HEARING RESUMES ON MONDAY 13 AUGUST 2012 AT 10.00 AM

JUSTICE COOPER:
Yes now Mr Reid.

MR REID:
Thank you Sir. On Thursday we'd just completed Dr O'Leary's first brief of evidence. So he's onto his second. So if he could please be re-sworn.

JUSTICE COOPER:
Yes.

ARTHUR JOSEPH O'LEARY (RE-SWORN)
Q. Dr O'Leary do you have your second brief of evidence in front of you?
A. I do.
Q. Would you please read your brief of evidence from paragraph 2 onwards.
A. This is my second statement of evidence. My relevant qualifications and experience were provided in my first statement of evidence.
This evidence comments on the evidence of Dr Nigel Priestley, Ashley Smith and Murray Jacobs as provided to the Royal Commission.
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.
Evidence of Dr Nigel Priestley.
I have read the evidence of Michael John Nigel Priestley. I generally agree with his evidence but I have some specific areas of disagreement and I comment further below.
Paragraph 15(a).
There is reference at paragraph 15(a) of Dr Priestley's evidence to the fact that he has little or no disagreement with the CTV report and expert panel report in relation to deficiencies in the design of the CTV building when considered against the code and the *best practice at the time*.

For any consideration of non-compliance in relation to the CTV building the bylaws and related standards are the governing criteria rather than some over-riding consideration relating to best practice. There are, in any event, considerable difficulties in my view in determining what is best practice within the profession at any particular time. If it is relevant to consider such issues, then in my view the enquiry should be as to the acceptable professional practice at the time.

Paragraph 15(b).

The critical vulnerabilities referred to in paragraph 15(b)(i) and (iii) of Dr Priestley's evidence are accepted. These two critical vulnerabilities are now seen as such but at the time the interior columns may well have complied with the relevant standards, although it is unlikely that the columns on grid 'f' complied.

The question of irregularities, lack of symmetry, paragraph 15(b)(ii), needs, in my opinion, to be put in the context of the understandings of the time and the understandable desire to have attractive and functional buildings. In my experience structural engineers prefer regular buildings with minimal eccentricity between the centre of mass and centre of rigidity, but this is impracticable for many buildings where layout functionality and associated considerations are major drivers behind the reasons for development.

As indicated in terms of paragraph 15(iii) and in my first statement of evidence, I have reviewed the diaphragm connection to the northern shear core and have concluded that the remedial work as designed would comply with NZS 4203 requirements in the north-south direction but not in the east-west direction.

Paragraph 16.

I accept all the areas of disagreement Dr Priestley lists in paragraph 16 of his evidence, except I do not have an opinion on his item (h).

Paragraph 18-26, 43-44, 63-71 and 75.

I have a general level agreement of level with much of Dr Priestley's evidence but I take the opportunity below to note some issues that reinforce my own views.

(a) Paragraphs 18-26 of Dr Priestley's evidence relating to the ERSA analysis.

(b) Paragraph 43 and 44. From my view of the drawings of the spandrel panels I have formed the view that they were too flexible to provide much resistance to the inter-storey drift along gridlines 1 and f.

(c) Paragraphs 63-71 on torsional eccentricity and the infill masonry on line a.

(d) Paragraph 75 on displacement capacity ERS analysis.

Paragraph 77 and 78.

I do not agree with Dr Priestley's paragraph 77 and 78 to the extent that he measures details in the building against "best practice to current state of knowledge". I have already commented on the difficulty in determining best practice in the profession. The design of the CTV building was only required to comply with the Council bylaws. Further, a Council design reviewer was only required to review the design for compliance with such bylaws and imposing a higher standard would not in any event be enforceable by the Council.

I now comment on paragraphs (a) to (g), paragraph 77, as follows: Paragraph (a).

NZS 3101:1982 covered ductile detailing for columns and the standard was widely held as being appropriate in 1986. In the next version of NZS 3101:1995 less rigorous requirements were introduced.

Paragraph (b).

As above the spacing of transverse reinforcement was covered by NZS 3101:1982 but made less rigorous in the subsequent version of the standard. This serves to illustrate the advancement of the normal practice of the profession as standards were modernised.

Paragraph (c).

This standard of cover to reinforcement of columns as provided in the CTV building is still allowed and is still the normal practice of the profession.

Paragraph (d).

I do not consider that there were high levels of axial compression in the columns. The columns complied in that respect with the standards and in my view reflected normal practice within the profession at the time.

Paragraph (e).

This paragraph relates to a stated lack of transverse reinforcement. Again, this is a matter relating to interpretation of NZS 3101. Up until a few years before 1986, this was not an issue that the profession had identified as being of concern. I was on an international committee in the early 1980s that was looking into the problem and as far as I can remember there was no basis for the design of beam column joints for shear even as late as the late ‘70s or early ‘80s. New Zealand researchers were right in the forefront of research into the problem of beam column joints in the late ‘70s and early ‘80s and there were several PhDs undertaken at both Auckland and Canterbury Engineering Schools to address the subject about that time.

Paragraph (f).

I accept that poor connectivity was a problem but the CTV building would not have been far out of step with common practice in New Zealand at that time.

Paragraph (g).

I agree that the lack of connection between floor diaphragms in the north core was non-compliant but the remedial work as designed would have brought it into compliance with NZS 4203 requirements in the north-south direction but not in the east-west direction. There has been work undertaken and incorporated in the current seismic loading standard NZS1170.5, that

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has shown that the standards current in 1986, that is NZS4203 and NZS3101 were non-conservative in their approach to this issue.

Paragraph 78.

Dr Priestley in his paragraph refers to a 1975 book author by Park and Paulay. This book was the basis for much of the contents of NZS 3101:1982. That standard had a predecessor (NZS 3101P) which was in common use from the early 1970s onwards. It was to the best of my knowledge a draft that was updated often as new research became available.

JUSTICE COOPER:

Q. Upgraded or updated?
A. Up- ungraded. Updated, upgraded.

EXAMINATION CONTINUES: MR REID

A. There may have been warnings about some issues in the Park and Paulay text book but I believe they would have been in the 1982 standard if they were thought important enough especially given that Park and Paulay wrote extensive parts of the 1982 standard. There were even some issues where the requirements were relaxed in 1995 version of 3101 including some aspects of column confinement.

Third statement of evidence of Ashley Henry Smith.

I have read the third statement of evidence of Ashley Henry Smith. In paragraph 6 of Mr Smith's evidence, he notes differences between himself and his co-author of the Hyland Smith report in the interpretation of NZS 4203 and NZS 3101. He further comments in the following paragraph 7 that members of the Expert Panel had differing interpretation of the standards in relation to column design.

As I have discussed earlier in my first statement of evidence clause 3.2.1 of NZS 4203 which Mr Smith refers to in paragraph 11 of his evidence is a general statement expanded upon in subsequent clauses. NZS 4203 refers to the material standards for details. Clause 3.2.1 does not in my view stand in its own right. The concrete material standard NZS 3101 is more specific and in the sequence of standard clauses between NZS 3101 and NZS 4203, NZS 3101 must in my view take priority as it is more specific and quantitative.

Much of Mr Smith's reasoning is based on load reversal into the inelastic range. This does not in my view take into account clause

3.5.14 of NZS 310 – sorry that's 3101 not 3102 1982 which sets the limit around whether inelastic or elastic performance applies. If the member (in this case columns) remains elastic at defined interstorey drift then none of the post elastic (inelastic) criteria apply.

Mr Smith does not in his evidence address in any detail the implications of clause 3.5.14. he makes passing reference to this in paragraphs 35 and 36 of his evidence but without discussion of the intent of the clause and its implications. In my view, his conclusion in the last sentence of paragraph 36 is not supportable because it is not based on the clear intent of clause 3.5.14.3 of NZS 3101.

Mr Smith at paragraphs 48 and 49 of his evidence briefly refers to beam column joints. There is a limited discussion of the beam column joint detailing contained in the Hyland Smith report. Mr Smith's reference to clause 4, 6.4.7 of NZS 3101 and as to what it means, is incomplete, as it does not include a discussion of compliance with clause 6.4.7(b), although he uses the more general reference of clause 6.4.7 to justify his opinion on confinement after quoting clause 9.8.4 of NZS 3101:1982. The columns complied with clause 6.4.7(b) of NZS 3101 if they were not required to be designed for earthquake loading.

Evidence of Murray Lionel Jacobs.

I have also read the evidence of Murray Lionel Jacobs. I comment on his evidence as follows.

Paragraphs 11 and 12.

Dr Jacobs concludes at paragraph 12 of his evidence that the building did not comply with clause 3.1.1 of NZS 4203 relating to symmetry. In considering this issue it is useful to refer to the commentary section C3.1.1 which is about mathematical models, mathematical methods is particularly relevant. It says, "Notwithstanding the availability of modern computers, considerable uncertainty exists in selecting a mathematical model representing the true behaviour of complex arrangements such as combinations of geometrically dissimilar shear walls…".

Clause 3.1.1 is the introductory clause to Part 3 “Earthquake Provisions” of NZS 4203 and in my view does not raise a specific standard compliance issue. This needs to be considered under clause 3.4.7.1. I have concluded in my first statement of evidence at paragraphs 32 and 33 that 3.4.7.1(b) was relevant to the CTV building.

JUSTICE COOPER:

Q. Does that mean that a designer didn't have to comply with clause 3.1.1?

A. Well 3.1.1 is an introductory clause which I will read out when I find it.

Q. Well I’ve got it in front of me you don’t need to read it out. My question is why it doesn't have to be complied with? Part of the standard isn't it? You saying it can't be complied with?

A. No it says “as nearly as practicable”.

Q. Yeah.

A. You know that's hardly a rigorous requirement as to whether it is, it’s, it’s not an absolute requirement.

Q. So do you not see limits around those words “as nearly as is practicable”?

A. There can be no –

Q. Could it be, could it be anything could it?

A. There are no rigorous limits given in the standard.

Q. So could it, it would have to yield to a client instruction for example to build a building that was not located symmetrically about the centre of mass of the building. Would that make, would that mean it was not practicable to comply with it?

A. It’s various shades of grey really because –

Q. Well is the practicability a matter of engineering judgment or is it a matter or something else?

A. It’s a matter of engineering judgment.

Q. Right.

A. But there are numerous cases where symmetry is, does not exist and the standard covers, 3101 covers the situation where symmetry is less than perfect, and also there's various clauses within 4203 that allows you to do certain things for moderately unsymmetrical buildings and I'll take you to an example of that for instance which is this clause on 3.4.7 horizontal torsional moments and it gives three situations related to torsion which is what symmetry is related to and it talks about structures for, in (a) if the structure is not more than four storeys high or for reasonably regular structures not more than four storeys high which are symmetric or of moderate eccentricity and then it goes on in (b) for reasonably regular structures which more than four storeys high with a high degree of eccentricity and then (c) is for irregular structures more than four storeys high and that's for any degree of eccentricity.

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Q. Yes, as I understand the pattern of this draughting, one would get into those clauses if for some reason it was not possible to comply with 3.1.1, isn't that the way it worked?

A. Well I maintain that 3.1.1, as far as it is practicable, doesn't give you a absolute requirement that you can judge symmetry by.

Q. So do you ignore it?

A. Sorry?

Q. Do you say you just ignore it and go to some other clause?

A. Well, you drill down further into the standard to find out what the quantitative governing criteria are.

Q. That sounds like "yes" to me? Might as well –

A. No you don't get –

Q. – might as well not be in there because you don't need to apply it because the phrase "as nearly as is practicable" means that you always have an out. Isn't that what you're saying?

A. Well how, I fail to see how you can judge "as nearly as is practicable" in a quantitative way?

Q. Well I thought you'd agree that was a matter of engineering judgement –

A. Yes.

Q. – as opposed to some other extraneous consideration?

A. But the engineering judgement is then governed by other clauses which are more specific in the standard which allow you to analyse your building and do certain things depending on their symmetry.

Q. Yes, so your position is really that clause 311 should be set aside because there's no practical way to apply it? One simply goes to other more detailed parts of the standard. Now I'm just asking the question, is that your view?

A. Yes.

EXAMINATION CONTINUES: MR REID

Q. Dr O'Leary you're at paragraph 27 I think.

WITNESS CONTINUES READING BRIEF OF EVIDENCE FROM PARAGRAPH 27

A. I've also read the evidence of Murray Lionel Jacobs. I comment on his evidence as follows. No it's 29 is it?

JUSTICE COOPER:

Q. Well you've read 29 too that's why I asked the questions I've been asking because of the comment, "The clause does not raise a specific standard compliance issue."

A. Yes.

EXAMINATION CONTINUES: MR REID

A. Paragraph 14.

WITNESS CONTINUES READING BRIEF OF EVIDENCE FROM PARAGRAPH 30

A. Dr Jacobs refers to clause 3.4.7 of NZS 4203. In my view and as noted above, the relevant clause for compliance purposes is clause 3.4.7.1(b) of NZS 4203. I disagree that clause 3.4.7.1(c) is the applicable clause for the reasons given in my first statement of evidence.

Paragraphs 15 and 16

Dr Jacobs refers to clause 3.2.1 of NZS 4203 and reaches the conclusion that the columns in the CTV building do not comply with that provision. I have considered this issue at clause 48 to 55 of my first statement of evidence.

Paragraph 18

I refer to my comments relating to paragraph 16 of Dr Jacobs' evidence above. In my view if the columns did not require to be designed as ductile then the spiral complied except possibly through the beam column joint.

Paragraphs 18 and 19

In my view the anchorage of spirals is covered by 3101 clause 5.3.29.3 of NZS 3101. I refer also to clause 3.4.1.2 of NZS 3101 (non-seismic design) where anchorage for non-seismic design is set out. I have examined drawing S14 and do not consider that there is significant congestion in the joint as a spiral for the joint and column above would start immediately above the bottom layer of reinforcing in the beam.

Paragraph 20

Dr Jacobs refers to clause 1.1 of the standard

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NZS3101 part one at page 15. I have addressed the minimum level of design expected earlier in the statement of evidence at paragraph 7 and 13.

Paragraph 31

I have considered Dr Jacobs' comments at his paragraph 31 relating to the reduced wall section of the north shear wall. I agree that this particular shear wall has a notch at level 1 so that it could possibly be classed as a slender wall. The question of whether a wall is stiff involves considering whether it is stiff or slender in relation to the frames in the same direction. There were only two frames spanning north-south which are quite flexible because of the long beam spans and flexible columns. On this basis, I consider the wall spanning north-south should be classified as stiff because they are relatively stiff compared to the frames on grid A and grid F.

