailing that it will resist moment, is that what you're saying?
A. That's what I'm saying, yeah, yeah.
Q. All right. Now given this is the case, in the event of an earthquake with lateral displacements in an earthquake would induce bending moments and other actions on both the beams and the columns, is that right?
A. That's correct.
Q. So is it right to say that those connections were not designed as pin ended?
A. Yes it is, yeah.
Q. Now I think you've drawn, do you draw a distinction between the beam column connections at line F and beam column connections in all the rest of the building?
A. Well there were different sized beams, and also the, this detail that you've drawn shows the bottom beam bars overlapping whereas on grid F the detail shows that they don't overlap.
Q. So even though there is that distinction, were either of those two sets of beam column connections designed as pin ended in your opinion?
A. No they weren't.
Q. Still on this topic of beam column connections, I'm just going to refer you to a couple of sections of the code and ask you to comment on Mr Harding's work. Firstly ENG.STA.0016.69.

WITNESS REFERRED TO SLIDE

Q. Clause 9.4.2, the bottom section of the right-hand column, I just ask for that to be enlarged and of you to read it for yourself?
A. Now, just referring to…
Q. Just read that to yourself?
A. Yes, okay.

1456

Q. Now have a look at clause 9.4.5 please, it's down the bottom and again I'll just ask you to read that to yourself.
A. Yes, okay.

Q. And do you want me to show you or do you – would you accept that 9.4.6 provides equations as to how much of the horizontal joint shear can be carried by the concrete mechanism.

A. Okay, yeah.

Q. Now so my question is we know that there was R6 spirals every 250 millimetres, would those have been sufficient to satisfy those code requirements?

A. I've said in my statement that I think as a minimum you needed the minimum amount required for shear reinforcement. There is a certain minimum amount of shear reinforcement which applied in the columns and also through the joints and anything that comes out of these formulas here would be in addition to that so, but even that minimum reinforcement was not provided so.

Q. So the question if I was to say, was one leg of an R6 spiral every 250 millimetres sufficient to satisfy the requirements I've just pointed out to you, would that be a yes or no?

A. Well there were some other requirements in addition to this one, so, but it was not sufficient to satisfy the requirements of the standard for the joint as far as I'm concerned, yeah.

Q. And Mr Harding I think had said that he intended there would be no spiralling through the joints themselves so it would follow that that could not satisfy those particular requirements if that was the case?

A. That's correct, I mean one of those 9.4.5 if we can expand that again.

Q. Bottom right-hand corner.

A. So I mean the design principle is that the joint shear shall be assume to be resisted by a concrete mechanism plus a truss mechanism comprising horizontal and vertical stirrups or bars, so if there were no horizontal ones the mechanism doesn't work.

Q. Thank you, now one or two questions on cranked splices and I refer you to BUI.MAD249.0284.15, I think Professor Priestley may have referred to cranked splice region of the –

A. In the columns?

Q. The columns, and just highlight the diagram on the left. In fact if we just take the top half of that diagram on the left and enlarge it please. So there is a region there in which the vertical bars overlap isn't there?

A. Yes.

Q. And so I'm talking about this particular region. Now if as was the case there were R6 at 250 spirals, for most bars the spirals will be well away from the crank given the width of the spacing. Is that right?

A. The spiral bars?

Q. Yes.

A. Well it would vary around the column but at one face they would coincide with the splice, on the opposite face they could be up to 250 millimetres away, so.

Q. Just considering that region of the cranked splices, is it correct that forces would be imposed on that cranked region in an earthquake?

A. Gravity and earthquake, yes.

Q. And is it right to describe the sort of forces in that area during an earthquake as bursting forces?

A. There could be, yes.

Q. So would you consider that R6 spirals at 250 would have been sufficient to resist those type of bursting forces?

A. No, in fact I don't have the clause in the standard but I'm aware that there is a clause that –

Q. Well let's look at 5.3.27.1 ENG.STA.0016.41. 5.3.27.1, top left. Is that the one you're thinking of?

A. Yes it is.

Q. So would you just comment on whether the building complied with that clause?

A. So if we start at the second sentence, this is talking about the top of the splice, where you're transitioning from the splice back to a single bar there is an offset in the bars, in the vertical bars and that's what this clause is referring to. The second sentence: "adequate horizontal support at the offset beams shall be treated as a matter of design and shall be provided by ties, spirals or parts of the floor construction." So that means if it's happening within the depth of the floor you've got that confinement, but if it’s above the floor you need to provide ties spirals or ties or spirals and they need to be at not more than 150 millimetres from the point of the bend so as I said, the 250 mm spiral spacing doesn’t comply with that.

Q. Thank you. I’ll show you another clause which I think is in that vicinity, 5.3.29.3, the top right-hand corner, just read that to yourself please?

A. Yes okay.

Q. That refers to anchorage, we can look at the drawings if you like, but did you see any evidence within the drawings of any type of anchorage as required by that clause?

A. No I didn't.

Q. Now finally you've referred in your evidence paragraphs 46 and 47 I think to changes to NZS3101:1995, we've had some evidence about the introduction of a tenancy in 2001 which the Council treated as a change of use and the Council’s position as stated in its opening was that it would have been required to be satisfied on reasonable grounds that in its new use the building would comply with the building code as nearly as is reasonably practicable for the same extent as if it were a new building. Firstly a question on the effect of those changes, which came into place, is it correct that the effect of the changes in NZS3101 in 1995 was that the minimum requirement for confinement of columns increased?

A. Yes I believe it did.

Q. So that if the CTV columns only complied with the minimum requirements of the 1982 code, they would not have complied with minimum requirements of the 1995 code, it would follow I suppose from that?

A. I don't think they complied with either, but –

Q. Yes.

A. – but the requirements did get more onerous in 95, yeah.

Q. In terms of what the Council might have given consideration or the owner and or the owner might have given consideration to doing to the building to bring it up to the 1995 level, would steel props adjacent to columns have been something which could have served that purpose?

A. Yeah, I mean that's a difficult one without – it's like a recognising that the columns are not adequate on their own and trying to reinforce them with another back-up load path if you like, that's the kind of philosophy there. I don't know whether that would have been the best thing to do but it possibly would have been an option, yeah.

Q. Well would that type of solution have served the purpose of bringing it up to the minimum requirement in 1995?

1506

A. There are other options such as wrapping the columns in a fabric or a steel jacket so there are other means of achieving the result that you need. I don't know whether steel props would have been the best option but there were options to do something.

Q. What were the other options apart from the one you've mentioned?

A. Well those are the main ones I can think of: providing a wrap around the columns to confine them effectively.

Q. Was that type of option available at that time in 2001?

A. Look I personally wasn't involved with those sort of projects at that time so I'm not sure.

JUSTICE COOPER ADDRESSES MR RENNIE

Some new matters arose during that.

MR RENNIE

Yes and I'm obliged to you Sir for that. I was listening carefully. I think the issues will best come out when the 'hot tub' of engineers address these issues and I say that because clearly there are matters of opinion here and I anticipate that realistically debating with individual engineers their particular opinion won't achieve a consensus which I see as being most useful to the Commission Sir but I am obliged to you for raising it.

CROSS-EXAMINATION: MR REID – NIL

RE-EXAMINATION: MR MILLS

Q. Just two questions and the first one I just want to take you a provision in the New Zealand Standards 4203 to see if this has any bearing on any of your thinking and it's ENG.STA.0018.16. Now it's the Commentary provision I'd just like you to look at. Can we just enlarge that third paragraph down, the one that begins "Pending...", left-hand column. I'd just invite you to read those first four lines Mr Smith and just tell me if this has any bearing on any of your thinking that you've been giving us in your evidence. You see that part says, "Pending the revision of various other New Zealand standards..... This standard (which of course is the '84 one) should be regarded as the master document with other standards where appropriate subject to it." Is that consistent with your thinking in the evidence you've been giving us about the way this all works?