Paragraph 36

Dr Jacobs concludes at his paragraph 36 that a critical important columns were not secondary elements. I disagree because the columns met the definition of a secondary element in clause 3.5.14.1 of NZS3101. I refer to paragraphs 38 to 43 of my first statement of evidence.

Paragraphs 39 and 40

The requirements in clause 5.3.32 is for shrinkage and temperature effects and not for diaphragm action. It is inappropriate to apply clause 5.3.32 to diaphragm action. There is a system of flooring 75 millimetres thick that according to clause 5.3.32 requires approximately half the reinforcing area that 664 mesh provides. Therefore that flooring system would have complied with clause 5.3.32 based on Dr Jacobs' reasoning. Yet as a diaphragm it would have been required to transfer the same diaphragm forces as the floors installed in the CTV building. In addition I note that in, it complied with the manufacturer's recommendations as indicated in paragraph 41 of Dr Jacobs' evidence.

Paragraph 47.

At paragraph 47 of his evidence Dr Jacobs refers to clause 7.3.4.3 of NZS3101 when discussing the minimum shear reinforcing in the column. He does not refer to clause 7.3.4.1 which has an exemption to the requirement of clause 7.3.4.3 members, for where the shear stress is less than half the shear that can be resisted by the concrete alone. The exemption in clause 7.3.4.1 is in my view satisfied for the interior columns. I refer to my paragraph 56 to 58 in my first statement of evidence.

Paragraph 50.

I agree with Dr Jacobs at paragraph 50 that the beam column joints are unlikely to comply. However if the columns could have been designed as non-ductile then 9.5.1 of NZS3101 would not apply.

Paragraph 53.

I refer to Dr Jacobs' paragraph 53 where he summarises where the CTV building design did not comply with the loading and concrete standards that applied at the time. I discussed each issue in turn as a summary of my evidence in terms of design and compliance issues:

(a) The issue of symmetry of the building layout is not in my view a question of compliance. There is no absolute requirement for symmetry. The building would in my view have been classed as a reasonably regular structure of moderate eccentricity. To this end, it was required to comply with clause 3.4.7.1(b) of NZS4203 which allowed two dimensional modal analysis of the structure. I refer to my first statement of evidence at paragraph 70 to 72.

(b) I believe that subject to assumptions and qualifications stated in my third statement of evidence the internal columns were not required to be designed for seismic loading (ductility). The columns in grid F however were required to be designed for seismic loading. I refer to my first statement of evidence at paragraphs 48 to 55.

(c) I do not comment elsewhere on this topic – this is C, I do not comment elsewhere on this topic but it does not in my view cause a compliance issue.

(d) The size and longitudinal reinforcing in the columns did comply with NZS3101. The discussion relating to the columns being too small and too heavily loaded is not supported by the standard.

(e) Some of the columns did comply with the minimum shear reinforcing of the standard but unfortunately not all of them. My calculations lead me to believe that the interior columns did comply but those on grid F did not comply. I refer to my first statement of evidence at paragraph 58.

(f) I agree that the diaphragm (slab) connection to the north shear wall did not comply with NZS4203 when the building was constructed, but as indicated in paragraph 10 above the design for the remedial work would bring it into compliance with NZS4203 in the north-south direction but not in the east-west direction.

MR REID:

Thank you Dr O'Leary. There is a third brief of evidence Your Honour that has been taken as read.

JUSTICE COOPER:

Q. Well yes but I have a question about it which can helpfully be asked at this stage. In paragraph 40(b) of your second statement you have said, "The internal columns were not required to be designed for seismic loading subject to assumptions and qualifications stated in my third statement of evidence." Now I can't – it will be my lack of understanding I am sure but I can't actually find the assumptions and qualifications to which that statement is referring. Can you help me with that?
A. I think that may be a misprint quite frankly. I think it is meant to refer to the first statement of evidence not the third.
Q. So we should change, "third," to, "first," in that paragraph?
A. Yes, yes please. Apologies for that.

CROSS-EXAMINATION: MR PALMER

Q. Mr O'Leary or Dr O'Leary in the 1980s was the code interpreted by local authorities and engineers to allowed widespread use of gravity frame shear wall structures such as the CTV building?
A. Yes it was an acceptable solution.
Q. And did that apply to many buildings in Christchurch?
A. I don't know I practiced mainly in Wellington at that time and in the practice I had, was involved with, we did a lot of multi-storey buildings up to about 30 storeys at that time and only one had a shear wall but that was peculiar to the Wellington industry.
Q. Would you say though that the approach was used in Wellington as well as –
A. Oh, yes it was, yes.
Q. And has the outcome of the earthquakes in Christchurch been to bring into question this particular design method or assessment or construction methodology?

A. I don't know quite how to answer that. I think it actually might indicate that some shear walls perform better than some frame buildings. It is the, sort of a mute point at the moment whether the shear walls or frame buildings are preferable in major earthquakes.

Q. So given your position in the industry and your knowledge of what is happening around you, is there more focus consequent upon the Canterbury earthquakes on this particular design methodology?

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A. I think there'll be a focus when codes are reviewed in the light of the Canterbury earthquake sequence that makes us go right back to our design philosophy regarding frame buildings and shear wall buildings.

Q. Just a point of clarification. Do I understand your evidence to be that you consider the CTV building to have had what would be described as moderate eccentricity?

A. Yes, it depends on how you model particularly the shear wall or the pair of walls on grid 1, one wall, and how you model the wall on grid 5. The wall on grid 5 isn't as flexible, isn't as stiff as some of the evidence we've heard would portray it. The wall on grid 1 is a lot stiffer than the corresponding wall if it was acted as a coupled wall because it does – didn't act as a coupled wall because the coupling beams were too stiff and too strong. It acted actually as a cantilever wall. So its relative stiffness is actually somewhat more than the calculations at the time might've indicated.

Q. But getting through all of that, your net, the net result of your evidence as I understand it is that it, you consider it to be of moderate eccentricity, is that correct?

A. It's moderate on the, on the far side of moderate.

JUSTICE COOPER:

Q. What does that mean?

A. Well it's certainly not symmetrical. It's moderate, tending towards highly eccentric. But it does depend to some extent on how you do the sums.

CROSS-EXAMINATION CONTINUES: MR PALMER

Q. But on the spectrum of possibilities you would put it in the moderate category, or that's certainly how I understand it?
A. I'd put it in moderate. I actually had a go at putting, looking at it in the definition of moderate and I found that it depended on some assumptions which were made in the modelling.

COMMISSIONER FENWICK:

Q. Yes because this determination of what's moderate or not. Now looking at that for me just standing back and looking at it. The north wall is 11 and a half metres long –
A. Yes.
Q. – or thereabouts?
A. Yes.
Q. The south wall, which you've indicated act as a unit, and I'm inclined to agree with you, it's just under five metres?
A. Yes.
Q. The ratio between the two is five metres is over the 11 metres is in excess of two. The stiffness is proportional to the length cubed, so it's something in excess of eight so that the north wall by my calculations, would you agree, is more, is stiffer than eight times the south wall?
A. I, I have a problem with the stiffness of the north wall because the north wall actually warps.
Q. Yes, and it also has flanges?
A. It has flanges yes.
Q. Which greatly increases relative stiffness?
A. Yes.
Q. I agree that warps, but even so it's hard to imagine, isn't it, being less than I would say 10 times the stiffness, I reckon it's more like 25. It's about 10 times the stiffness of the north to the south? I don't see how you can argue it down much. Would that be your feeling given the flanges?
A. Yes, it's relatively stiff, yes.

Q. Okay, so if it's 10 times as stiff and it's 25 metres between the two again, not worrying about you know the odd metre here and there. The centre of stiffness would be within about two metres of the north wall?
A. Well I don't think it's that close.
Q. Well if you push it so it's going to deflect uniformly, two walls with no torsion, you've got to push it so that if it's 10 times as stiff as that, you've got to have 10 times the load in the north wall as the south wall. So you have to push it at about two metres away from that wall. Where's the fallacy in that?

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A. I wouldn't have put it that close. I would have thought it was sort of somewhere between grid 3 and 4 possibly tending towards grid 4.
Q. Four metres out?
A. Well –

JUSTICE COOPER:
Q. Did you say grid, grid 4?
A. Grid 4's the –
Q. Sorry.
A. Somewhere between grid 4, probably closer to grid 4 than grid 3.

COMMISSIONER FENWICK:
Q. Right. Can you do the mental arithmetic to justify that figure if you accept that the wall, ratio of wall stiffness is of the order of 10 to one which I believe it's actually higher than that. Do you accept it's of the order of 10 to one?
A. Okay.
Q. So that would place us about one tenth of the distance away from the north wall wouldn't it, roughly, two metres, two and a half metres. I mean if you really want to you could put it three or four metres out, it doesn't worry, it doesn't affect the argument. Would you agree?
A. Maybe four metres.

Q. All right. Four metres out. Now if we do that then the load that would go on the south wall would be roundabout one eighth roughly of what you get on the north wall just sharing it out, the loading at the centre of stiffness?
A. The translational load.
Q. Translational load.
A. Yeah, yeah, yes.
Q. Now if we now put the centre at the force at the centre of mass, forget about the point 1(b) because we're doing this all by mental arithmetic –
A. Yeah.
Q. – centre of the mass.
A. Yeah, yep.
Q. 50% of the load, roughly, is going to go to the south wall and 50% to the north wall. Almost (inaudible 10:42:01).
A. Yes except the –
Q. The torsional stiffness of the finger walls if you like would alter that but I can tell you now it doesn't make a 5% difference.
A. It may be more than 5% but okay.

JUSTICE COOPER:
Q. Well have you done the calculations yourself?
A. I haven't done them rigorously no.

COMMISSIONER FENWICK:
Q. Look I'm prepared to accept, if you accept the 5%, and I've done them, it's less than that but if you accept that then we are going from a force of one eighth to 50% or 45% which by my calculation is about three times as high a load due to torsion as it is due to translation.
A. Okay, yes.
Q. Now if it's three times as high the moderate eccentricity was defined as when the load in that critical element went to 75% of the load due to translation. So we're going at about four times the moderate eccentricity level. Now it doesn't matter how you calculate it, whether it

comes to three or two and a half times, it's way beyond the 75%. Now clearly what I believe the code was saying is the swing you can have due to torsion should not induce the displacement of more than 75 of the displacement due to uniform application of load, of node torsion. Yet when you, however you do this calculation, whatever assumptions you make you're getting a figure which to me it looks like way in excess of 75%. Let's leave it and you can think about it. I can even lend you a calculator if you can't do it in your head, which I'm sure you can, if you think about it, but I'm doubting this eccentricity as being moderate and I don't know (inaudible 10:44:06) lead to a standard that makes any difference because they introduced this concept of moderate eccentricity but I can't see how it actually gets used but to me the eccentricity is extreme and I just wondered if you would agree with that or not.

A. I'll find the clause. I think I tend to agree with the, I'm trying to find the particular clause, the one on torsional eccentric – 3.4.7 –

Q. It's all on the commentary.

A. 3.4.7 I agree it doesn't actually, you can go to (b) even for a higher degree of eccentricity.

Q. That's right. I mean it's an odd thing to put in. To my way of thinking it's just a warning sign. Watch out.

A. Yes I'd agree with that yes.

Q. Yeah but I don't myself but I mean I, I would welcome your comments later on when you had time to think about the relative stiffness values and where it is and thank you for answering and I apologise for breaking into your thing but it seemed an issue that I felt should be raised.

CROSS-EXAMINATION CONTINUES: MR PALMER

Q. A slightly different question. This is the first time you've given evidence isn't it Dr O'Leary?

A. In this, for the Royal Commission yes.

Q. In this hearing and did, did your analysis of the building lead you to make any observations about the connections with the south shear, the south shear wall?
A. Yes. Not in a quantitative sense but I have looked at the south shear wall, photographs of it, and to some of the photographs taken by Mr Heywood and I think that the south shear wall may have separated from the slabs or the other way around a lot earlier in the collapse sequence than has been discussed.
Q. Are you suggesting there that that separation may have occurred before, for example, any column failure?
A. It would not surprise me, no.

COMMISSIONER FENWICK:
Q. The south shear wall.
A. Yes.
Q. You commented that it acted as a, you believe acted as a single unit not as a coupled wall?
A. I believe so.
Q. Do you have any calculations or what's the basis of your belief that that's how it behaved?
A. I added up the shear capacities of the coupling beams and I looked at the tension capacity of one side of the wall and it was a significant mismatch.
Q. With the gravity load on top of it?
A. The mismatch was such that the gravity load made relatively little difference.
Q. Thank you.
A. And I think there's further corroboration of that in that the coupling beams showed very little distress from the crack patterns.
Q. So the coupling beams were just too strong?
A. Yeah there was also something else –

JUSTICE COOPER:

Q. Hang on. Do you agree with that?
A. Yeah I think the coupling beams were too strong and also if you have a look at the east section of the wall between levels 1 and 2 you get a classical tension in shear crack pattern which –

COMMISSIONER FENWICK:
Q. You know about?
A. It's the first time I've ever seen it actually.
Q. So that PhD wasn't wasted?
A. It wasn't wasted no.
Q. Just one more thought to add to this. We did send out a minute asking people and to that was attached a diagram. So I wondered whether you'd looked at the implications to the floor slab along that coupled wall and the diagram we sent out is showing how due to the strain of the coupling, the diagram (inaudible 10:49:00) coupling beams and it goes on the diagonal, it actually elongates and that elongation actually then applies to the, pushes the floors apart. I just wondered if you'd, you know, if you'd looked at that well –
A. No I hadn't.
Q. – because that might have an effect.
A. No I hadn't but I did sort of feel that there was so little distress in the coupling beams that they probably didn't start to elongate. It looks to me as though they remained elastic.
Q. If you get a flexural crack there which I think you would have, then you've got elongation. It's small but it occurs right from the elastic stage.
A. Yeah except it was elastic, yeah.
Q. Look thanks that's very useful. We could put up the diagram showing it but I think you've pretty well indicated to me that I think you're agreeing there could be, there could be restraint due to those floor slabs?

1050

A. Oh, undoubtedly there's constraint.
Q. And of course –

A. It is very difficult to know how much constraint as you well know.
Q. Yes, sure but we are talking about a small constraints –
A. Mmm.
Q. So unless the concrete cracked, could be high, if the concrete did crack you've just got the mesh and of course the restraint is too big the mesh will fail but I mean that is the sort of order of things isn't it?
A. Yes, yes, yes.

JUSTICE COOPER ADDRESSES MR PALMER

QUESTIONS ARISING: MR PALMER – NIL

CROSS-EXAMINATION: MR MILLS
Q. Just a few questions Dr O'Leary, really as much as anything to just try and make sure that your views are entirely clear on all of the points that you are making about code of compliance and also the role of the reviewing structural engineer at the Council?
A. Yes.
Q. So first to just to confirm your general position on this, am I correct that your position is that at the date of permitting the CTV building was at least in some respects, not code compliant?
A. That is correct.
Q. But your view I take it is also that the Council reviewing engineer could not have been expected to pick up any of those areas of code, non-code of compliance?
A. They would have been difficult to pick up given the time apparently that was available to do the structural checks.
Q. So does that mean that in the end really you are coming down and saying that a reviewing Council engineer could not have been expected really to have picked up these areas of non-compliance that you've identified?
A. The environment at the time for the structural engineer doing the check would have been, this is a shear wall structure and there are certain

things I don't need to consider for a shear wall structure and with that environment I think it was a legitimate position to take at the time.

Q. All right I do need to ask you a third time and I wonder if I could just give me a yes, no on this which I am sure it must lend itself to. Is it your view that the areas of non-code compliance at permitting that you have referred to in your evidence are not ones which at that time a reviewing Council engineer could be expected to have identified? Your view, yes or no?

A. An experienced reviewing engineer could have?

Q. Could have?

A. Yeah, um...

Q. Do you think should have?

A. No I don't think so considering the environment at the time and the time available to do a check.

JUSTICE COOPER:

Q. Can I just ask you about the implications of that because here we have a building which in the only system available is given to the Council to check to see whether it complies with a standard adopted by the bylaw. Are you saying that there was something about this design that was too difficult for areas of non-compliance to be picked up by a Council reviewing officer?

A. Well the Council reviewing officer doesn’t do a full peer review.

Q. So –

A. There’ll be shades of significance of various issues, I think in the ideal world the Council officer should have picked those points up.

Q. But the system depends upon the Council verifying that a given design complies in those days with its building bylaw. That is what the system turns on, isn’t it?

A. Yes.

Q. Well was this, was the design of this building too difficult for Council officials to perform that function properly?

A. No.

Q. Well what is the point you are making?
A. Well to pick up whether it complies or not with the bylaw would require a level of peer review that is quite extensive, you know, calculations would have to be poured through for all the issues related to stirrup spacings, not just whether they comply with a particular section of the code but then you've got to look at the detail of how they comply for instance.
Q. So if you weren't going to do that, shouldn't you ask for some sort of certificate of compliance from the applicant for the permit?
A. Well I don't know whether there was – that certificate was asked for or not but it was often done – a design certificate was asked for.
Q. Well my question was, *should* it have been?
A. Well I think the Council reviewing officer or the City Council are entitled to rely on design certificates if they are provided.
Q. That is not answering the question Dr O'Leary. You remember the question I asked you?
A. Should the, a Council engineer –
Q. Should the Council confronted with the situation where, on your evidence, doing a proper check would be time consuming, should the Council ask for a design certificate?
A. Well if they weren't going to do the check in detail, a design certificate would have been appropriate to ask for.
Q. Should it have been asked for?
A. This seems to me to be a Council policy question rather than a technical question.
Q. Well I don't want you to answer policy questions Tell me as an experienced structural engineer what should have happened in your view. So far you have told us, I think, that to check this building properly would have required a very thorough check carrying out calculations –
A. Any significant building, yes.
Q. - which you say the Council can't be expected to do, is that right?
A. Yes.
Q. So if you are not going to do that, how are you going to work out that the building complies?

A. Well the only way you can, if you do that is to get a certificate from the engineer basically that the building complied.

Q. Well is that what the Council should have done in this case?

A. Yes, yes.

CROSS-EXAMINATION CONTINUES: MR MILLS

Q. Dr O'Leary are you aware that the evidence that the Royal Commission has heard in relation to Mr Tapper's role in this, might well suggest that he did identify some areas of real concern about the structural design of this building. Were you aware of that?

A. I was aware that he had identified some concerns and I have seen the letter that he wrote longhand and I have actually looked at the permit drawings that were signed off.

Q. Yes?

A. And tried to reconcile his questions and most of them, most of the questions were – well put it this way, some of the questions from the drawings didn't need to be asked because the information was there so he was obviously looking at a somewhat less than complete set of drawings.

Q. Yes I think that is likely. Do you think he was on to some significant issues that have now been identified by you as areas of non-compliance in relation to the diaphragm wall connection?

1100

A. Well it's difficult to know because I don't know, I've never seen the drawings he was actually looking at. So no, I couldn't answer that question, what might have been in his mind.

Q. There's no indication at all in the evidence, nothing in the evidence that would support a conclusion that at least in relation to the connections between the north wall and the diaphragm that there was any change between what he looked at and the permit drawings?

A. I can't remember what the question was that he posed about that.

Q. And he also identified an issue about column stirrups –

A. Yes.

Q. Would you agree that that could well be encompassing some of the issues that you've now identified as areas of non-compliance?

A. Well that boils down to the clause 3.5.14.1 and 3. Now if the decision on that was that it could go to part A rather than part B, then the issue is, I mean that resolves the issue. Because you can do – if it complies with part A it's not very difficult to show that the columns did comply.

Q. But if –

A. If it complies with C, ah, B then that becomes a totally different issue.

COMMISSIONER FENWICK:

Q. Can I just, sorry Stephen, can I just butt in now. If he didn't have a computer and he didn't have the calculations, and therefore he would not have had the output from the calculations. He actually would've had no simple way of determining the deformation applied to those columns to tell whether they were defined as primary or secondary members would he?

A. No, it appears he didn't have the computer output and so he didn't have a simple way, no.

Q. Forced up, he didn't have the calculations when he wrote that because he asked for the calculations?

A. Ah, I didn't pick that up actually, but no without the calculations it would've been very difficult. Well he would have had to sit down and do his own analysis.

Q. I mean he might well have had doubts about it, but I don't think he agreed me, he actually would've had no way of knowing. He might've said, "Oh, that looks suspicious," but he would've had no way of actually judging whether it was a valid secondary element, whether the columns or the beam column joints are valid secondary elements or not would he? Do you agree with that?

A. I think if you look at it from that direction that's right. If you look at it just as a shear wall building then you'd have a bit more measure of comfort.

Q. Yes, could well have been alarmed by the eccentricity. He could've spotted that one but – and realised that Paul was pushing things perhaps, but probably couldn't have –

A. No, no there's no easy way of finding that and first of all you've got to have the deflection which he wouldn't have had without the computer output, or the calculations, and until you get that there's nowhere to start basically. You've got to have that analysis.

CROSS-EXAMINATION CONTINUES: MR MILLS

Q. If we just assume for the moment that as Commissioner Fenwick really put it to you in part, that Mr Tapper saw enough to arouse suspicions and concerns, is that what you would expect of a Council reviewing engineer?

A. Well, he looks at the drawings and they were incomplete as we suspect. There would've been sufficient on them to arouse his concerns, and it did arouse his concerns. But we don't know between that stage whether the concerns he had were satisfied, well it is apparent that they were satisfied by the additional, or the set of drawings, the complete drawings that he got and there may have been some other verbal comment, we don't know.

Q. I have two questions for you coming from that. If the reviewing engineer identifies issues of concern, issues that cause some alarm about the structure of the building but is not in your view expected to go on and do detailed calculations, what in your view ought the Council reviewing engineer to do at that point?

1105

A. The, if he, if he, if he has concerns he needs to be satisfied either by calculations or by a certificate of some description from the designer.

Q. Right.

A. That his concerns are unfounded.

Q. And your view as I understand it is that the connections between the north shear core and the floor diaphragm were non-compliant at the time of permitting?

A. Correct.

Q. So if Mr Tapper had concerns about that north wall diaphragm connection he would, they in fact weren't meant by what was ultimately put in for permitting were they?

A. No.

Q. I want now just to run through with you what I think are the areas of non-compliance that you've identified just to be sure –

A. Okay yeah.

Q. – that we're on the same page on this. Now I just put it to you in, and you agreed but I'll just run through it again just to keep the list coherent. The first area of non-compliance that you identify is that diaphragm north wall connection at the time of permitting?

A. Correct.

Q. And you say that it's still non-compliant in the east-west direction even after the drag bars are installed?

A. Correct.

Q. You say that the columns on line F, they were non-compliant in relation to the loads that they were required to carry?

A. Well they were non-compliant in that they should have been designed as seismic, with additional seismic requirements of the standard so that means that the ductility requirements I mean in the code had to be met.

Q. Yes. Yes.

A. And also the shear in the column requirements needed to be met.

Q. Yes.

A. It's much less clear from the standard whether the shear requirements may have actually still applied.

Q. Well as I read your evidence you've said that they didn't comply, are you changing your view on this now.

A. Is that on shear sorry?

Q. Yes.

A. Yeah, all I'm saying is it's less clear from the standard but I believe they didn't comply.

Q. You believe they didn't comply?

A. Yeah.

Q. And that related to that or I take it the more detailed explanation for why it doesn't comply in relation to shear is that there inadequate confinement of those columns?

A. No the shear doesn't actually have, I mean in the, in the standard the shear and the confinement are different issues.

Q. All right so what is it about those columns that make them non-compliant for shear?

A. Just insufficient stirrups.

Q. All right.

A. And insufficient, and the stirrup spacing is too, sorry spiral in this case.

Q. Yes spiral.

A. The spiral was non-complying in both spacing and size.

Q. All right thank you.

A. I think –

JUSTICE COOPER:

Sorry I didn't hear the second word?

MR MILLS:

Spacing and size.

MR O'LEARY:

Spacing and size. It's in my first brief of evidence somewhere.

CROSS-EXAMINATION CONTINUES: MR MILLS

Q. Yes I realise this is going over some things you've said but because it's dotted through your two briefs I just want to get –

A. Yeah.

Q. – this –

A. Yeah.

Q. – ticked off really so we know exactly what your position is on this. Then you say and if you want to check your brief, this is your first brief at paragraph 69, as I understand it you say that the beam column joints on line F, this is the word you use, “are unlikely to comply”?

A. I didn't do the calculation. I had a look because a calculation actually needs again the computer output.

Q. Yes.

A. But from inspecting it I couldn't see how they could have complied.

Q. All right so based on the work you have done you’ve concluded it didn't comply?

A. It didn't comply yes.

Q. Thank you. Then, you then in your second brief at paragraph 32 I think take a step beyond that, I’ll just let you look at that.

WITNESS REFERRED TO SECOND STATEMENT

Q. To be sure that you’re happy with the way I’m going to put this to you. You refer there I think to the other beam column joints at least I think that's what you’re referring to and you say it’s, that possibly it’s non-compliant in all the beam column joints. Is that what you’re referring to there?

A. Yes. That's right.

Q. Now what is it that would, that you would need to do?

JUSTICE COOPER:

I’m sorry Mr Mills I’m lost did you say paragraph 32.

MR MILLS:

I did sir of the second brief, have I got the wrong paragraph number? It’s possible late at night that's what I’ve done.

JUSTICE COOPER:

Well whether or not it’s tiredness Mr Mills it doesn't seem to be the right.

CROSS-EXAMINATION CONTINUES: MR MILLS

Q. I agree it doesn't. I’ll find it for you later. But in any event the witness has accepted that that's what he said?

A. It's in – it's the last few words I say "and the spiral complied except possibly through the beam column joint".

JUSTICE COOPER:
Oh I see.

CROSS-EXAMINATION CONTINUES: MR MILLS
Q. Yes, yes. Well –

JUSTICE COOPER:
Q. That's, that's on the assumption that the columns did not require to be designed for ductility?
A. Yes. They certainly didn't comply if they were required to be designed for ductility.

CROSS-EXAMINATION CONTINUES: MR MILLS
Q. Yes well I was going to ask you that generally that these conclusions you've reached about non-compliance they all are on the assumption that ductility wasn't required?
A. Correct.
Q. Yes. And I think you've just confirmed to His Honour that if ductility was required then we've got a lot more non-compliance?
A. Yes.
Q. Now just on this question that I asked you about the statement that even on the basis of ductility not being required that possibly there's non-compliance in all the beam column joints. What would you have to do so satisfy yourself or perhaps put a bit differently to move you from possibly to probably not compliant?
A. I, I nee- I would need an analysis of the structure. The, the analysis of the structure that was done by ETABS would have most likely excluded the columns from the analysis one way or another.
Q. Yes.

A. Now what you, what I would need to be able to say whether they complied or not was another analysis that actually looked at what you might call the secondary frames to see how much they drifted and what alterations there were to the moment diagrams.
Q. Yes.
A. Because without that you can't, you can't do it. It's not just a simple calculation.
Q. Yes so when you say there that there's possibly non-compliance in all of the beam column joints, that's equivocal only because you haven't seen or done –
A. Yeah.
Q. – the full analysis that you'd require to firm things up?
A. That's correct.
Q. Right. On the view that you've taken that it was appropriate for the columns to be non ductile in the way they were designed am I understanding correctly that that's based on the level of interstorey drift that that building would be subject to?
A. That's correct.
Q. And am I also correct that the conclusions you have reached and worked from on the interstorey drifts are based on the calculations that Mr Harding did?
A. They're based on, yes, there's, there's some interstorey drifts which are tabulated about page S11 I think it might be.
Q. Yes. In Mr Harding's calculations?
A. That is the only indication of the interstorey drifts that I had.
Q. Yes and so that's what you've worked from?
A. That's correct.
Q. Now I don't know whether you heard Mr Henry's evidence but are you aware that by reference to Mr Henry' evidence and it was at paragraph 109 of Mr Henry's evidence that Mr Harding accepted that in determining the interstorey drifts he had only calculate, only calculated these from the centre of mass, are you aware that that's what he said?
A. Ah, yes.

Q. And he acknowledged that he hadn't done an interstorey drift calculation from the corners of the CTV building. Are you aware of that?
A. I was unaware that he had acknowledged that but I can understand the position he came from.
Q. Yes.
1115
A. Because if he used the 2D ETABS analysis it would not report the deflections –
Q. At the corners?
A. – at the corners.
Q. Yes. Do you agree that corner deflections would be greater than at the centre of mass?
A. Almost invariably.
Q. Yes. So your conclusion –

JUSTICE COOPER:
Sorry, that means, you just left the words "at the corners" out of the question.

MR MILLS:
Oh, did I?

JUSTICE COOPER:
Yes.