A. Well I think obviously what it's saying that it wasn't entirely consistent within itself because this clause says that it becomes the master document, implying that there are some conflicts with the other standard whereas the other clause we looked at before seemed to say that it was intended that it co-ordinate with 3101.

Q. So how do you read that provision? I know how I would read it but how would you read it?

A. I suppose you would read it that the concrete standard came in in 1982. This is 1984 and I guess there's a more recent document but I would still read the particular thing we're looking at when we're talking about that was a particular definition of a secondary element which was quite particular in a certain clause in 3101 which I would still say within that standard that is what they meant by secondary elements but that was the only context that I was looking at it in but I think to design a building you need a loading standard and a concrete standard and to use both so.

Q. Yes but I have to say for myself I wouldn't have read this any differently and this is ultimately I suppose a matter for submission and legal argument but I tell you how I would read it. You tell me if you disagree with this that the later standard is being said to be the master document until such time as the earlier standards are revised and if you've got a conflict or difficulty of reconciliation between earlier standards and this one then, subject to the language where appropriate, this one is the one that controls, others are read subject to it. In other words, endeavour where appropriate to fit around what's in the 1984 standard. Is that how you would read that?
A. That's how I'd read it, yes.
Q. Now whether that is the correct reading will, as I say, be a matter of submission ultimately. Now the only other thing I wanted to ask you in an effort to try to minimise the differences between people where possible is to ask you, you've heard the evidence that Dr Jacobs gave this morning?
A. Yes.
Q. Are there any differences between your view that you've been giving us in your evidence and the evidence you've heard Dr Jacobs give which you regard on these questions of code interpretation as being significant, any differences of significance?
A. I wouldn't like to say that I agreed with all of Dr Jacobs, it's possibly some things were not explained in the same terms I would use but I didn't see any major discrepancies there so I think we need to get more specific if you think there's a difference between us.
Q. I'm not suggesting there is. I'm just trying, where possible, to sweep away any differences that may not really matter in the end in terms of the way these provisions apply which is why I put it in terms of is there anything that you're aware of in your evidence and Dr Jacobs' evidence which you regard as being a difference of significance?
A. Well I'm not sure. I think the particular thing is this clause 3.5.14 in 3101. I've said that I think it would not allow the detailing. I just wasn't

clear on what Dr Jacobs' interpretation of that particular clause was, whether he'd had such a, shall we say, definitive view as I have.

Q. Well I'll ask him the same question when he comes back to give his reply evidence so we'll get it from both sides but you're not, I take it, identifying anything that jumps out at you as being a significant point of difference?

A. No, no.

COMMISSIONER FENWICK:

Q. Mr Smith, in your paragraph 11 you talk about all the buildings as a whole that all its elements shall resist seismic forces or movements or that caused failure or a risk to life shall be designed to possess ductility?

A. Yes.

Q. Now how do you define that level of ductility, how do you define it?

A. Well I've defined it by saying that I would think the limited ductile provisions would come under that in certain circumstances.

Q. Can we have WIT.JACOBS.0001.9 please. So there are some load displacement diagrams on columns. Do any of those columns have ductility?

A. Well (inaudible 15:14:26) zero axial load or low axial load. They would have some ductility, yes.

Q. So how would you define that level of ductility on that curve? It went to .07 and it looks as though it started to yield, making an approximation, at about .005?

A. Well I think the, obviously the bottom curve would in my, just ah, of those three curves the bottom curve has ductility. The top curve does not have ductility.

Q. How are you defining ductility?

A. Well as the, um, you look at the yield where it's got the yield happening, at the change of angle from sloping to horizontal and a certain look at that ratio compared to the total rotation.

Q. Let's look at the one with an axial load of $0.4f'_{ca}A_g$, the top one.

A. The top one, yes.

Q. So the point at where it starts to fail looks at about .015. The point at where it yields is .005. Do you say that's got no ductility?
A. No, it's got some but it's, um, very limited.
Q. Okay, I'm saying how much ductility does it require, that clause?
A. I see.
Q. How do you define it, or do you say it's only defined if it happens to be designed to be ductile? How do you define it?
A. I mean the limited -
1516
Q. - Or you say it's only defined if it happens to be designed to be ductile?
A. Well, I mean the limited ductile provisions are theoretically for ductility 3.
Q. Is that clause saying it's got to be limited ductile?
A. No it's not. But that would be –
Q. There is doubt isn't there?
A. There is, there is, it's not a clear definition, no.
Q. So it's your interpretation that what, when you answered my question has a ductility of 3 in your interpretation, that's not ductile. Now I'm not saying that's wrong but it's not defined anywhere, so there is, don't you think, some margin of doubt there?
A. I guess so, yes.
Q. I'm not saying it's good design. You were asked about the steel props and of course that was a solution that was put into the PGC building and that failed in a different manner. The steel props would provide an alternative load path for axial load wouldn't they?
A. Yes they would yes.
Q. And wrapping round confinement would of course provided a better load path, do you agree?
A. Yeah it's, I would tend to, yeah I agree, yeah.
Q. But it wouldn't have done much to the beam column joints would it?
A. No it wouldn't.
Q. In fact it could've made them worse?
A. I don't know if it would make it worse.

Q. If you confine the column are you going to reduce its strength or increase its strength?
A. You're going to increase the strength.
Q. And so that column is now going to be able to transfer more or less load into the joint?
A. More.
Q. So the joint is now better or worse?
A. It's potential to feel more stress, yes.
Q. So the joint, the potential of doing that actually could make it worse?
A. Could do, yes.

JUSTICE COOPER:

Mr Carter?

QUESTIONS FROM COMMISSIONER FENWICK CONTINUES:

Q. I've got one more question. The CTV building, one strong wall, one ductile wall and we have a, I mean I think you quite rightly said that the S factor should be the same, though in fact the standard does, sorry, the commentary does say, "Well maybe you can change it." I agree that rational interpretation of how would stay exactly the same. If we have an S factor of .8, when the person was writing the code would they have been thinking there were at least two braced walls in that frame, in that structure. What do you think?
A. I'm not sure what they were thinking.
Q. Let me take it a little bit further. You've got one wall which is really going to behave elastically, isn't it, because it –
A. Yes.
Q. – you can't put more load on it because the other one's going to be flexing backwards and forwards?
A. That's correct.
Q. So would you agree that all the energy dissipation is going to occur in one wall?
A. Yes.