COMMISSIONER FENWICK:
Q. Look I think he used a 3D analysis not a 2D analysis?
A. Well, when I looked at the calculations I couldn't work out what had, he'd done because the calculations imported a set of deflections I knew not where from, but only could assume that they were from an ETABS analysis because there was nowhere else where they could've been likely to come from.
Q. Yes.

A. But in those days you modelled the building as 3D and then you constrained it to a 2D analysis, constrained it to act in only one direction, I mean in one plane.
Q. Yes I'm going by the fact that he followed Henry's calculations, and Henry said, "The centre of mass". Well the centre of mass was a highly eccentric building he was following. So if he was getting reflection of the centre of mass it must've been a 3D to have given that value. So that's probably need to check up –
A. I, I was unsure –
Q. – but I understand it was 3D.
A. – about exactly how the sequence of the logic ran.

CROSS-EXAMINATION CONTINUES: MR MILLS
Q. My recollection, although we can check it from the transcript, is that Mr Harding accepted that he had done a three dimensional modal analysis but that will be a matter of records.
A. Quite frankly I'm surprised at that, because in the mid '80s we were doing a lot of multi-storey stuff in Wellington and we, it wasn't until fairly late in the '80s that we were starting to use 3D analysis because the computing horsepower required was beyond what we had in-house. We could only really, there were some pseudo, and I say pseudo in that there was some reservation 3D analysis that you could do from 2D analysis which I never quite understood how they made the giant leap.

COMMISSIONER FENWICK:
Q. If it helps, this analysis was done on the University of Canterbury computer?
A. I realise that but I'm still rather surprised that they would've done a 3D analysis, but fine if they did, that was good.

CROSS-EXAMINATION CONTINUES: MR MILLS
Q. It was also a matter of evidence by Mr Henry about what he did on Landsborough House and again it will be a matter of record, but my

recollection is that was three dimensional modal analysis and that was provided to Mr Harding for him to follow when he did the CTV. But let me just come back to the –

COMMISSIONER FENWICK:

Q. Hang on, we can confirm it must've been a 3D analysis because Mr Henry came up with different shears on the walls for the box sections, so it was certainly rotating in torsion. So it must've been 3D.

A. I see, it was, yes.

Q. I think you can assume that because Harding followed the same steps, it would've been a 3D analysis.

A. Well I must admit I had my suspicions that something like that must've happened when I looked at the deflections in the north-south direction because they look to me as though they had a significant amount of shear component in them, a torsion component in them.

CROSS-EXAMINATION CONTINUES: MR MILLS

Q. If I can just come back and make sure this time I insert the missing words. I take it that you have agreed that corner deflections on the building will be greater than at the centre of mass?

A. Yes.

Q. So when you have given your opinion about the fact that the columns could appropriately be designed as non-ductile in the CTV building, and base that on Mr Harding's calculations of inter-storey drift, do you accept that that conclusion you've reached won't have taken into account the deflections at the corners?

A. Well, no that's not strictly correct because I've only said that it, they were possibly or probably allowed to be designed as non-ductile with the interior columns which weren't a great distance away from the centre of mass. The columns on grid F were, I don't think they complied anyhow.

1120

Q. Yes, is it possible that if you knew what the actual deflections were for that building that you might have a changed view on other columns as well?
A. Oh, if I knew what the individual column deflections were –
Q. Yes –
A. – I could do the calculations. If I knew that incidentally I would also have a lot more confidence in the displacements or drifts that were being reported by the computer analysis.
Q. Yes, thank you. Now the next point I just want to run through with you is that you have got a number of paragraphs where you deal with this question about whether it was appropriate to design these columns as non-seismic and I am going to put it to you that at least as I read your evidence and I will take you to the relevant paragraphs, that your conclusions that it was appropriate to design on a non-seismic basis seem quite tentative and I will just take you to these points to see if that really is your, is intended to be, rather tentative and equivocal on these points or whether it is just the way you have expressed it.
And first of all it's your first brief at paragraph 50 and you will see that you say there, this is commenting on the Hyland-Smith report, "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 is also in question and possibly incorrect."?
A. Yes.
Q. Is that – you are intending I take it to be less than certain about that?
A. Well, yes, because I was less than certain about the deflections that I started with and I had no way of checking those without doing my own analysis, I mean sophisticated analysis.
Q. Yes, so we are back to the point you and I discussed a moment ago?
A. Yes.
Q. Then at paragraph 55 of the same brief and probably for the same reason, you will see that you say there, "In conclusion I consider that the decision to design the columns as members not designed for seismic

loading *may* have been justified," and I take it once again it is a deliberate careful and slightly uncertain conclusion?

A. Yes, it is, yes. I did an analysis, a hand analysis taking the drifts from the calculations and they complied.

Q. Yes.

A. And I wasn't going to go out on a limb and say these drifts are absolute –

Q. Yes.

A. – because, well...

Q. So all of this is turning on the inter-storey drift issues, is it?

A. Yes.

Q. Yes. And probably again, this is just matter of ticking them off because I imagine you would give the same answer, paragraph 56 begins with an, "if". "If the interior columns," so same answer I take it –

A. Yes.

Q. – you are continuing this uncertainty –

A. Yes.

Q. – that you have already expressed?

A. The key is that clause 3.5.14 –

Q. Yes, similar statement at paragraph 58. I don't need to take you to that in detail –

A. No.

Q. – but it is the same intent to reserve your decision.

COMMISSIONER FENWICK:

Q. Apologies for butting in again, but it seems the right time. When you did your calculations on the drift?

A. Yes.

Q. Allowable drift, the commentary says if it is a moderate axial load, level, which I assume these columns would have been moderate to high axial load levels –

A. Moderate to high.

Q. – you should use I gross?

A. Yes.
Q. So when you did your approximate calculations did you use I gross?
A. I used I gross for the columns and 50% I for the beams.
Q. For the beams.
A. And I adjusted for the beam flange according to what the standard says although I have always had some suspicion about that.
Q. Well I agree, yeah but it did say take quite a wide section of flange –
A. Well they don't actually say you take quite a wide section but it still seems to me to be, especially with the, what is effectively a rim slab.
Q. Yes, yes, the common theme –
A. A bit hard to interpret –
Q. – there is a code of clause telling you how much flange to take and it is reduced in the next standard but it was –
A. Yes and I used the reduced one actually.
Q. You used the half value, right?
A. Mmm.
Q. Okay I think the half one came in the following standard, not the '84 or '82 standard?
A. I can't remember which one exactly but yes it's, but I used I gross.
Q. You did use I gross.
A. In the columns yes.
Q. Right and I think that, you probably agree with me, it's probably appropriate because the inter-storey deflection was based on 55% of the peak value I suspect this vouch for an error in code writing, I don't know, but two over two point two over SM while the ductility was four over SM, which gave you a smaller values?
A. I'm not quite sure where that particular sequence of calculations is coming from, I still have a problem with the 55%.
Q. Well we can talk about that perhaps some other time –
A. Yes, yes.
Q. – and I can tell you where it comes from.
A. It's not totally apparent from the code.

CROSS-EXAMINATION CONTINUES: MR MILLS

Q. All right, well I don't need to pursue the other paragraphs then where you've got similar slightly equivocal language because I think the underpinning and the reason for that's been established clearly enough.
A. Yes.
Q. I just want to ask you about this change of opinion that you've had on the effect of the drag bar installation that occurred. And I think there's a, I think you'll remember this is in your original brief, what I would call version number one because there's been several iterations, you didn't say what you've subsequently said, that you think there's still non-compliance in the east-west direction even after those drag bars were installed. I wonder what it was that caused you to have second thoughts about your earlier conclusion on that, do you remember?
A. Well my recollection of my original brief that was filed was that I did have, that I was concerned in the east-west direction. That's my recollection, but I don't have a copy of that evidence in front of me.
Q. Well I can find it for you easily enough over the break but that, I think it is clear that you've had a second view on it, but I take it you don't remember having any second thoughts on this issue?
A. No.
Q. All right, now when you say now that we've got compliance in the north-south direction –
A. Yes.
Q. – as a result of those drag bars going in, but not at the east-west, you are aware I assume that drag bars weren't installed on levels 2 and 3. You knew that didn't you?
A. I knew that yes.
Q. So when you say that there's compliance as a result of the drag bars going in, are you saying compliance at all levels including those where no drag bars were installed?
A. My calculations show that levels 2 and 3 actually did comply, because the forces in fact are triangular distribution up the, up the, and it was very close in fact that level 4 might've complied as well but I didn't

choose to go there. The drag bars had been put in on that level so fine. But no, 2 and 3 did comply without the drag bars.

Q. All right.

A. They do, yeah they did comply.

JUSTICE COOPER:

Dr O'Leary, I wonder if you could, over the adjournment, give further considerations to the questions about eccentricity that were put to you by Commissioner Fenwick as well?

HEARING ADJOURNS: 11.29 AM

HEARING RESUMES: 11.45 AM

CROSS-EXAMINATION CONTINUES: MR MILLS

Q. Doctor I just want to ask you one or two questions about clause 3.4.7.1 in NZS 4203, this question about the horizontal torsional moments that has been discussed already. Have you got that?

A. Yes.

Q. If you want it brought up on the screen I can, I'll just give you the number for it, the reference for it and then it can be brought up if people want to refer to it on the screen. It's ENG.STA.0018.53. I don't think we need to wait for it, you've got the copies in front of you haven't you? Now you've, just before the break there was the discussion about what Mr Harding had done and how he had interpreted this and I think it's a matter of record that he regarded this as requiring a three-dimensional modal analysis under 3.4.7.1(b). Now you agreed I think earlier this morning that there is room for differing interpretations of moderate eccentricity. That's, you accept that don't you?

A. Yes.

Q. And when you look at this clause 3.4.7.1(b) you agree with me that effectively what it is giving to the engineer who's making these decisions, a discretion as to how to proceed under that provision?

A. Yes.

Q. So there's an exercise of judgment involved in deciding what to do here?

A. Yes.

Q. And do you agree with me that in exercising that discretion or bringing that judgment to bear that 3.4.7.1(b) is recommending that if you have got a structure of the kind that's being described here at the beginning of that sub-clause that use three-dimensional modal analysis.

A. Yes.

Q. The result then would be that if Mr Harding made the decision that that was what was required here there could be no criticism of that decision.

A. No.

Q. Now one final point before I sit down. Were you here when Mr Nichols in the course of cross-examination said, and he was looking at the issues around Mr Tapper's letter, and he was asked about symmetry of the building and he said that, "If I had been checking that I would have been really concerned about that symmetry issue," and this, just for the reference, this is the transcript at point 20120806.79. We don't need to bring it up but just for the record that's the reference. Were you aware, were you here when he said that?

A. No.

Q. Okay well given that he did does it surprise you that Mr Nichols who was a former Council reviewing engineer identified that as something that would have almost immediately caused him concern?

A. The lack of symmetry of the north core was not an unusual situation for that type of structure.

Q. Yes it wasn't, I may not have put to you sufficient of what he said. It wasn't just the size of the north core it was what he described is that the shear walls have got to have some equitability between them. So it was the imbalance between those two walls that he's referring to. Does it surprise you that he would have identified that as an issue that he would have been really concerned about if he'd been checking the permit plans?

A. I, I think it is good that he identified it as maybe a problem. I wouldn't have got too excited about it personally because –
Q. Is it more than you would have expected by your hypothetical reviewing engineer that he was able to identify that very quickly?
A. Well I think, you know, it's good that he did and he was able to identify it quickly and, yes.
Q. And is that what you would have expected from your hypothetical reviewing engineer that they would have been expected to pick up that issue of the lack of equitability between the north and south shear walls?
A. Yes I wouldn't describe it as equitability but yes it was, it was there, it was on the drawings and he picked it up, yes.
Q. And you have I think agreed that you would expect your reviewing engineer to have identified that issue?
A. He'd have to in his own mind to have an assessment of what the building really looked like.
Q. Yes and that you would expect your hypothetical reviewing engineer, as Mr Nichols said, to be really concerned about that imbalance?
A. I wouldn't be, no I wouldn't expect him to be concerned about it. It wasn't an uncommon situation.
Q. What, to have the large north exterior shear wall or the lack of balance between the two shear walls?
A. The lack of balance between them.
Q. That was common at the time was it?
A. Not uncommon. Architects like to have their core at the back of the building, away from the public spaces.
Q. But it's the task of structural engineers isn't it to constrain architects to designs that are safe?
A. Yes, yes that's right but I, it's not, the issue isn't absolute whether it's safe or not safe. The issue is you do certain things to have the building comply with the standards, with the bylaw which is basically the, what's accepted as being appropriate.

Q. But the question from the perspective of the, your hypothetical Council reviewing engineer, was at the level at which you say they should be working you've agreed with me I think that what Mr Nichols identified by this lack of, as he put it, equitability –

A. Yes.

Q. – but this lack of, this imbalance between the south coupled shear wall and the big north core –

A. Yes.

Q. – that is something you would expect a reviewing Council engineer to note.

A. Yes.

Q. But are you also saying that isn't something that should have been expected to raise any alarm bells?

A. No I didn't say that.

Q. All right then put it in your words as to what you do say please.

A. It would, the reviewing Council engineer would note it and then he would look at the drawings and the calculations if he had them to see whether it was adequately accounted for, but it wouldn't necessarily, if I was confronted with something like that it wouldn't have raised alarm bells inasmuch as I would have looked at it and said, right, we've got an unsymmetrical situation, we make the appropriate design decisions based on that.

Q. And would you say that among those further enquiries that the reviewing Council engineer ought then to have taken would be to look closely at the way in which the shear walls were anchored to the rest of the building?

A. Yes that's a pretty fundamental issue.

Q. Yes and you've agreed earlier on that if that had been looked at closely it would have been apparent that the diaphragm connection to that north core was inadequate?

A. It would certainly have been apparent. It was suspect, yes.

Q. Yes all right. Thank you very much.

CROSS-EXAMINATION: MR ELLIOTT

Q. Dr O'Leary I think you've been here listening to much of the evidence that the Commission has heard. Is that right?

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A. No I was here last week and then at the start of this set of hearings but I haven't been in in between times.

Q. I'm just going to ask you to comment upon what appears to be Dr Reay's position about the evolution of the design of this building. The position appears to be Dr Reay's perspective that he knew Mr Harding had little experience in the design of multi-level buildings and the use of ETABS. He says that he did not know NZS 3101 as well as Mr Harding and it now turns out that Mr Harding did not actually know about the secondary elements clause 3.5.14. Dr Reay has taken on this design in his firm and let Mr Harding design this multi-level commercial building. According to Dr Reay he provided no supervision unless Mr Harding asked for it and no review and no check of the permitted drawings. Now I think you have 40 years of experience including design management so do you consider Dr Reay's conduct in taking on the design of this building and leaving it to Mr Harding to do acceptable for a sole principal in an engineering firm?

A. Well I imagine that it depends how much confidence he has in Mr Harding. It wouldn't have been something I would have done but I think to, for me to comment on Dr Reay's con- conduct in this is sort of inappropriate and not something I really qualified to comment on.

Q. Well you have made comments haven't you about the classification of the columns as secondary elements?

A. Yes.

Q. And so I'll ask you some questions about that?

A. Okay.

Q. And I'll start by referring you to the code of conduct BUI.MAD249.0529.

WITNESS REFERRED TO CODE OF CONDUCT

Q. It'll come up on the screen in front of you doctor, you've given evidence that you've read and you agreed to comply with the code of conduct?

A. Yes.
Q. So you understand don’t you that you have an overriding duty to assist the Commission?
A. Yes.
Q. Impartially?
A. Yes.
Q. You understand that?
A. Yes.
Q. And you understand that you’re not to advocate for any, for the party who has engaged you?
A. No that's fine. Yes.
Q. Do you understand that?
A. Yes, yes.
Q. And, and you are complying with that in the evidence you give?
A. Yes.
Q. I’m going to ask you some questions about the behaviour of the CTV building when exposed to earthquake forces?
A. Yes.
Q. Just a few questions on that?
A. Yes, yes.
Q. Now there's a diagram that can assist us BUI.MAD249.0486.1.
A. Yes.

WITNESS REFERRED TO DIAGRAM

Q. So this shows doesn't it that the CTV building structure consisted of firstly a north core, correct?
A. Yes.
Q. South coupled wall?
A. Yes.
Q. Frames on lines 1, 2, 3 and F?
A. Yes.
Q. And I think you’ve also said in your evidence that you think there was a part of a frame on line A is that right?

A. Well line A has a part frame in that it has columns and beams for about half height. The beams are about, I think they're level 1, 2, 3, ah, 2 to 3 there might be a beam at 4 as well which could be construed as a frame.
Q. Now just considering what would have happen to that structure in the earthquake on the 22nd of February and we're talking before collapse?
A. Yes.
Q. Before failure?
A. Mhm.
Q. Obviously when the ground moves the whole building moves correct?
A. Ah, the base of the building moves.
Q. Following which?
A. Following which the various floors move somewhat. They lag, their movement lags.
Q. And so earthquake loads are imposed on the whole structure?
A. Yes.
Q. And that includes the north core, south wall?
A. Yes.
Q. And the frames?
A. Yes.
Q. So those loads are not imposed upon one particular part of the structure are they?
A. No they're imposed related to the stiffness of the structure and those sorts of things.
Q. And during an earthquake parts of the structure will also interact with each other, is that right?
A. Yes.
Q. And they would transmit forces to each other?
A. Yes.
Q. And the forces that move through a building may have consequences in terms of individual members the way they are affected by those forces?
A. Yes, yes.

Q. Now those questions I've just asked you relate to the performance of the building in an actual earthquake?
A. Yes.
Q. I'm now going to ask you to consider this building from a design perspective?
A. Mhm.
Q. Now the code NZS 4203 required that a particular amount of load be calculated. Is that right?
A. Yes, yes.
Q. And the structure was then required under the codes to be designed to sustain that amount of load?
A. Correct.
Q. Correct? And in order to do that what the designers of the CTV building would have done was to assign that load to particular parts of the building namely the north core and the south wall but not the columns, is that right?
A. Yes. Ah, yes that's apparently what they did.
Q. So there's a difference, isn't there, between the actual behaviour of a building in an earthquake in which earthquake loads are imposed on the whole building and the design of this building which included assigning loads to certain parts of the building for design purposes, just, there is a difference isn't there?
A. There's a difference yes.
Q. Let's go to the definition section of NZS 4203:1984 ENG.STA.0018.18 please?
A. Yes.

WITNESS REFERRED TO DEFINITION SECTION NZS 4203:1984

Q. If you just highlight the elements definition?
A. Yes.
Q. So you see primary elements?
A. Yes.
Q. That refers, firstly you note that refers to the basic load resisting structure, agreed?

A. Yes.

Q. And then it goes on to talk about beams, columns, diaphragms and shear walls do you agree?

A. Yes.

Q. And it's not surprising that the beams, columns, diaphragms and shear walls are within that definition because those are the parts of the structure which as you've agreed would be exposed to earthquake loads in an earthquake?

A. Yes.

Q. Correct?

A. Yes.

Q. And you'll also see there that the definition identifies those as necessary for the building's survival when subjected to specified loads, do you see that?

A. Yes.

Q. Now if we go further down the page to the definition of horizontal force resisting system?

A. Yes.

Q. If that can be highlighted please, that refers to that part of the structural system to which the horizontal forces prescribed by this code are assigned and you'll note those two words there "are assigned" at the end?

A. Yes.

Q. And those words reflect the difference we've just discussed don't we between a basis load resisting structure and that part of the system to which horizontal forces will be assigned for the purpose of code compliance and design is that right?

A. Yes and that particular definition or no, although not as a definition is expanded upon in 3101.

Q. Now in the design process we could take it that the design engineer would be aware that there will be a seismic force resisting system in the building?

A. Yes, yes.

Q. And for design purposes what the engineers on this building have done is to assume that certain parts of that system can be put to one side as it were while assigning forces to other parts of the system, namely in this case the north core and the south wall?
A. Yes. That's right.
Q. Now I'll refer you to bylaw 105 ENG.CCC.004A.86.

WITNESS REFERRED TO BYLAW

Q. If we could just - .86
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Q. And if we can highlight clause 11.1.5 please, the whole clause. So doctor you'll see that this clause is talking about the general structural design method et cetera.
A. Yes.
Q. How it begins, and then do you see in clause (d), if you just read that to yourself please.
A. (Witness reads). Yes.
Q. So that clause is saying that, "Collapse and irreparable damage shall be avoided." Agreed?
A. Yes.
Q. "And the probability of injuring to or loss of life of people in and around the building shall be minimised," do you agree?
A. Yes.
Q. And we can take it for granted that the Council office in reviewing the permit application would have been aware of those words?
A. Yes.
Q. And should have had those words uppermost in their mind when considering the application for the permit.
A. I, I guess so, yes.
Q. And in fact the designers of the building should have been aware of that particular wording. Is that right?
A. Yes.
Q. And should have had the words of 11.1.5(d) uppermost in their minds. Do you agree?

A. Yes.

Q. And in fact every engineer designing buildings to which that bylaw applied should also have had those words uppermost in their mind, agreed?

A. Yes.

Q. Now I'm going to ask you some questions about the evidence you've given when compared to these words.

A. Yes.

Q. So I've written them down for you. I'm going to hand them up to you so you have them in front of you.

A. Yes. Thanks.

Q. Do you agree what you have in front of you says, "Collapse shall be avoided?"

A. Yes.

Q. "And probability of injury to or loss of life of people in and around building shall be minimised?"

A. Yes.

Q. Do you see the words refer to the risk to people both inside and outside the building?

A. Yes.

Q. Do you agree that –

MR REID:

I hesitate to interrupt Your Honour but I think it's important that the witness be, the whole of 11.1 be put to the witness if he's going to be questioned so directly about it. There's 11, in particular 11.1.6, his evidence has been that, his evidence is in relation to compliance with the standards and in relation to the bylaw he said that, that his understanding of the bylaw is that compliance with the standard is sufficient.

JUSTICE COOPER:

Q. Yes. 11.1.6. Do you see that Dr O'Leary?

A. Yes.

CROSS-EXAMINATION CONTINUES: MR ELLIOTT

Q. You've read that 11.1.6 Dr O'Leary?
A. Yes, yes.
Q. Although I think that the second, the relevance clause comes from NZS 3101:1982.
A. Yes.
Q. The words in 11.1.5(d) -
A. Yep.
Q. – refer to the possibility of risk to people inside and outside the building.
A. Yes.
Q. Do you agree that injury or loss to people could be caused by either a full or even a partial or localised collapse?
A. Yes.
Q. On the next page, .87 of the bylaw, I'm sorry, no we'll go back to the definition of primary elements, ENG.STA.0018.18.
A. Yes.
Q. And the reference to primary elements. Now as we've already identified the reference to, there is reference there to beams and columns and diaphragms and shear walls.
A. Yes.
Q. So you would agree that if one was looking to minimise injury or loss of life –
A. Yes.
Q. – the designer would want to give close consideration to the design of those parts of the structure?
A. Correct.
Q. And, similarly, a Council officer reviewing a permit application should give close consideration to the design of those parts of the structure. You agree?
A. Yes within the limits of what he's able to do.
Q. Now I'm going to refer you to a section of the commentary from NZS 3101, ENG.STA.0016A.
A. 3101.

Q. Point 46. It'll come up on the screen doctor, .0016A.46.
A. What's the clause?
Q. Do you want to see the original clause. I'm not going to be asking you a question about the commentary as it relates to the clause. I'm just going to ask you some questions about an element of the commentary which is more general. So let's see how we get on.
A. Okay but I, I like to see the commentary in context thank you.
Q. Yes. Well we're looking at the commentary to 4.3.1.2.
A. Yep.
Q. All right.
A. Yes.
Q. And if we could highlight the bottom right-hand paragraph please of the commentary. Now firstly that includes reference to columns with confining reinforcement complying with 6.5.4.3(a) or (b). I can take you to these if you want but do you accept that reference to those clauses is a reference to the seismic or ductile detailing provisions of columns?
A. Let me have a look and see what they say. Yep.
Q. And that relates to the transverse reinforcement of columns.
A. Yes.
Q. And it's in the section on, Requirements for Members Designed for Seismic Loading.
A. Yes.
Q. And that includes reinforcement relating to plastic hinge regions of columns.
A. Yes.
Q. That's what we're talking about in that particular section of the commentary. So would you just read to yourself that section down to the words "readily attained" and then I'll just ask you to comment.
A. (Witness reads). Yes okay.
Q. So do you agree that firstly this is telling us that for the columns designed with the ductile detailing we've just described –
A. Yes.

Q. – the confinement leads to increased effective concrete strength. It's telling us that?
A. Yes.
Q. And you agree with that?
A. Yes.
Q. And it's telling us that it would make columns more effective in bearing gravity loads as well as loads relating to earthquakes. Do you agree it tells us that?
A. Well it does, yes, I mean it doesn't say specifically that but it does.
Q. And you agree with that proposition?
A. Yes.
Q. And it tells us that ductility is demonstrated, meaning columns designed in that way are more likely to retain strength for longer even when exposed to vertical accelerations and P-delta effects?
A. I don't see where it says will retain their strength for longer.
Q. All right well do you agree with the proposition that ductility will lead to the columns retaining strength for longer even when exposed to –
A. You don't mean on a time basis, you mean as they get loaded –
Q. Yes.
A. – more heavily?
Q. Yes.
A. I took you for longer as being related to time.
Q. No, no as loads increase.
A. Okay, yes.
Q. Now back to the bylaw.
A. But that's not all that the, that clause says.
Q. No. Well it may say other things but I'm asking you about those things. Now back –

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A. Well what it says actually that if you provide ductility in columns and it doesn't say gravity loaded columns, you're allowed to use a ø which is the capacity reduction factor of .9 rather than the relatively heavily loaded columns that aren't confined, you use a capacity reduction factor of .7.

Q. Yes, and that's the area Mr Smith has commented on in his evidence. He said capacity reduction factors?

A. Yes but this is not only in seismic design, this is, if you, if you confine a column to the requirements of probably 6.5.4.3(a) or (b) you've got to be a bit loose on that because I haven't actually read exactly what it says, but the philosophy is that if you confine a column, well confine a column, then you can actually design it for more strength than you can if you don't confine it. And that's just for a gravity loaded column, it's got nothing to do with seismic.

Q. Yes, thank you, now back to the bylaw, ENG.CCC.00448.87?

A. Yep.

WITNESS REFERRED TO SLIDE

Q. When it comes up, oh, this is a different page from in the bylaws, just highlight the ductility, 11.2.5.2. The words there, "The building as a whole and all of its elements that resist seismic forces or movements et cetera shall be designed to possess ductility." You've read those words?

A. Yes.

Q. Now looking at the words that I've written down which you have in front of you, if an engineer was to treat those words in front of you as his or her objective?

A. Yes.

Q. And the elements they should regard as those resisting seismic forces or movements are the same ones referred to as primary elements, namely beams, columns, diaphragms and shear walls. That's right isn't it?

A. Um, could you repeat that question?

Q. If the designer was using the words they've written in front of you as their objective, the elements they should regard as resisting seismic forces or movements are the same ones in the definition of primary elements, namely beams, columns, diaphragms and shear walls, it must be right mustn't it?

A. Well according to 4203 that's correct.

Q. Or to put it a different way, if one was looking to minimise the probability of injuries to or loss of life and avoid collapse, it would be less consistent with that objective to not treat columns as elements resisting seismic forces wouldn't it?

A. You could put it that way.

Q. Now your evidence is that the CTV columns can be classified as secondary elements under clause 3.5.14.1 –

A. Correct.

Q. – of NZS 3101 –

A. Correct.

Q. – I'm going to invite you to consider that clause, ENG.STA0016.28?

WITNESS REFERRED TO CLAUSE

A. Yes I know, I've got the clause here actually, yep.

Q. Firstly, if we can just highlight the bottom of the left-hand column which, in which the wording begins?

A. Yes.

Q. So the clause begins, "The secondary elements are those which do not form part of the primary seismic force resisting system, or are assumed not to form such a part, and are therefore not necessary for the survival of the building as a whole, et cetera."?

A. Correct.

Q. Now firstly, those words, especially the section saying "are assumed" –

A. Yes.

Q. – that captures the distinction we've discussed earlier doesn't it between the system or structure which in fact is subject to earthquake loads and those parts which for design purposes the engineer will assume not to form part because loads have been assigned to other parts?