Q. It's going to be working twice as hard as say a moment resisting frame where you're going to have at least two going, or structurally you had two walls that were similar were going?
A. Yes.
Q. Do you agree?
A. Yes.
Q. So what should the S factor be? The S is meant to represent the structural form in its ability to dissipate energy?
A. Oh, I see what you're saying, so in fact a higher than one could've been used, or should've been used here.
Q. What I'm saying is should S be .8 or perhaps S should be 1.6 or perhaps it should be somewhere between the two. Do you think there's some doubt about that?
A. Possibly, I mean when we looked at it in the DBH report we evaluated S as one, so.
Q. Forget the DBH report, what's your opinion? What would you do if you were faced with this building?
A. Okay, there is one, I would like to bring up one clause in this loading standard if I could to answer that.
Q. I've got it marked here I think. Yes it's ENG.STA.0018.47.
A. You're going to read my mind, which clause I'm…
Q. I hope that's the right one, that's the commentary on the left-hand side and it's the ray, yes.
A. No it wasn't what I was looking for.
Q. Okay well if you look –

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A. Okay, so your question was should a factor, a higher S factor be used. I mean I hadn't really considered it, I consider the asymmetry as an issue which has caused problems and your – how would I treat that I mean there's various ways. You're saying that possibly increasing the S factor would have been a way of increasing the first yield load on the south

wall which would decrease the effect of that post elastic deformation which I entirely agree with, yes.
Q. It would double the energy dissipation wouldn't it, compensate for having to work twice as hard?
A. Yes it would, yeah.
Q. I mean that's not something I suspect everyone would pick up, but I just wanted your reaction to that and thank you for that. I'm not quite sure if you're agreeing with me or not but that's fine.
A. Okay. I mean if I could just bring up this clause on page 33 and 4203, someone's got a reference for that.

JUSTICE COOPER:
Q. It's 0018.38.
A. So we're just talking about the top if we could just isolate it to 3.1 and C3.1, so the top two.
Q. Top third of the page?
A. Top third of the page if you like. So I'm, in particular so we're talking about the asymmetry of those walls irrespective of what you choose for an S factor, the commentary, the last sentence in the commentary, "geometrically dissimilar resisting elements are unlikely to develop their plastic hinges simultaneously and ductility demand may also be increased by torsional effects". Now I think that's a problem with the asymmetry regardless of what S factor you're using for your design, you know, so I think that was – it's definitely an issue.

QUESTIONS ARISING – ALL COUNSEL – NIL

WITNESS EXCUSED

JUSTICE COOPER:

Mr Rennie, I didn't quite get the reference that you were making to I think it was a note. I thought you said a Foreword to the commentary of NZS3101.

MR RENNIE:

Yes Sir, I can give you the reference for that. You're referring to the passage I read out in relation to the relationship between the two standards Sir?

JUSTICE COOPER:

Yes. I didn't quite catch it at the time and I haven't been able to find it since.

MR RENNIE:

Unfortunately Sir I turned the thing away but it's at the beginning of 3101.

JUSTICE COOPER:

Do you want to tell me after the adjournment?

MR RENNIE:

Yes Sir, in fact I can find it for you now Sir, it's page 12 Sir, thank you. I presume Mr Elliott has it and I've now managed to find mine, it's page 12 Sir and it's the second to last paragraph on page 12 of the 1982 edition of 3101. It simply said, Sir, "It should be noted that some provisions in this code are based on proposed amendments to 4203 which at the time of the publication has been finalised."

JUSTICE COOPER:

I thought you said it was in the commentary.

MR RENNIE:

No I'm sorry Sir, if I said that I intended to say foreword.

JUSTICE COOPER:

Yes well the author of those words was exposed wasn't he or she, in a way as to what became of the proposed amendments, whatever they were.

MR RENNIE:
No Sir, this one pre-dated 4203. 3101 came out in 1982 and as I apprehend it, the two were proceeding in parallel and obviously 4203, well I would say obviously, but it is by inference, 4203 was sufficiently far advanced that it was possible to take it into account in 3101.

JUSTICE COOPER:
Well I assume that's so but it nevertheless had the status proposed amendments.

MR RENNIE:
Well I apprehend that that refers to the process that standards go through before they become formally effective Sir.

HEARING ADJOURNS: 3.27 PM

HEARING RESUMES: 3.46 PM

JUSTICE COOPER:
Mr Reid.

MR REID:
Yes may I please the Commission there are two briefs of evidence for Dr O'Leary. Well there is actually three. The third one pertains to the Latham matters so it won't be being read today.

JUSTICE COOPER:
That is to be taken as read isn't it that brief?

MR REID:

Yes as I understand it.

JUSTICE COOPER:

Yes.

MR REID:

So the first two are, have been the subject of some amendments and there are amended briefs that have been filed and the amendments are highlighted in a text so Dr O'Leary can refer to those as he goes through.

JUSTICE COOPER:

Yes all right, well you better call him officially because then the transcribers will know what to put down in the transcript.

MR REID CALLS:

ARTHUR O'LEARY (SWORN)

Q. Your full name is Arthur Joseph O'Leary?
A. Yes.
Q. You're a retired structural engineer?
A. Yes.
Q. You have had experience, extensive experience, sorry you have had extensive design and design management experience of commercial buildings with emphasis on earthquake engineering during your professional career?
A. Yes.
Q. And that is a career that has spanned some 40 years?
A. Correct.
Q. Dr O'Leary could you please read your brief of evidence out loud from paragraph 2 onwards.

WITNESS READS BRIEF OF EVIDENCE FROM PARAGRAPH 2

A. I graduated from the University of Canterbury with a Bachelor Engineering Civil with first class honours in 1966 and was awarded the degree of Doctor of Philosophy in civil engineering from the University of Canterbury in 1970. My doctorial thesis was entitled, "Shear, flexure and axial tension in reinforced concrete members."

Upon completion of my thesis, I was employed by Morrison Cooper and Partners a Wellington based consulting engineering practice for two years. I then travelled to the United Kingdom for two years and worked for two consulting practices, one being Mott Hay and Anderson (subsequently becoming Mott McDonald) and the other Ove Arup and Partners.

Returning to New Zealand in 1974 I was reemployed by Morrison Cooper and Partners staying with them through various practice merges until retirement at the end of 2010. The practice was known as Sinclair Knight Merz Limited at the time of my retirement. During my employment in New Zealand I held various technical and management positions including:

Structural engineer and senior structural engineer in Morrison Cooper and Partners and Morrison Cooper Limited.
Civil structural engineer in Morrison Cooper Limited.
Shareholder and board member of Morrison Cooper Limited. Shareholder of Kingston Morrison Limited.
Shareholder and principal of Sinclair Knight Merz.
Wellington structural engineering manager of Sinclair Knight Merz.
Earthquake engineering practice leader for the world wide practice of Sinclair Knight Merz until retirement.
Senior consultant within Sinclair Knight Merz until retirement.

JUSTICE COOPER:
Q. Mr Reid, this is a long and impressive qualifications which I am sure are not controversial. Would anybody object if I suggest that from here on it be taken as read because I mean we have read this and we are familiar with your experience, qualifications so. Paragraph 15.
A. Okay.
PARAGRAPHS 2 TO 14 TAKEN AS READ

EXAMINATION CONTINUES: MR REID
WITNESS CONTINUES READING BRIEF OF EVIDENCE FROM PARAGRAPH 15
A. My evidence.
(a) outlines my understanding of the loading and concrete standards forming part of the Christchurch City Council bylaw 105 1985 in force when the building permit for the CTV building was issued;
(b) considers the design, construction and standards issues set out in section 9 of the CTV building collapse investigation for the Department of Building and Housing, January 2012, Hyland-Smith;
(c) provides some further observations on aspects of the Hyland-Smith report and the expert panel report; and
(d) discusses aspects of the William T Holmes peer review of the Hyland-Smith report.

I have read the code of conduct for expert witnesses and agree to comply with it. I confirm that all of the matters to be addressed in my evidence are within my areas of expertise.

I have reviewed the original calculations for the structure of design of the CTV building undertaken by Alan Reay Consulting engineer. These calculations are pages numbered G1 to G79, S1 to S57, and F1 to F52. The last page is presumably F52 although unnumbered. I have focused my attention on the pages in the G and S numbered series as these are the relevant pages associated with the main seismic resistant design of the structure of the CTV building. The F numbered pages appear to be specifically related to foundation design.