A. Correct.

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Q. And the words, "...or are assumed not to form such part and are therefore not necessary for the survival of the building as a whole". It is right isn't it that those words link the assumption about whether the element will form part of the structure for design purposes to the question of whether the element is necessary for survival of the building?
A. As a whole.
Q. Agreed?
A. As a whole,
Q. As a whole?
A. Yes, the words, "As a whole," are very important.
Q. Well we will come to that. Let's consider the columns in the CTV building. Do you agree that they supported the gravity load?
A. Yes.
Q. They supported the whole floor slab at each level between lines 1 and 4?
A. Individual ones didn't. The combination of all of them did.
Q. The columns?
A. Yes.
Q. I am not talking about a particular column?
A. Well...
Q. I will let you know if I am.
A. Well I go back to, "As a whole," you said the whole of the floor, each individual column does not support the whole of the floor.
Q. Are you looking to argue a proposition I haven't put to you yet? Are you anticipating what I am might put -
A. Well I am wondering when the proposition is coming.
Q. You are answering it before I put it?
A. Well I am just getting –
Q. Why would that be if you're assisting the Commission?
A. I am making sure that the whole of 3.5.14.1 is looked at rather than taking just bits of it.

Q. All right, well I am going to work through it. I am talking about the columns –
A. Yes.
Q. And you agree that the columns supported the whole floor slab between lines 1 and 4?
A. Do you want me to be pedantic about this because in fact the whole floor slab was not supported on the columns –
Q. (inaudible 12:21:57) –
A. – part of the floor slab was supported on the walls.
Q. Apart from that part supported by the walls?
A. Mmm.
Q. Columns supported –
A. Being supported by the columns.
Q. That would have been evident I take it to a Council reviewing officer?
A. Yes.
Q. Very clear?
A. Yes.
Q. And it would have also been evident to a Council reviewing officer how much vertical and horizontal steel had been drawn for columns and beam column connections, very clear from the drawings?
A. Yes.
Q. Well let's assume for a moment that the designer of the CTV was entitled to treat the columns of the CTV as not part of the intended primary structural system?
A. Yes.
Q. Refer you to ENG.STA.0016.24. This is, I am going to refer you to clause 3.5.1.5.
A. S3101 is it?
Q. That is correct.
A. 3.5.1.5 is it?
Q. Yes.
A. Yep.

Q. "Consequences of failure of elements that are not part of the intended primary system for resisting seismic forces shall also be considered."
A. Yes.
Q. Again I am saying well let's assume that there was an entitlement to consider columns as not part of the intended primary system?
A. Yes that is right, yes.
Q. This clause requires the designer to consider the consequences of failure of the columns in that case, doesn't it?
A. Yes.
Q. And for the purpose of our discussion I will define failure as ceasing to perform the function of holding the floor slabs up. Can we work with that definition?
A. Yes that is one definition.
Q. So the consequences of failure in that sense would certainly be injury or death to people in and around the building?
A. Probably, yes.
Q. And those consequences should have been clear to a Council reviewing officer?
A. Yes, yes.
Q. Going back to clause 3.5.14.3(a), ENG.STA.0016.28?
A. Yes.
Q. If we look in particular at subsection 3, so if you highlight the bottom right where it says 3.5.14.3?
A. Yes.
Q. Down to (a) and (b)?
A. Yes.
Q. You have looked at Mr Harding's calculations, have you?
A. Yes.
Q. He did not do any calculations to determine whether the columns would be elastic at V delta, did he?
A. No.
Q. A Council officer looking at the calculations could have determined that, there was no such calculation?

A. Yes.

Q. By looking at the calculations?

A. Yes.

Q. Or if there was insufficient time to do so, he or she could have simply asked Mr Harding, have you carried out that calculation?

A. Yes.

Q. To which the answer would have been, “No.”?

A. Well, no because there’s actually a note in the calculation to that effect. Well it doesn’t actually say that but it is a broader note that basically says something along the lines of not required to have a ø of 0.9 or something like that, I can't remember exactly what they are.

Q. Well I think he didn't know about clause 3.5.14 so it would be surprising if he carried out that calculation, wouldn’t it?

A. Possibly, I don't know.

Q. Now the calculation of delta, can be determined under the code by either static analysis or spectral modal analysis, is that right?

A. Yes.

Q. I am just going to ask you to look at a part of the foreword to NZS4203 ENG.STA.0018.14.

A. Yes I have got that.

Q. Just highlight the paragraph beginning, “Designers should...” which is about halfway down the page?

A. This is on page 9, yes?

Q. Yes.

A. Yes.

Q. So this is telling the designer firstly that the precise properties and materials are unknown, do you agree?

A. Correct, yes.

Q. That the precise properties of structural elements are unknown?

A. Yes.

Q. That interaction between elements is uncertain?

A. Yes.

Q. That the total design technique involves a degree of imprecision?

A. Yes.
Q. And that the use of more advanced techniques can lose validity?
A. Yes.
Q. Is it right also that the modification factor V in V delta itself also incorporates an assumption, namely the assumption that there will be equal displacements for the building in both its elastic and inelastic states?
A. No.
Q. You don't agree with that?
A. No.
Q. A building could experience displacements greater than V delta in an earthquake couldn't it?
A. Yes.
Q. And if the columns are not designed for ductility there would be a point where they reached their elastic limit if the loading was to continue to increase?
A. Correct.
Q. And at that point failure would be much more rapid than if designed for ductility?
A. Correct.
Q. So I want you to consider the effect of fixing V delta as the threshold by which the ductility of columns is determined?
A. Yes.
Q. So firstly it means that compliance with V delta could only be determined by various calculations?
A. Yes.
Q. And those calculations have many inputs or have some inputs?
A. Yeah they have significant inputs, yeah, yes.
Q. And engineers carry out those calculations?
A. Yes.
Q. And presumably engineers may disagree about the inputs?
A. Ah, possibly.

Q. And where compliance is an issue, there may well end up being disagreement about inputs as in fact we have now got here at the Royal Commission –
A. Correct.
Q. – in the session about to come?
A. Yes.
Q. So I just want you to consider the following then I will pose a question or two. If the columns in the CTV could be classed as secondary, you say they would not need to be designed for ductility if elastic up to V delta. That is the position, isn't it?
A. That's the position of the standard.
Q. But as you have said, in principle columns could and in fact would fail once beyond their elastic limit if loadings continue to increase?

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A. Well exactly at what point they would fail but they, that's the limit set in the code. They will, they will survive for some increase in load but not particularly well define how much.
Q. The validity of the calculation of V delta is affected by properties of materials being unknown?
A. Approxi –
Q. Has that happened before?
A. – approximated, yes.
Q. By the interaction of elements in a building being extremely uncertain?
A. Yes.
Q. By the total design technique involving imprecision?
A. Yes.
Q. And by advanced techniques of analysis potentially losing validity?
A. Yes.
Q. Let's have a look at those words in front of you again that I've given you in relation to loss of life et cetera, on the piece of paper?
A. Oh, right, okay yep.

WITNESS REFERRED TO DOCUMENT

Q. Would you just read those words out please?

A. “Collapse shall be avoided.”

Q. And?

A. “Probability of injury to or loss of life of people in and around the building shall be minimised.”

Q. Yes, so my question is: which of the following is more consistent with satisfying those objectives? On the one hand classifying the columns as secondary elements, in which case the identification of V delta becomes critical in determining whether they should be designed for ductility, notwithstanding the uncertainty we’ve discussed. Or, treating the columns and beam column connections as part of the primary seismic force resisting system and detailing them for ductility?

A. But the columns weren’t part of the seismic, the seismic design reinforcing system.

Q. Which of the following is more consistent with the objective in front of you?

A. But I don’t see how I can give you a definitive answer to that when the question in fact is, doesn’t represent the, what the columns were. The columns were not part of the seismic design reinforcing, seismic resisting structure.

Q. By design?

A. By design and by the very commonly accepted principles of structural engineering.

Q. Design principles you’re talking about?

A. Design principles, yes.

Q. And the design as we’ve discussed is different to what actually took place, which is that the whole building including the frames were exposed to earthquake loads?

A. They were exposed to earthquake loads, but they weren’t part of the seismic design resistance.

RE-EXAMINATION: MR REID

Q. You were questioned by my learned friend, Mr Mills, about the relationship between the outcome of the ETABS analysis as recorded in

the Harding calculations and the corners of the, and the columns on the corners of the building, do you recall that?
A. Yes.
Q. How did you account for that when you did your hand calculations?
A. Well if you do a 2D analysis, an ETABS 2D analysis, you can't directly account for that, in fact you have to do a supplementary analysis of the frame to pick that deflection up. Because what the 2D analysis says is in one direction it deflects so far and it doesn't give a rotation and it doesn't actually give the forces resisting rotation. So in the east-west direction for instance, it would report the displacements in the drifts in an east-west direction, but it would not report any displacement in the north-south direction. And for the columns on centre line F you'd have to, you need the north-south ETABS analysis.
Q. So were you able to account for that issue in your calculations?
A. No, I couldn't.

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QUESTIONS FROM COMMISSIONER FENWICK:

Q. Yes, the north core wall stands out from the building?
A. Yes.
Q. And creates a couple of re-entrant corners?
A. Yes.
Q. Now when they're talking about regularity in…?
A. Yes.
Q. If I can find it, 3.4.7 I think, it highlights the problem of re-entrant corners in the first section of the paragraph?
A. 3.4.7.

WITNESS REFERRED TO PARAGRAPH 3.4.7

Q. See 3.4.7.1?
A. Yes.
Q. I don't know if we need to bring it up or not?
A. Yeah. I see that.

Q. Horizontal torsion et cetera it highlights in there the, the, the problems you get with re-entrant corners? Do you think that re-entrant corner could have raised alarm in a building checking inspector?
A. Is this in a commentary or in the –
Q. It's in the commentary.
A. Oh it's in the commentary yeah.
Q. It's, it's first, first under –
A. Yeah.
Q. Should bring it up I think, this is NZS –
A. No I see yeah no it's I, I've got it.
Q. NZ - just it's ENG.STA0018.53?
A. Sorry, Commissioner Fenwick the, the question is whether that would raise alarm bells?
Q. Yes.
A. In this particular building? It wouldn't've to me because I wouldn't've considered those re-entrant corners to be alarming or particularly significant. The – my understanding of the code and it's, it's discussed somewhat more as my understanding in the second to last paragraph of that commentary clause where it says "the severely eccentric buildings with LTU or similar irregular floor plans seismic separation of the wing is" and my understanding of why it's there is to stop projecting bits of the structure having a life of their own effectively by introducing secondary modes of –
Q. And you don't think that action could have occurred, you know you've got –
A. Not of –
Q. – fairly stiff walls and then you've got flexible beams and frames spanning out to the east…?
A. Not necessarily design of getting the loads into that wall were appropriate because I put it another way you could have a very regular building in plan that filled in those corners that would have been, no-one would have argued wasn't irregular, was irregular in plan but the actual structural response of the building wouldn't have changed. It would have

made it easier to get the loads into that core lot, that wall on grid 5 but that's all that would have changed.

Q. It’s just someone else brought up that as an issue. Right now the, the other point you highlighted is the lack of coupling between the wall and the floors?

A. Yes.

Q. In the east-west direction?

A. Yes.

Q. Where you have a high shear in the wall?

A. Yes.

Q. And a moment that goes, bending moment that goes into the floor?

A. Yes.

Q. Did you do any calculations to see if the actions could be resisted or did you just look at it and said, “Oh I don’t like it”?

A. No, I’ve done the calculation to have a look at that and the calculation was effectively done for the floor slab between the two western north-south walls because there was effectively no other.

Q. There's only the, only the floor between wall C and wall C D (inaudible 12:3:20)

A. Yeah there's a little bit of floor which is really only a stair landing between –

Q. That's right.

A. – the next two which I think is legitimate to discount because I can't actually see what it would do.

Q. Yes.

A. And you can't in my view, and I’ve done the calculation, develop a shear mechanism to get the load from grid line 4 to grid line 5 either by conventional truss action because you can't develop a chord strength or by the more modern strut-tie model because you still haven't got a chord or you actually have a chord but it’s spread over I think I did a calculation about two or two and a half metres of height of the wall which is not legitimate in my case because you’ve still got to develop that out into the slab beyond grid line 4.

Q. Now the actual floor they –
A. Knocked a hole in it as well.
Q. Yeah the floor itself, that's right yes, but the actual floor tore didn't it outside line –
A. Yeah.
Q. – outside line 4 into the floor?
A. That's what I find very strange which makes me believe that there's, the, the –
Q. The bending moment has to go, bending moment and shear have to go into that area don't they?
A. Yeah well that's why I have a, a conviction quite a strong conviction that the wall on grid line 5 was never loaded in any, anywhere near its capacity.
Q. So, so do you think that that tear probably started fairly on, early on in the earthquake and that would change the flexibility of the whole structure wouldn't it?
A. It would if that tear started –
Q. (inaudible 12:41:11) necessarily go the whole way but it would just, just propagate a metre or two into it and you've got a much more flexible –
A. It's like, like cracks in any, any structure they tend to sort of alter the, the response significantly.
Q. So that might well have protected that north wall from quite a lot of loading going into it?
A. Quite possibly.
Q. From the east-west direction?
A. Quite possibly.
Q. Increase the period?
A. Yes.
Q. Increase the deformation on the columns?
A. Yes.
Q. And possibly even been a trigger for the collapse?

A. I don't think it was a trigger for the collapse but I wouldn't discount it. I think the trigger for the collapse in fact was associated with the south wall.

Q. Well you may be right but if the north wall became more flexible –

A. Yes.

Q. – because of that tear?

A. Yes.

Q. It could've increased the –

A. It would increase the drift.

Q. Could've increased the torsional actions going through I suppose couldn't it it's rather hard to see how it could have increased the, the force?

A. It might have actually reduced the torsional action but it certainly would have increased the drifts.

Q. Mhm, yeah. Have you had any more thoughts about the maximum eccentricity and the effect there?

A. Well I had to get back to my fundamentals. The extreme condition for that particular style of structure is actually to have the centre of rigidity on grid line 5, which means that all the load that is taken by that wall but you've got no torsional resistance.

Q. If we accept there are warping and we're not looking at the centre of rigidity assuming (inaudible 12:43:00) then the centre of rigidity will move in?

A. Yeah, yeah, but don't we consider the translational effect on the walls is not to do with the stiffness of the walls. The stiffness of the walls is only taken into account when you're looking at the tortional restraint, in other words it what dictates the difference between the centre of mass and the centre of rigidity and that's where the loads start to be unequally shared or more or less unequally shared.

Q. I don't know if I quite follow what you're getting at?

A. Well if you look at the translational effects you've got a shear from the earthquake in the east-west direction. You've got let's assume for simplicity that the centre of mass is equi-distant between each of the walls –

Q. Yep.

A. – and half the shear goes to each wall.

Q. Yep.

A. Yeah, and then so that the load on the south wall varies somewhere between V over 2 and V, and a load on the north walls spare- similarly.