Q. Just stop you there Dr O'Leary.

MR REID:
Your Honour, the next few paragraphs just outline really the material that Dr O'Leary's reviewed.

JUSTICE COOPER:
Yes.

MR REID:
That could be taken as read.

JUSTICE COOPER:
Yes any objection to that course, yes thank you.

PARAGRAPHS 18, 19 TAKEN AS READ

EXAMINATION CONTINUES: MR REID

Q. Dr O'Leary could you please go on to, I think I will have you read paragraph 20.

WITNESS CONTINUES READING BRIEF OF EVIDENCE FROM PARAGRAPH 20

A. I am aware that there was remedial work undertaken in 1991 to bring an identified deficiency in the connection of levels 4, 5 and 6 floor slabs to the north shear wall up to compliance within the requirements of NZS3101 and NZS4203. I have been unable to check the calculations undertaken at the time of the discovery of the need for remedial work as the reproduction of the calculations supplied to me was not of a quality to be readable. However I have checked the adequacy of the agreed loads to be transferred from the slab to the walls and as I later note except in the east-west direction they were in my view appropriate to comply with NZS4203 after the remedial work had been carried out.

The loading and concrete standards in the bylaw.

To address these issues I will firstly comment on the overall approach to both the loading and the concrete standards requirements of the time. Both the loading NZS4203 and the concrete NZS3101 standards apply to the CTV building design. The loading standard is independent of the construction materials used in the building. The concrete standard is a material standard uses the loading criteria derived from the loading standard and applies those criteria for the use of reinforced concrete as the construction material.

The introduction to NZS4203 contains a general statement in clause 3.2.1 that, "The building as whole and all its elements that resist seismic forces or movements or that in case of failure are a risk to live shall be designed to possess ductility." This requirement, is in my view, put into context by the commentary clause C3.2 which states in part, "the general requirement for ductility must at present be qualitative rather than quantitative except for buildings designed to dissipate seismic energy by ductile flexural or yielding."

Clause 3.2.2 of NZS4203 requires that, "Structural systems intended to dissipate seismic energy by ductile flexural yielding shall have adequate ductility." Clause 3.2.3 then states, "Adequate ductility in terms of clause 3.2.2 shall be considered to have been provided if all primary elements resisting seismic forces are detailed..."

There is no quantitative guidance in NZS4203 as to what is adequate ductility and how to provide for it. However clause 3.2.3 refers the designer to the appropriate code which in the case of the CTV building is NZS3101 and which provides quantitative guidance.

The requirements of clause 3.3.3 of NZS4203 are related to ductile frames and not secondary elements. Thus if it can be shown that the frames on grids 1, 2 and 3 and F of the CTV building were secondary elements, then clause 3.3.3 would not apply.

The CTV structure may have been designed to dissipate seismic energy in ductile flexural yielding of the shear walls but from the observations in the Hyland Smith report, and from reviewing photographs of the shear walls after the collapse, it would appear that it did not. Rather it would appear that the onset of collapse occurred before any of the shear walls yielded significantly except with the possible exception of the wall on grid 1, and therefore the shear walls were not called upon to dissipate energy by flexural yielding.

Although the shear walls were designed as ductile, because of the likely sequence of failure of the CTV building, they were (except with the possible exception of the wall on grid 1) not called upon to dissipate energy in a ductile mode of failure. The likelihood that there was an early diaphragm disconnection at probably level 2 and/or 3, meant that instability of some of the columns in their failure precipitated collapse before the shear walls were fully loaded. In other words a collapse sequence was somewhat independent of fully loading the shear walls.

Section 9 of the Hyland Smith report, pages 109 to 120 identifies design construction and standards issues in relation to the possible causes of the collapse of the CTV building in the 22nd February 2011 aftershock. In the following parts of my evidence I address the issues raised in this section of the Hyland Smith. As a starting point however it is necessary to review relevant aspects of earlier sections of the report, and then I will make some general comments about structural design of the CTV building.

I reviewed tables 1 and 2 on page 26 of the Hyland Smith report. I have found no clear indication in the report as to how the 1986 non-ductile detailing and 1986 ultimate drift figures have been determined. Reference to appendix F of the report has not assisted me as one of the columns to which table 13 and 14 apply is different from those in tables 1 and 2, one of the columns that is. The common column is F2. At level 3 chosen level in table 1, the 1986 non-ductile detailing limit of table 14, is 0.69 percent which is inconsistent with the 0.6 in table 1. The 1986 ultimate drift of 1.1 percent in table 1 is consistent with table 14 in one direction, earthquake, north-south shows 1.12 percent drift, but very different for the other direction earthquake, east-west shows 0.46 percent.

Further the last paragraph on page 27 continuing onto page 28 contains a statement. "… Under this interpretation elastic performance of the secondary members of the CTV building was required to be demonstrated at 55 percent of the design maximum or ultimate earthquake drifts". For the CTV indicator columns the applicable drifts for this check are 1986 non-ductile detailing figures, refer table 1 and table 2. These references are not clear. The origin of the 55 percent figure and what it means are not adequately explained either in the body of the report or in appendix F. In this respect I also refer to the Holmes peer review at page 3.

In addition the discussion of "drift capacity of columns" on pages 27 and 28 of the Hyland Smith report seems to assume that the commentary to NZS4203 was mandatory, which it is not. The analysis techniques discussed in the commentary clause C3.2 were not available to designers in 1986. They were only available to researchers at the time.

The CTV building horizontal load resistance was provided by shear walls. The shear wall configuration of the CTV building would have placed it in the category of a moderately eccentric structure according to clause 3.4.7.1 of 4203 in a commentary to that clause. I accept that there is room for differing interpretations of the definition of moderate eccentricity in the commentary to the clause but I believe that the intent of the definition was to limit the amount of torsional resistance provided by elements required to resist predominantly translational modes. Reasonably regular structures more than four storeys high, even with a high degree of eccentricity, were allowed by clause 3.4.7.1(b) of NZS4203 to be analysed by quite unsophisticated methods of analysis, (the static method or two dimensional modal analysis), although the standard recommends (but does not require) a more sophisticated three dimensional modal analysis. If the two dimensional methods of analysis were used, calculating torsionally induced inter-storey drifts would be precluded. I discuss this further later in my evidence.

The CTV building structural configuration meant that the torsional resistance to a horizontal earthquake generated effects was shared by two separate combinations of shear walls. The east-west wall on grid 5 and the wall on grid 1 provided efficient torsional resistance. The walls running north-south between grids 4 and 5 provided a second independent structure to provide torsional resistance.

The understanding of torsional resistance in the structures of this type at the time of the CTV building design was that the torsion would be shared by both sets of torsional resisting elements, but if one was fully loaded by translational effects of an earthquake, then the other could be designed to resist all the torsional effects.

Q. Can I just stop you there Dr O'Leary, you read that last sentence as resisting instead of restraining in the third line of 35.

A. Yeah, it's restrain.

Q. Thank you.

A. This was in my view a widely held understanding amongst structural engineers at the time, and until recently.

A. The corollary of the ability to share the torsional resistance was that if a wall (in the case of the CTV building the wall on grid 1), was highly loaded by translational effects of the earthquake (to the extent of significant yielding), then the torsional load that it may have been resisting would be shared to the other torsional resisting sets of walls.

This in effect should guard the structure from a wall failure resulting from a combination of torsional and translational loading.

NZS3101 defined primary and secondary elements and had different seismic resisting requirements for each class of element. Relevant definitions were contained in clause 3.5.14, but the differing design requirements were covered in other parts of the standard.

An important issue in considering the compliance of any design with the standards of the day is the interpretation of clause 3.5.14 of NZS3101. My interpretation of 3.5.14.1 is that the beams and columns of the CTV building were group 2 secondary elements.

This is supported by the first sentence in the commentary to clause 3.5.14.1 of NZS3101 which states in part "the definition of a secondary element is more particular than that in NZS4203, and includes such primary gravity load resisting elements as frames which are in parallel with stiff shear walls and do not therefore participate greatly in resistance to lateral loads… " The clause further states that, "Frames in parallel with slender shear walls should be designed and detailed as fully participating primary members …"

There is no applicable definition in NZS3101 as to what is a stiff or slender shear wall in terms of 3.5.14.1, but I consider that the widely interpretation at the time would have been whether the frame would provide a significant contribution to the lateral load resistance of the structure. On that basis the shear walls of the CTV building are likely to have been regarded as stiff by both the designer and a Council reviewing engineer.

The frames running east to west on grids 1, 2 and 3 were not designed to resist any lateral load. There were only two frames running north-south, one on grid F and the other a part frame on grid A and again those were not designed to carry any lateral load. All the lateral load resistance was provided by the shear walls as can be seen from a review of the structural calculations. Accordingly the shear walls were likely to have been regarded by the designer as stiff and the Council reviewer could in my view have reasonably come to a similar conclusion.

On the basis that the frames including columns were group 2 secondary elements then clause 3.5.14.3 of NZS3101 becomes applicable:

(a) The intent of clause A is that when secondary elements have deformations less than a defined value, (V times Delta) and remain elastic, they do not have to comply with the additional seismic requirements of the standard.

(b) Sub clause (b) requires the additional seismic standards to be met when the element starts to yield at deformation less than the defined value.

(c) Sub clauses (c) and (d) do not influence the discussion in this section of my evidence.

(d) Sub clause (e) requires an elastic theory shall be used to at least the deformation level compatible with one-quarter of the defined value. This is a requirement to avoid excessively large post elastic deformations under any circumstances. It is not directly applicable to the CTV building design as it does not govern any relevant compliance requirements.

(e) I interpret sub clause (f) to mean that even if elastic theory is applied at deformations greater than half the defined value, then limited ductility requirements may be applied which are covered in clause 14 of NZS3101. There is no upper limit of deformation identified in this requirement but it would not be logical to extend the upper limit of the requirement to the defined value, as that situation is already adequately covered in sub clause (a). I believe that sub clause (f) limits were to be applied when the limit of elastic theory lay between half and the full defined value of deflection but not at the full defined value.

In summary the design and detailing of the secondary members of the CTV building should have satisfied the applicable sub clauses (a) or (b) of NZS3101 clause 3.5.14.3. The beams and columns on grids 1, 2, 3, a and f were in my view group 2 secondary elements.

Returning to the Hyland Smith report I do not consider that it is appropriate for the methods of analysis not available except as research tools or excluded by the standards of the day to be used to make assessments as to whether the analysis design in detailing of the CTV building complied with the standards of the day. I provide two examples to illustrate my point:

(a) Firstly I refer to appendix F of the Hyland Smith report – Method, first paragraph at page 253. The displacement compatibility analysis referred to in this paragraph used Cumbia software from a paper published in 2007.

(b) Secondly the ERSA modelling used to perform various analyses included the effects of flexible foundations. This is referred to at page 236 of the Hyland Smith report. Clause 3.8.1.2 of NZS4203 specifically stated that for the purpose of computing deformations, foundations rotations were to be neglected. This means that the computed inter-storey drifts used to investigate compliance in the Hyland Smith report are in conflict with the provisions of NZS4203 and almost certainly give drifts that are larger than those where foundation rotation was not allowed for in the model.

I accept that the use of the most up to date structural modelling in computer analysis techniques to ascertain the reasons for the collapse of the CTV building is appropriate. However I have a different view in relation to using these techniques to identify compliance/non-compliance, with the standards current at the time of design. In my view only analysis techniques available to the practising structural designer at the time the design was carried out should be used for the identification of compliance. Even the use of research tools available to the research community but not the practising engineer is not appropriate when considering compliance issues.

JUSTICE COOPER:

Q. Now, just on that point is there anything in the standard itself that would justify that stance?

A. Well the standard of practice expected was the normal standard of practice at the time, and the normal standard of practice at the time would not have included what is only available to the research community.

Q. I can understand that being said if one was judging whether somebody had been negligent or not but on the pure question of whether the standard is complied with, surely that's an objective matter which can be ascertained by application of the state of knowledge at the time. Do you think the authors of the standard would not have been writing it having regard to knowledge in the academic community at the time it was written. Is that what you're saying?

A. Well I think I cover it a little later but I'll put it this way. The standard was published, the concrete standard 3101, was published in 1982 and it grew out of a provisional standard first published in 1970 which grew out of a set of lecture notes that Professor Paulay and myself were subject to from Professor Paulay and Professor Park. Now both of those gentlemen were on the 1982 Standards Committee. Well the committee that actually carried on till 1982 and anything that would have been considered necessary for the standard would have been included in the standard. They wouldn't have left issues out of the standard that they considered were necessary for good practice.

Q. I can understand that proposition but it seems to me slightly different from the one that you were making but anyway that's fine.

WITNESS CONTINUES READING BRIEF OF EVIDENCE AT PARAGRAPH 46

A. "Most of the evidence below is based on my interpretation of clause 3.5.14.3 of NZS 3101 as discussed earlier. I now discuss the design issues at pages 109–115 of the Hyland Smith Report.

Building inter-storey drift limits.

The Hyland Smith Report concludes at page 109 that: " … it is therefore debatable whether the drift limits were satisfied". I believe that this conclusion is open to question for at least three reasons.

(a) Clause 3.8.1.2 of NZS 4203:1984 expressly states that "Computed deformations shall be calculated neglecting foundation rotations". The analysis from which the report draws its conclusions allowed for some level of foundation rotation although the report acknowledges that allowing for foundation rotation was contrary to what was required in NZS 4203.

(b) In addition, the method of analysis in Appendix F was undertaken using techniques unavailable to designers in 1986. The factor of 0.85 for frame effects referred to at page 253 of the report is based on a paper published in 2007.

(c) The report, page 109, also notes that the drift requirements were satisfied " … if no account was made of the effect of inelastic deformation initiating in the south wall at the K/SM deformation levels". Recognition has not been given to the fact that drift levels in NZS4203 were set making allowance for inelastic deformation. This is explained further in Commentary clause C3.8.1.2 of NZS 4203.

Q.

Drift Capacity of Columns and Column Confinement

The drift capacity of columns and the column confinement are intimately linked. As two adjacent floors of a building move horizontally relative to each other (inter-storey drift), the columns between the floors develop vertical curvature which produces bending moments (flexure) in the columns. The bending moment is a function, among other things, of floor element strength and stiffness at each end of the column.

As the inter-storey drift increases, the bending moment in the columns may start to yield the concrete and/or the reinforcing. This is the onset of plasticity and to sustain this plasticity, the column needs to be ductile. How ductile the column is, depends on how much confining reinforcing (spirals or stirrups) is provided in the column.