Q. Well if we're –

A. (inaudible 12:44:14)

Q. – if we're doing in terms of statics it's V over 2 to each approximately?

A. Yeah, that's right yeah. And then you add –

Q. But –

A. – your torsional restraint is a plus or minus on each wall.

Q. No because when you do it by statics V upon 2 to each the more flexible wall's going to move much further than the stiffer wall so it's got the torsion in it?

A. Oh yes, yes, yes I agree with that yes.

Q. So –

A. But, but what you do is you add the translational effect which is V over 2 on each.

Q. Right.

A. Roughly.

Q. Translational effects not V upon 2 it's – the translational effect if you only have the same translation in both walls is proportional to the stiffness of those two walls?

A. I've got to have another think about that.

Q. Okay, look thanks.

1245

A. But the extreme condition is in fact the centre of rigidity on gridline 5.

Q. Yes, look I agree with you. It's almost, in fact one can almost say it's a metre outside it but we know it's not because of the –

A. I agree.

Q. – the fact that the sections going to warp –

A. Yeah that's right.
Q. – so it's got to move in, yes. But okay.
A. But it doesn't actually make any difference to the interpretation of those two clauses on the torsion.
Q. I agree.
A. Irregularity.
Q. I agree, but someone looking at it would well say, "Hey, it's a very high, it's more than moderate, it's very high." Perhaps that should raise alarm bells?
A. Yes, yes, it's all a matter of degree, yep.

QUESTIONS FROM COMMISIONER CARTER:

Q. From the beginning of the understanding of this building, the importance of the south shear wall has been discussed and it was an element that was added to the original concept and then became a very important part of the structure. Now I just want to try and reconcile the forensic work with the, which included the observations of witnesses to the collapse, and indeed even your own recent statement that you were of the opinion that the south shear wall had separated from the structure rather early. In fact quite early in the process I think you felt. And yet, calculations which are used to determine where the weaknesses are in a structure, you'll design them so that they were capable of carrying the loads that have to be sustained. Can you reconcile how that south shear wall would've separated from the slab in terms of the translational loads that were transmitted into it through the east-west motions and then the twisting on the slabs and the north-south direct loads that had to be sustained by the connections?
A. Well –
Q. So there has to be a weakness there which was triggered fairly early in the collapse sequence if your –
A. Well I don't believe it was necessarily triggered by east-west earthquake. I think it could've been triggered by north-south earthquake and this is, I haven't done some calculations to this but if you look at

level 2 and its attachment to the south shear wall you've got to develop quite a high load in the north-south direction to pull that wall over so it remains connected to the building in its weak direction, just as a slab's standing up, you've got to develop quite a high load to pull that wall over so it conforms to the building deformation. Now if you get a very high pulse to the north I have a suspicion that the reinforcing into that wall could not develop that load, and I think, my suspicion also is that the, only the starters would've been effective in trying to develop that load. And the other thing that makes me suspicious of this is that if you have a look at a photograph by Mr Heywood. He shows a photograph of three slabs stacked one on top of the other, close to the south shear wall, and they're about that far from the south shear wall. Now if you do your simple –

JUSTICE COOPER:

Q. So what's that, it's about?

A. Eight hundred or.

Q. Eight hundred?

A. Yeah. If you do your sums looking at the slab length on a diagonal, it looks to me as though those slabs rotated about grid line 2, for one floor. I haven't quite worked out why some of them only rotated one floor not two or three, but that's another issue. I think what – and there's a further thing that makes me worried about the south wall is that there seems to be very little evidence that it actually resisted any significant loads from a pulse to the east. It only appears to have resisted a significant pulse to the west. So there's something fishy going on with that south wall, and I think, I think that south wall had one or two of those slabs at level 2 or 3 torn away from it, from the first quite high pulse to the north.

QUESTIONS FROM COMMISIONER CARTER CONTINUES:

Q. Could that have been partly involved with the torsion loads, not the?

A. No nothing to do with the torsion.

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Q. Nothing to do with the torsion, thank you.

QUESTIONS FROM COMMISSIONER FENWICK:

Q. I would like to come back and just ask you one thing about the beam column joints on the line.

A. Yeah.

Q. Can we have BUI.MAD249.0284.20 please. This brings up a plan of the beam column joints on line F where – there's, a transverse beam coming in?

A. Yeah.

Q. And it's the still, it is the diagram just above the figure 12, near the middle of the page –

A. Yes.

Q. And the one above it?

A. Yes.

Q. You can see it there?

A. Yes.

Q. Okay now if you look at the one above, you can see the longitudinal bars?

A. Yes.

Q. There are three of them, I think they are 24 millimetre, I am not sure about that I think they are 24, either 24s or 28s but it doesn't matter.

A. Yes, yes.

Q. A very lightly reinforced section, that of course is filled with concrete with the slab over the top and then you look directly below it, you see the section through it and do you see the bottom steel comes along, we have lost it, go down please, directly below where you are there, below eight down to 12, that's it?

A. Yes.

Q. You see the bottom steel is bent up so it doesn't actually, there's no continuous bottom steel through that beam column joint?

A. No.

Q. Be quite interesting on when it was cast you have had, bound to have a crack through there due to heat of hydration reduction and (inaudible 12:52:05) shrinkage and all these sorts of things. I just wondered if you felt that drawing, that detail was actually code compliant. Can you have a beam column joint where you don't have steel going through? I am sorry that is an impossible question but I –
A. It is compliant with the ACI, or it is compliant with what ACI was at that time, where, in those situations you allowed to just terminate your bottom bars, 50, um, six inches into the joint and –
Q. With no continuity?
A. I think it applied to beams, it certainly did with slabs.
Q. Well certainly for beams you had to anchor them into something but here we are talking about a beam column joint where presumably one's transferring moment?
A. It doesn't comply with the anchorage requirements for anchoring bars into a – anchoring bottom bars into beam column joint, no.
Q. Could that detail transfer bending moment?
A. Yes.
Q. In one direction only?
A. Ah, I think it probably, partially in both directions quite frankly because although they are non-compliant as far as anchorage is concerned they still actually develop quite a significant load.
Q. You don't think there'd be a crack through there?
A. Oh, there is likely to be a crack through there.
Q. Between the two bars?
A. Yes.
Q. So you have just got top steel? Look thank you, you have given me your answer –
A. I think you are right but I still think that it is capable of actually transferring quite a significant amount of tension which increases the shear on that joint.
Q. How would you rate that detail A or A plus or D for fail?

A. I would have liked to have seen two hooks crossed and taken as close to the other cage as possible. There are requirements actually in the standards.

Q. You would hope that Professor Paulay did not look over your shoulder if you had drawn that detail?

A. I know what he would do.

QUESTIONS FROM JUSTICE COOPER:

Q. Well would you have drawn a detail like that?

A. No.

Q. Now in 1986, you were practising in a firm of engineers?

A. Yes.

Q. What firm was that, remind me?

A. At that time it was Morrison Cooper Limited in Wellington. We had a small Auckland office and we had a Christchurch office too.

Q. In this case we have had evidence that Mr Harding designed this building and it went off to the Council for checking without any checking within Dr Reay's firm. Mr Harding I think was not a principal or a director of that firm. Would that have been usual or unusual in your experience for plans of a six storey building to be sent out of the designer's firm without any checking within that firm?

A. It would be unusual. The calculations wouldn't necessarily have been checked but some senior person would have gone through the drawings to see whether they looked right and were coherent. It was before the days of, of course a formal QA but it would be unusual for someone not to have a, cast a critical eye over at least the drawings.

Q. And now when unusual, my question, and I assume your answer has an assumption about what standard practice was, underlining it within engineering firms in the mid 1980s, is that basis on which you are answering the question?

A. Well we were, on New Zealand Standards quite a significant size firm so we might have done things somewhat differently than smaller firms.

Q. Do you know – do you have a feel for that?

A. Not really, not definitively no.
Q. Well supposing this notional person had had a look at these plans to see if they felt right, what would the conclusion have been do you think?
A. Well I still believe that in the Christchurch environment which was moderate seismicity you'd look at a building like this and say it is a shear wall structure and the corollary to that might be well, the limits in the code are set actually for probably frame structures which were a lot more flexible. It should be okay to be non-seismic but I don't know whether that was the common attitude in Christchurch but it is a possibility that the attitude was, it's a shear wall structure it will be okay as far as ductility is concerned or lack of same.
Q. Well supposing it had been you checking this building. Would you have wanted to actually find out a bit more about the calculations of how the two walls that were being relied on for lateral resistance to ascertain whether they would in fact work as intended?
A. I think I would have asked a few questions, yes, because that was one of my responsibilities at that time.
Q. Within your own firm?
A. Well I was effectively the most senior structural engineer apart from the section leader who also happened to be on various of the standards committees so we were pretty aware of what was going on.

1300

Q. I think when Mr Elliott asked you a question that you didn't think was appropriate to answer about, which was couched in terms of Dr Reay's conduct, and you shrank from answering that question but you said it would depend on how much confidence that he had had in Mr Harding.
A. Yes.
Q. Now Mr Harding hadn't designed a building like this before. Would, should that have been a relevant consideration in, something taken into account in deciding whether or not this design should be checked by somebody else?
A. Well I think that's a fair comment, yes.

Q. Can I infer from your answers that, well no let me ask you. If it had been you would you have left it to the unsupervised decision-making of somebody who hadn't designed a building such as this before?
A. No.

JUSTICE COOPER:

Now Mr Palmer any questions arising from any of the Commissioners' questions?

QUESTIONS ARISING: MR PALMER

Q. Just dealing with these, these last questions about, essentially about checking Mr Harding's work. You were never in a small firm were you –
A. No.
Q. – or a small/medium sized firm. You've always been in a large firm.
A. Well in Wellington at that time we were, mid-'80s, we were about 100 strong, we would have had 15 or 20 structural engineers.
Q. And Wellington in terms of seismicity is a higher level of design –
A. Yes.
Q. – code for earthquakes isn't it?
A. Very much so.
Q. Christchurch you said was moderate.
A. Moderate in terms of relative to the New Zealand environment.
Q. When you answered the Commission's questions before did you have in mind that, about supervision of Mr Harding. Did you have in mind that he was an engineer with 13 years' post-graduate experience?
A. I didn't realise it was that much.
Q. Did you have in mind that he was an engineer with 10 years' post registration experience?
A. Yes my understanding was though that he hadn't done much structural work. Most of it had been more general civil.
Q. Well if you were to be informed that he worked for a solid stables at the beginning, Hardie & Anderson, with good training and had designed

other four-storey buildings, would that make a difference to your answer?

A. It'd be, wouldn't be so black and white no.

Q. And particularly when he's working in a firm with another engineer who perhaps didn't have that much more experience than him if any, would you expect that engineer to be checking his work?

A. Well I've always lived, worked in a collegial environment which gives me a somewhat different attitude.

Q. Yes but what would you expect in those circumstances I put?

A. I'd have been surprised that someone didn't, that the most senior engineer in a small practice didn't take some interest in what was going on.

Q. What about if that senior engineer didn't have a significant amount more experience in designing these sorts of buildings than the engineer in question?

A. I'd still think he'd have taken an interest.

Q. If you were that senior engineer would you take some comfort from the fact that all of the plans submitted to the Council would be reviewed by qualified engineers?

A. Not a lot, no.

Q. Why not?

A. In the '80s we did some pretty interesting things that were well researched but they were alternative solutions I think you'd call them and they were very innovative. We wouldn't have expected a Council engineer to know where we were coming from even.

Q. I just want to ask you a further question about, that answer of innovation. You're not suggesting this building was, was in the same category of innovative buildings that you were designing in Wellington are you?

A. No.

Q. This was –

A. Every building has a little bit of innovation, you know, we take little steps. Sometimes we take bigger steps all at once.

Q. But this was a relatively standard building wasn't it?

A. It was, it had some interesting pre-cast details which were pretty innovative at that time.

Q. Wouldn't that give the Council engineers inspecting the building cause to take more care than otherwise?

A. Yes, but if someone's being innovative the assumption I guess is made that that person being innovative has a fair knowledge which isn't just of standard structures.

Q. But as you said this wasn't particularly innovative was it?

A. It was, some of the pre-casting was reasonably innovative.

Q. So with those answers how, how do you without receiving a design certificate satisfy yourself as a Council that the building was compliant with the bylaws?

A. Well just because it's innovative doesn't mean to say it doesn't satisfy the bylaw. I think the design certificate is probably quite important from that point of view if it's outside your normal experience as a reviewing engineer.

Q. And if one is never given or sought, doesn't that fall upon the reviewing engineers to take greater interest?

A. I guess so.

Q. I've just got one further question to ask in relation to something that Commissioner Carter asked you about the south shear and the slab connections. I think my understanding of your evidence is that you consider that the collapse may have initiated in terms of the reinforcing connections to the slabs probably around level 2, is that correct?

A. Yes I wouldn't, well put it this way I wouldn't rule it out. It seems as though it's a very legitimate place to start such an analysis.

Q. Now have you been at the hearing sufficiently long enough to hear the evidence about the phenomenon referred to as strain hardening and I realise that there is some issue of terminology but it's the only term that I would use because it's the only one I'm aware of okay?

A. I've heard that evidence being given, yeah.

Q. If the reinforcing at the connections between the slabs and the south shear had been subject to prior weakening through this phenomenon, would that enhance the probability of your possible collapse scenario?

A. Well it wouldn't, it wouldn't be strain hardening. What I'd call it would be brittle failure.

Q. And calling it –

A. Quite possibly in mesh.

Q. So calling it brittle failure would you say that that would have, that would enhance the possibility of your collapse scenario?

A. It's a possibility that it could have contributed, yes.

HEARING ADJOURNS: 1.09 PM

HEARING RESUMES: 2.16 PM

JUSTICE COOPER:

Mr Mills was there anything further you wished to ask?

MR MILLS:

No nothing from me Sir.

CROSS-EXAMINATION: MR ELLIOTT – NIL

RE-EXAMINATION: MR REID

Q. Firstly, Dr O'Leary I just wanted to see if we can crystallise whether there's anything in your evidence that's changed as a result of the cross-examination that we've had today. So firstly in relation to the questions that were put to you by Mr Elliott. Can you just advise the Commission whether any of your views have been changed as a result of that cross-examination?