The drift limits discussed in paragraph 47 of my evidence equally apply to the drift capacity of columns. Accordingly, I consider that the conclusion in the Hyland Smith report at page 110 that the columns "were required to be designed using the additional seismic design provisions of NZS 3101:1982" is also in question and possibly incorrect. In my opinion, the more detailed analysis in Appendix F of the report does not clarify the issue for various reasons. The most critical columns in the centre of the building were not included as sample columns in Tables 13 and 14.

I have concluded that the internal circular columns (located at Grids B2, B3, B4, B5, C2, C3, C4 and C5) and including the circular columns at A/B1 and B/C1, comply with the requirements of clause 3.5.14.3(a) of NZS 3101. That is, it was appropriate to detail these columns as "members not designed for seismic loading."

JUSTICE COOPER:

Q. Was there a column at B5?

A. There's four internal columns on bends 2 and 3.

EXAMINATION CONTINUES: MR REID

Q. Dr O'Leary, are you referring to the column notations as they appear on the plan for the building?

A. Grid references.

Q. Perhaps you could have a look at BUA.MAD249.0284.1. That's the signed plans.

A. S 15. They're columns C6, 7, 8, 9, 12, 13, 14.

Q. Just bring it up Dr O'Leary. MAD249.0284.16. So that's the plan from which the notations come, is it?

A. I'm sorry. There's a D4, D5. We've got them wrong. It's B2, B3, C2, C3, D2, D3 and E2, E3. I'm sorry about that.

JUSTICE COOPER:

Q. So located at Grids B2,B3, C2, C3, D2, D3 and E2, E3?

A. That's correct, yes.

WITNESS CONTINUES READING BRIEF OF EVIDENCE

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A. "That is, it was appropriate to detail these columns as "members not designed for seismic loading". This discussion is based on the inter-storey drift –

Q. I'll just stop you there, sorry you read that as "discussion", it should be, "This conclusion."

A. Oh, sorry yes. This conclusion is based on the inter-storey drifts given in Alan Reay calculations at pages S15 and S16. It is not known how these drifts have been calculated but it is likely that they were from an output from the ETABS analysis that I understand was carried out at the University of Canterbury. The output from that analysis has not however been able to be found at this time.

The actual drifts do however seem reasonable to me. The drifts in the north-south direction are approximately five times those in the east-west direction. This may have been influenced by the drifts on grid F being magnified by torsional effects. The drifts may be the maximum drifts from any of the relevant load cases. Torsional effects would have a much lower influence on the east-west drifts.

The circular columns on grid F did not in my view meet the criteria of clause 3.5.14.3(a) of NZS 3101, as they did not remain elastic under the factored drifts (K/SM). They should, in my view, have been designed to clause 3.5.14.3(b) of NZS 3101, (additional seismic requirements of the standard).

The confinement reinforcement in the columns is governed by clause 6.4.7.1(b) of NZS 3101 for non seismic columns. The confinement spiral for the non-seismic columns, shown on the drawing S14, complied with that clause for the non-seismic columns. The confinement spiral for the columns on grid F should in my view have been designed in accordance with clause 6.4.7.1(a) and thus did not comply with respect to either the spacing or the size.

In conclusion I consider that the decision to design the columns as "members not designed for seismic loading", may have been justified according to the standards of the time, except for the columns on grid F

which should in my view have been designed for seismic loading.
Minimum shear reinforcing of columns.

If the internal columns did not need to be designed as seismic elements the test as to whether they required shear reinforcement would have been covered in clause 7.3.4.1 of NZS 3101. The relevant test was if the shear stress across the columns was less than half the shear strength allowed to be provided by the concrete, then no shear reinforcement was required. I considered a typical circular column over several floors and have concluded that the shear stress was low enough not to require shear reinforcement for a column designed as non-seismic.

There is nothing in NZS 3101 relating to columns designed for seismic loading that alters that test, except in certain circumstances that do not apply to the circular columns. Clause 7.5 allows shear to be carried by the concrete (I refer to equation 7.41 and clause 7.5.2.2). The restriction in clause 7.5.2.2 is met and so some shear is allowed to be carried by concrete. The two back references to clause 7.5.3.1 are also satisfied and support the analysis above. I note that the requirements of clause 7.3.16.1 appear to apply only to slab/column joints. This is clarified by clause 9.2.1.

In summary I consider that the internal columns on grids 2 and 3 did not require shear reinforcement if seismic detailing was not required. It is likely, however, that the circular columns on grid F were required to be reinforced for shear. Therefore in my view neither the size not the spacing of the spiral would have complied with NZS 3101.

Spandrel panel separation.

The issue raised in the Hyland Smith report at pages 110 – 111 about the spandrel panel separation is in my view essentially a construction matter. A note on the drawings or in the specification related to minimum separation would have provided assistance to the building contractor but it is not in my view a design error or a standards compliance issue as such. My calculations indicate that a clearance at either end of each panel of 10 millimetres would have been sufficient.

I do note in the CTV building non-linear seismic analysis report, (Ref:11033-00 revision 0 February 2012) that the spandrel panels were modelled as "planer, linear, elastic shell elements located along the column centre lines", (page 20). I do not believe that this is correct because the panels were quite flexible longitudinally as the vertical part of the panel was torsionally flexible and it was offset from the columns. Therefore the influence of the column being restrained by the panels has been given undue weight in the modelling outlined in that report, that included the spandrel panels. I consider that any influence which the panels had in the premature onset of failure of the columns on line F would be relatively minor.

I believe that my views as to the spandrel panel influence on the seismic performance of the CTV structure, are supported in the statement of evidence of Michael John Nigel Priestley at pages 43 and 44.

The Compusoft report on modelling the CTV structure states at section 5.6, "Spandrels have been modelled as planer, linear elastic shell elements located along the column centre lines as shown in figure 18." As discussed above this is not in my opinion an appropriate approach to modelling the spandrel panel effect on the response of the structure. I consider therefore that any part of the Compusoft analysis that contains a spandrel panel modelling is not correct and conclusions drawn from it cannot be supported.

Beam column joints.

This issue is referred to at page 112 of the Hyland Smith report. I have considered the requirements of NZS3101 for shear reinforcement in the beam column joints of the frames, particularly on grids A2 and 3 but my comments also apply more generally to grids 1, 4 and F. The wording of the standard is not entirely clear given that there is some scope for conflict between the requirements of different sections of the standard. I will, however identify each clause that may be relevant and then comment on how I believe it should be interpreted.

Clause 7.3.16.1 of NZS3101 states, "When gravity load, wind, earthquake, or other lateral forces cause transfer of moment at connections of framing elements to columns, shear resulting from moment transfer shall be considered in design of shear reinforcement in the joint." Although the clause is in the chapter on shear and torsion, it is a general requirement related to beam column joints which is covered in detail in chapter 9 of the standard.

Clause 7.3.16.1 is a general statement and is expanded on in clause 7.4.1. Clause 7.4.1 requires a minimum level of shear reinforcement in a beam column joint, not designed for seismic loading. The spiral shear reinforcement in the beam column joint did not in my view comply. It provides about half the minimum requirement but the question then arises as to what is required for a joint that is designed for seismic loading? There is no requirement as such for transfer of moments to columns in clause 7.5 which is the relevant clause for the design of shear resistance in members designed for seismic loading. The relevant clause for beam column joint shear are in chapter 9 of the standard but it does show an inconsistency in the standard.

There is a further issue about how clause 7.3.16.1 should be applied. It is one of two clauses that contain requirements for slab connections to columns. It appears to stand on its own but its location in the standard it is not logical and there is a general reference later (in clause 9.2.1) that seems to infer that all the requirements is 7.3.15 and 7.3.16 relate to slab connections to columns.

I believe clause 9.4.1 of NZS3101 is inconsistent with clause 3.5.14.1(a).

COMMISSIONER FENWICK:

Q. Is that the right clause, if I could just check that clause please?

A. I think there might be a misprint there.

EXAMINATION CONTINUES: MR REID

Q. Carry on.

A. I consider that clause 9.4.1 should be part of general clause 9.3 and is misplaced as part of general clause 9.4. Even structural members such

as columns in very stiff buildings are subject to seismic load reversals and on that basis the inference is that all such columns should be subject to clause 9.5.
However clause 9.5.1 "general" states that the clause only applies to "… beam column joints that are subjected to forces arising as a result of inelastic lateral displacements of ductile frames". This is very much more specific than what may be inferred from clause 9.4.1. This leads me to conclude that clause 3.5.14.3(a) may well govern the beam column joints of grids 2 and 3 of the CTV building if the analysis of the structure showed that the building complied with requirements of that clause.

JUSTICE COOPER:

Q. To the extent I'm following this at the moment, should that reference to 3.5.14.3(a), are you talking about the same provision as you were at the beginning of paragraph 67?
A. Yeah I think that should have been 3.5.14.3(a) as well.
Q. So going back to –
A. That's a misprint.
Q. – paragraph 67, if we change that to 3(a)?
A. Yeah that's correct.
Q. Now just let me see if that makes this easier to follow.
A. Probably not.
Q. All right, thank you.
A. I think I was half way through –
Q. Sixty-eight, this leads me to conclude –

EXAMINATION CONTINUES: MR REID

A. This leads me to conclude that clause 3.5.14.3(a) may well govern beam column joints of grids 2 and 3 of the CTV building if the analysis of the structure showed that the building complied with the requirements of that clause. However it is unlikely that the beam column joints in Grid F complied with NZS3101.

I conclude that the beam column joints for all circular columns were unlikely to comply with NZS3101.

Plan assymmetry and vertical irregularity.

The Hyland Smith report at page 112 discusses the CTV building plan asymmetry and vertical irregularity. My interpretation of clause 3.4.7.1 of NZS4203 is that the plan of the building was not asymmetrical. NZS4203 would in my view class the CTV building as being a reasonable regular structure of moderate eccentricity. I categorise the structure as reasonable regular on the basis that all floors are the same shape and size and apart from the bottom and top floors the inter-storey height is constant. The height differences in the bottom and top storeys are not in my opinion excessive.

Commentary clause C3.4.7.1 of NZS4203 has a definition of moderate eccentricity with which I believe the structure complies although I accept that the definition is not entirely clear. The NZS4203 commentary has, as an aim, achieving symmetrical structures but it is certainly not a requirement of the standard. Methods of analysis of unsymmetrical structures are incorporated in the standard.

The comments in the Hyland-Smith report at page 112, do not in my view reflect the correct interpretation of the standard especially as understood at the time by practising structural engineers.

Wall on line A.

This issue is raised at pages 112 to 113 of the Hyland-Smith report. it is clear from both the drawings and the calculations related to the masonry wall on line A that the wall was to be seismically separated from the seismic load resisting elements of the structure. The design was in my view consistent with this position.

It is far from clear whether the drawings show the top course to be grout filled as indicted by the Hyland-Smith report at page 113. I understand that when work was being carried out on the wall after the September 2010 earthquake the top course of the block work was observed not to be filled to the top. The design calculations have a very clear detail as how to construct that joint and show the course as not being fully grout filled. See page G53 of the CTV building calculations.

Further, greased vertical bars were shown on the drawings between the top of the block work and the beam above. See drawing S9. Even the detail in the calculations for the design of the vertical greased bar to resist the out of plane loads from the earthquake generated forces was included. This shows a clear intention for the block wall to be separated from the beam above and I believe that an experienced contractor would have understood that intention. The site foreman confirmed in an interview with the Department of Building and Housing that this was his interpretation.

In my opinion the block walls on grid A complied with the seismic separation principles of NZS4203.

Diaphragm connection.

This matter is raised at page 113 of the Hyland-Smith report.

Disconnection of the floor slabs diaphragms from the northern shear core walls has been suggested as a cause at least in part of the collapse of the CTV building, in several of the collapse scenarios of the Hyland-Smith report.

Most of the horizontal load generated by an earthquake is attributable to the floor slabs of a structure. The load from the slabs has to be transferred into the horizontal load resisting elements of the structure, the shear walls in the case of the CTV building and from the shear walls the load is transferred into the surrounding ground through the foundations. The question whether the load path from the floor slabs to the shear walls complied with NZS4203 and NZS3101 has been discussed in the Hyland-Smith report at page 113 and in more detail in appendix G.

I have reviewed the calculations related to the diaphragm connection to the north core walls and have concluded that the connection did not comply with NZS4203.

I have also reviewed the loads agreed by Holmes Consulting Group and Alan Reay Consultants for the remedial work undertaken in 1991.

There is a letter from Alan Reay Consultants Limited to Holmes Consulting Group dated 2nd February 1990 received by Holmes Consulting Group on 7 February 1990, noting the agreement of the loads required to be resisted by the remedial work. I have concluded that, with that work completed, the connection at all floors complied with NZS4203 in the north-south direction. I do not believe that the slab connection to the north core shear walls complied in the east-west direction.

Q. Just stop you there Dr O'Leary. There's a – my learned junior has pointed out to me that from the words, "in the north-south direction," at the end of the second to last sentence to the end of the paragraph, is an amendment from the material, the brief as it was originally filed, so on that basis would you like to, would like to make amendment by underlining that piece?

A. Yeah fine. That is the last sentence is it?

Q. That's from, "in the north-south direction..."?

A. Oh, right, yeah.

Q. Through to the end of the last sentence?

A. Yep, okay.

JUSTICE COOPER:

Q. So it is the last sentence.

A. Yes.

EXAMINATION CONTINUES: MR REID

Q. It's the last five words of the second to last sentence plus the last sentence.

A. Well just to confuse the issue there was an incorrect standard number put in there as well.

JUSTICE COOPER:

Yes so Mr Reid, what is – the correction is the words?

MR REID:

"In the north-south direction." "I do not believe that the slab connection to the north core shear walls complied in the east-west direction." It is that whole piece.

JUSTICE COOPER:

Right, so in its previous iteration he simply said the connection complied once these remedial works had been done?

MR REID:

Yes he did.

JUSTICE COOPER:

Now he's saying complied in one direction, not the other.

MR REID:

Yes.

EXAMINATION CONTINUES: MR REID

WITNESS CONTINUES READING BRIEF OF EVIDENCE FROM PARAGRAPH 82

A. The Hyland-Smith report appears to indicate that there was no diaphragm/wall connection failure until after structural failures that started the sequence of events leading to total collapse. However this seems to be a controversial point. I hold the view that an early disconnection at possibly level 2 or 3 could have occurred leading to instability of internal columns between levels 1 to 2, or 2 to 3. This is supported by Mr Holmes' peer review of the Hyland-Smith report at page 15, first and second paragraphs.

Robustness.

I now refer to the question of robustness raised in chapter 9 of the Hyland-Smith report pages 113, 114. Robustness is a concept that was understood by structural engineers at the time of the design but my understanding was that if the design complied with the standards of the day (in this case NZS4203 and 3101) then the required robustness was regarded as being incorporated in the design.

Documentation.

The issues raised at page 114 of the Hyland-Smith report under, "documentation," are covered in various paragraphs of this evidence. Construction joints in paragraphs 89 and 90, masonry infill in paragraphs 73 to 75, and spandrel panel gaps in paragraphs 59 to 62. The issue of starters from precast beams on grids 1 and 4 is I believe shown on drawing S15.

Construction Issues.

The Hyland-Smith report refers to a number of construction issues relating to the CTV building. Item 2 on page 4 of the letter of the Royal Commission to the Christchurch City Council dated 27 February 2012 refers to these construction issues. My headings below relate to the list of issues in the Royal Commission's letter.

The concrete strength distribution.

The concrete strength distribution and other issues related to the concrete testing have been the subject of two submissions made to the Royal Commission. These are BUI.MAD249.0373 et cetera, et cetera -

Q. You don't need to read that –

A. Yeah. It is inappropriate for me to add any further comment on the concrete strengths as reported in the Hyland-Smith report and the Hyland materials report given that it is not my area of specialist expertise.

Lower than expected concrete strengths.

I refer to paragraph 865 above.

Cores taken from a line 4-D/E column.

I refer again to paragraph 86 above.

Construction joints. Surface finishes of construction joints in the pre-cast concrete were specified in clause 3.6 and 3.12 of the pre-cast concrete specification for the CTV building. Section 2.3 of the concrete specification required compliance with NZS3109:1980. "The contractor shall comply with all requirements of NZS3109:1980 except where specified otherwise herein." Preparation of construction joints is covered in that standard.

It would be unusual in my view for a Council building inspector to give detailed scrutiny to how construction joints were prepared. Very frequent presence would be required onsite and even then many construction joints would already be covered up although they may have been prepared only a few hours earlier and others would have been inaccessible. I would not therefore realistically expect a Council building inspector to pick up poor preparation of construction joints.

Smooth precast to cast in situ interfaces on pre-cast beams.

I refer to paragraph 90 above. S

hell beam reinforcing steel poorly developed in line C wall.

The fact that reinforcing steel details at the interface of pre-cast members to in situ concrete were not as detailed on the drawings, would be very difficult to pick up by a building inspector doing limited inspections. I would be surprised if a Council building inspector were to pick up such an error.

Connection at line 4-DE column missing a bar.

I refer to paragraph 92 above.

North core being significantly out of plumb.

I am not surprised that the north core was out of plumb after the collapse of the building. Before the building collapsed the north core carried the load of the floors spanning into it, particularly along grid line 4. After the collapse that load was distributed more uniformly over the adjacent site. This would have been, thus, sorry, thus there would have been elastic rebound of the foundation soils under the north core when it was unloaded by the collapse. The unloading on grid 4 would have been much greater than on grid 5 and so the north core would have tilted towards the north. I have not done any calculations to estimate the tilt as the detail is a geotechnical issue.

Vertical seismic separation joints in the masonry infill.

The vertical seismic separation joints in the masonry infill may have been compromised by mortar on the outer face. This would have been very difficult to observe as the wall was built adjacent to an existing wall. Intermittent inspections of the gap are unlikely to have picked up the problem if there were only isolated instances of mortar in the gap. If the problem was more widespread it could possibly have been expected to be discovered by some building inspectors. However if it was isolated then I would not expect that a building inspector would be likely to pick up on this issue.

I do not know what level of inspection was stipulated in the consultant's terms of engagement. The ultimate responsibility for construction lies with the contractor. It is up to the contractor to construct the building according to the drawings and specification. The contractor is the entity that is in control of the site, particularly with a new building project. What level of construction observation is to be provided by the consultant structural engineer is a matter for the owner. The consultant will advise the owner on the level of observation that would be appropriate, but the final decision is for the owner.

A reviewing engineer's assessment of compliance with NZS 3101 clauses 3.5.14.3(a) and (b).

I have been asked to comment from my own perspective as to how a reviewing council engineer could reasonably have interpreted the CTV building plans in the light of the standards of the time. I discuss this in the following paragraphs of my evidence.

Clause 3.5.14.1 of NZS 3101 defines a secondary structural element which is then addressed in the rest of the clause. Paragraphs 38–40 of my evidence discuss the interpretation of this clause, but I draw attention to it again as it has a significant influence on the discussion which follows.

The columns appear to have been secondary elements as they are, as stated in clause 2.5.14.1 of the standard, "Not necessary for the survival of the building as a whole under seismically induced lateral loading."

A reasonable interpretation of this requirement is that a column could fail resulting in a localised (part) failure of the structure but not collapse. The consequence of such an interpretation is that the requirements of clause 3.5.14.3 then become relevant. Of particular interest is clause 3.5.14.3(a). If the imposed deformations (v times delta) were not exceeded under the criteria in this clause then the seismic requirements need not be satisfied.

This raises essentially the same debate as to whether the columns complied with NZS 3101.

Related to these issues is the point that a council reviewing engineer is likely to have looked at the overall design and noted that it was a shear wall structure. The reviewing engineer would know that shear wall structures are relatively stiff and therefore probably fall into the category of a structure covered by clause 3.5.14.3(a) of NZS 3101. The conclusion flowing from this would have been that the gravity load columns, ie, all those in the CTV building did not need to comply with the additional seismic requirements of that code. On this basis the reviewing engineer would in my view have been justified in assuming the columns complied.

Q. Stop you there Dr O'Leary. You read the word "could" in the last line, it was "would"?

A. I meant "could" sorry.

Q. That should be "could"?

A. On this basis the reviewing engineer could in my view have been justified in assuming the columns complied.

For Christchurch which was in an area of only moderate seismicity, an assessment that went along the lines of "gravity only columns in a building with adequate shear walls should not need to be designed for seismic loading", had some reasonable basis.

Further observations of the Hyland Smith and expert panel reports.

I refer to section 11 of the Hyland Smith report and clause 5.14 of the expert panel report. With minor variations, the recommendations of both reports identified five potential vulnerabilities in large earthquakes derived from the performance of the CTV building in the February 22nd aftershock.

There is an inference that all five potential vulnerabilities were not particularly to the CTV building but were rather a generic issue related to the standards of the time. This is particularly true for non-ductile columns and diaphragm connections where there is a direct reference to the problem not being addressed in standards prior to NZS3101:1995 and NZS4203:1992.

Two dimensional elastic response spectrum analysis was allowed for buildings such as the CTV building. I refer again in paragraph 33 of my evidence. The two dimensional modal analysis noted in clause 3.4.7.1(b) of NZS4203 is what a two dimensional ERSA would produce.

How the two dimensional ERSA analysis was carried out was that the structure was modelled in three dimensions and the earthquake, then the earthquake input was applied along the two principal axes of the model as two separate analyses.

In ETABS at that time the diaphragms were modelled as rigid and a model of the structure was able to be constrained to respond in the direction of the earthquake input only. This meant that out of plane torsional effects were not reported in the output from the analysis and therefore torsional effects were not readily available from the model. Applying the NZS4203 prescribed accidental and actual eccentricities to the model, in combination, was intended to take account of torsional effects in a quasi three dimensional way.

Holmes peer review of the Hyland Smith report and the Hyland materials report.

On page 3 of Mr Holmes peer review report he discusses gravity frame ductility as required by code. I agree with the first sentence. "This issue is discussed in several places in the overall documentation but in my opinion is never clear".

I agree in general terms with Mr Holmes' peer review but I am of the opinion that he places undue emphasis on where design for limited ductility should be used. See paragraph 42E above for my understanding of when limited ductility would be used in the design of the CTV columns.

Q. Thank you Dr O'Leary. You have a second brief of evidence?

A. Yes.

HEARING ADJOURNS: 4.55 PM

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