A. No.

Q. What about the questions that were put to you by Mr Mills or by the Commission?

A. Well, I reviewed whether this building may be, what are the words, "Have a high degree of eccentricity," and maybe it could be interpreted as that. I think it's interesting to note that the actual degree of eccentricity does not affect how you treat torsional – horizontally torsionally induced moments, so I think it's fair to say that probably more closely to a high degree of eccentricity than moderate.

Q. And do you, in coming to that view, do you make a distinction between symmetry and eccentricity?

A. Yes, I rather make the distinction between regular and eccentricity. I'm still strongly of the view that the structure is reasonably regular, but tending towards a high degree of eccentricity.

Q. Now, I ask you, following on some questions from Mr Mills about the issue that arose in questions from him regarding the output of the ETABS analysis and the location of the columns in the building. Does the fact that the output from the ETABS analysis was located in the centre of the building affect your analysis as it applies to the internal columns?

A. No it doesn't. The internal columns are relatively close to the centre of rigidity/centre of mass and I'm not saying they're close, but they're relatively close so that there's relatively little rotational – horizontal rotational displacement associated with the internal columns.

Q. And is that the case for the columns on line F?

A. No line F, the torsional rotation could increase the inter-storey drifts of those columns by a very significant amount. It could be double it or something like that. I haven't done the sums but, you know, you just need to look at the plan and the torsional resisting elements and you can see which columns are going to be most affected by torsion, line F the one.

Q. And just one final thing. My friend, Mr Palmer, asked you some questions about the way that the Council may have viewed the plans when they came into the Council's offices. I just wanted to refer you to

Mr Tapper's letter of the 27th of August and that is BUI.MAD249.0141.12?

JUSTICE COOPER:

.14?

RE-EXAMINATION CONTINUES: MR REID

Q. .14.

WITNESS REFERRED TO SLIDE – LETTER FROM MR TAPPER

A. Yes.

Q. Now Mr Tapper in the first part of this letter refers to computa – the design drawings needing to be signed?

A. Yes I see that.

Q. And attaches some significance to that?

A. Yes.

Q. Now if I could just be, refer you to BUI.MAD249.0284.1?

WITNESS REFERRED TO SLIDE – SIGNED PLANS

Q. These are the signed plans?

A. Yes.

Q. I'll just wait for them to come up.

A. I've got a copy of them here actually.

Q. Yes?

A. Yes.

Q. We'll just have a look perhaps at .3, one of the first pages that's signed of the drawings themselves?

A. Yes.

Q. You can see that they're signed?

A. They are.

Q. What significance would you attach to the fact that the drawings are signed?

A. Well I would see a signed set of drawings as being considered by the practice as these are the drawings as to, I hasten not, I'll use it again, these are the drawings to the best of our knowledge comply with the requirements of the standards and the bylaws presumably.

QUESTIONS FROM COMMISSIONER FENWICK:

Q. If I could extend that questioning just a little bit?
A. Yes.
Q. You've said that the increase in the torsional deformation has increased the deformation on line F. Would it also have increased the deformation on line 1 and on line A?
A. It certainly would in line 1 and line A.
Q. Because line A, though, it's got flexible columns on, it's got an odd situation in the first two or three levels hasn't it?
A. Yes.
Q. Because it's got the brick infill?
A. Yes.
Q. And then you've got the brick with the gaps in but the rods going through –
A. Yes.
Q. – and therefore the columns, the beams on that line, though they are relatively flexible really are held against rotation aren't they?
A. Yes, they're relatively flexible which means that it builds up relatively low moments in the columns.
Q. So the columns actually have to, have to actually deform quite a lot more don't they, because those, though those beams are flexible it's supported at intervals which stops it deflecting, when actually the actions in the columns become quite interesting in a way don't they? I know they're more slender columns but they're, they're taking a bigger proportion of the drift aren't they, because of the –
A. But if you have a look at the, if you have a look at the joint. Because the beams are flexible, it is, you need more drift to build up the same moment in the column.

Q. But the beams can't deform can they because they're restrained by the blockwork and the bars which go from the, bars which are greased and they're going in at 600 or 800 centimetres?
A. On hogging you're unlikely to be able to deform them much but in sagging you actually could a significant amount.
Q. Well the bottom of the bar –
A. They're pretty flexible bends.
Q. – the bottom of the bar's in concrete, then you've got a gap –
A. Yes.
Q. – grease bar, it's rather hard to see it going down or up.
A. Well that was a, that's the object of a grease bar is to allow the beam to move vertically –
Q. So it's just in compression?
A. – independently of the blockwork.
Q. Okay, yes, in spite of the lateral deformation, it's hard to believe.
A. Well actually there's a calculation in the, that shows that the grease bar was designed to stop out of plane bending.
Q. Yes?
A. So it seems pretty obvious to me that there was a really good understanding of what that grease bar was meant to be doing.

1425

Q. Okay, thank you.

QUESTIONS ARISING: MESSRS PALMER, MILLS AND REID - NIL

WITNESS EXCUSED

MR REID:

Sir just one point for clarification.

JUSTICE COOPER TO MR REID:

Q. Yes.

A. Dr O'Leary's happy to come back at any point and indeed I think that's been discussed in relation to the drag bar part of the enquiry. He may need to come back in relation to that. So he can answer any questions at that point that may arise.
Q. Right well I'm not sure that there's anything really outstanding.
A. No.
Q. The eccentricity issue's really rather been covered by your re-examination hasn't it?
A. Yes.

JUSTICE COOPER ADDRESSES COMMISSION – CONFIRMED DR O'LEARY NO LONGER REQUIRED

JUSTICE COOPER:
Mr Mills.

MR MILLS:
The next witness Sir is again, Dr Jacobs coming back to give his reply evidence.

MR MILLS CALLS

MURRAY LIONEL JACOBS (RE-SWORN)

Q. I'll just get a few minor formalities again Dr Jacobs just for the record. Your full name is Murray Lionel Jacobs?
A. Yes it is.
Q. And you are here to present your second brief which is a reply brief dated 1 June 2012. Can you just confirm that you have that in front of you? That it does bear that date?
A. It's signed at the back.
Q. Yes it is, yes.
A. Yes I have that.
Q. All right and thank you and this evidence is in reply to the evidence that Dr O'Leary has given.
A. That's correct.
Q. All right. With that said I invite you to start reading at your second paragraph and Commissioners obviously this will be easier to follow if you have open at the same time Dr O'Leary's evidence to which it's replying.
A. This evidence is in reply to Dr O'Leary's second statement. It's not the amended statement.
Q. No I appreciate that. When you say it's not the amended statement you mean it was done before those relatively small amendments were made to it?
A. That's correct.
Q. Yes if there's anything that matters, have you seen the amended statement?
A. Yes I have seen the (inaudible 14:28:26).
Q. Well if there's anything that that's changed in terms of what you want to say then you just indicate it and we'll let you say it.
A. Thank you very much.
Q. Right so if you just start reading at your paragraph 2 please.
A. I consider that NZS 4203:1984 clause 3.2 Ductility is a direct instruction that all the elements that resist seismic forces or movements **or that in**

the case of failure are a risk to life shall be designed to possess ductility. The words from the commentary mentioned in Dr O'Leary's paragraph 22 are most relevant.

*"The general requirement for ductility must at present be qualitative rather than quantitative **except for buildings design to dissipate seismic energy by ductile flexural yielding"**.*

The CTV building was designed to dissipate energy by ductile yielding of both the north shear wall and the south coupled shear wall.

Further, 1.1.3 Definition Section of NZS 4203:1984 notes:

"SECONDARY ELEMENTS means elements such as partition walls, panels or veneers not necessary for the survival of the building as a whole but subject to stresses due to loadings applied directly to them or to stresses induced by the deformations of the primary elements.

*PRIMARY ELEMENTS means elements forming part of the basic load resisting structure, such as beams, columns, diaphragms or shear walls **necessary for the building's survival when subjected to the specific loading** [emphasis added]".*

These definitions make it clear to me that under NZS 4203:1984 the internal and external columns and beams, as well as the shear walls, are primary elements of the CTV.

Clause 3.2.3 does refer to the appropriate material standard for ductile detailing as Dr O'Leary points out. The commentary C3.2.2 however does give guidance on what is required.

The requirement of clause 3.2.2 is in effect a practical approximation for the assessment of section curvature ductility demand. A more rigorous analytical approach which is applicable only to reasonably rectangular frames –

Q. Regular?

A. Sorry, you're quite right – regular frames without sudden changes in storey stiffness is the method using the following approximate criteria.

"The building as a whole should be capable of deflecting laterally through at least eight load reversals so that the total horizontal deflection at the top of the main portion of the building under the

loadings of equations 4 and 5 and calculated on the assumption of appropriate plastic hinges is at least four times that of first yield –"

Q. Can I just ask you, is it appropriate or appropriated?

A. Appropriated it is.

Q. Okay.

A. "– without the horizontal load carrying capacity of the building being reduced by more than 20%".

Where U = 1.0D+1.3Lr (which is life load reserve reduced) + E for earthquake, U = 0.9 D+E.

This requirement should have raised a signal that the internal columns were unlikely to remain elastic during this design demand. The deflection of the shear walls at four times the calculated elastic deflection would likely generate moments in the top and bottom of the load bearing columns that would have required ductile detailing for integrity. Also there would be a likely need to provide substantial shear reinforcement in the columns to prevent a sudden catastrophic shear failure. The assumptions that there is negligible shear in the columns and hence that the concrete could carry the stress would be hard to sustain.

I agree with Dr 'Leary that the shear wall on line (5), the north wall, did not show signs of plastic yielding. The shear wall on line (1), the coupled shear wall, may well have yielded at the base but was brought down by the floors collapsing on the inside of that wall.

The building collapsed before the main north shear wall was required to carry its design load. There are many theories as to what happened– collapsed first, but what is very evident is that the columns collapsed causing the catastrophic failure a short time after the earthquake.

I consider that NZS 4203:1984 clause 3.4.7.1 (c) applies to this building. The building has a high degree of eccentricity in the east-west direction and the floor plan is irregular hence clause 3.4.7.1 (c) applies.

There are also re-entrant angles between the major seismic resisting element, the north shear wall, line (5), and the rectangular floor plate which made it difficult to structurally connect the shear wall to the main part of the building. The main resisting element of the building is outside the floor plate. Only a small length of slab approximately 3.75 metres long reducing to 2.75 metres at the wall interface is available to transfer the horizontal shear to the main seismic resisting element in the east-west direction. The building appears regular in the vertical direction between floors.

The two shear walls that resist the seismic shear in the east-west direction are very unequal in stiffness and strength. The computer analysis carried out by Compusoft found that the north shear wall on line (5) had

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much greater stiffness that the south coupled shear wall on line 1 and I think we've, I found it was about 25 to 1.

Even though NZS4203 1984 clause 3.4.7.1 (b) recommends rather than states as mandatory that a three dimensional analysis be used for such structures, a design engineer should carry out a three dimensional analysis which took into account eccentricity for a structure such as this. Alan Reay's firm ARCE undertook a three dimensional analysis although I have not seen the results.

The north shear wall on line 5 is in the shape of a C section 11.65 metres deep with the centre of stiffness outside the building. The south shear wall on line 1 consists of two walls, two metres deep joined at each floor with coupling beams. Dr O'Leary alludes to the problem of two unequal walls in the same directions when he states in paragraph 32,

"...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."

This problem was well known at the time of the design of the CTV building. A paper published in the bulletin of New Zealand National Society for Earthquake Engineering Volume 13, number 2, June 1980 by T Paulay and R L Williams addresses this. Professor T Paulay, Professor of Civil Engineering, University of Canterbury is well known and internationally respected expert on seismic performance of buildings and concrete shear walls. The paper states on page 118 under the heading, "Torsion."

"...as in all structures in seismic areas, symmetry in structural layout should be aimed at. Deliberate eccentricities should be avoided if possible because uneven excitations may aggravate eccentricity and this in turn will lead to excessive ductility demand…"

Q. Thank you, I think you have written, "may lead," is that what you meant to say –

A. Yes, "may," I am sorry, I do apologise.

"...in lateral load resisting elements situated far away from the centre of rotation."

An example is then given of a shear wall arranged very similar to the CTV building where a shear wall of unequal stiffness is placed at each side of the building. Quoting.

"An example of the unintended inelastic response of two ductile shear walls is illustrated in figure 13a. Because of the centre of mass CG is approximately at the centre of the plan, approximately one half of the earthquake induced load E will have to be resisted by each of the end walls at A and B. It may be difficult to prevent wall A from having a lateral load carrying capacity considerably in excess of that on wall B. Hence energy dissipation due to inelastic deformation may well be restricted to wall B only which, as a result of this, could be subject to a displacement delta much larger than expected. Irrespective of the relative stiffness or strength of the two shear walls, structures in which only two principal planes of lateral resistance exist parallel to either major axes, are likely to be torsionally unstable during large inelastic seismic excitations".

The diagram shows the two different sorts of structures there, one with the small wall and a large wall and the other with more of a balanced sedt of walls, situated around the edge of the building so they're good at taking torsion.

The description of wall A and B in the paper is very similar to the situation of the north and south shear walls on the CTV building. Wall line 1 the south wall is as for wall B in figure 13a and wall line 5 the north wall is as for wall A in figure 13a, except wall line 5 in the CTV building is outside the main building floor plate, which is worse.

The smaller shear wall, south wall, could have been subjected to a displacement delta much larger than expected, as it outlined in this paper. This would mean that the columns in lines 1 and 2 would have also been subject to greater displacements than expected. These columns were brittle. They have not been detailed for ductility. They could only accept a limited amount of deflection before failure. The code has warned many times about the limitations of assumptions, analysis and the uncertainty of the behaviour of unsymmetrical structures. These are the reasons that the warnings and requirements have been outlined. This warning was given in his paper by Professor of Engineering in Christchurch University many years before the CTV building was designed. It was available in the widely read bulletin of the New Zealand National Society for Earthquake Engineering and is reflected in the many instructions in the code about eccentric buildings.

There is a plan there showing the arrangement of shear walls which we have seen quite a few times before.

I do not agree with the statement of Dr O'Leary that the east-west wall on line 5 and the wall on line 1 provide efficient torsional resistance, for the reasons outlined in the paragraphs 17 to 20 above and in the paper by Professor Paulay.

The smaller wall on line 3 will cause excessive deflection and aggravate the eccentricity. The return wall lines C and DE in the north shear core are slender and relatively close together. They were not effective in resisting torsional loads from earthquake forces in the east-west direction. The walls at the end of C shaped north shear wall on line 5 were also not connected to the slab with sufficient capacity to take the tensile forces from torsion induced seismic loadings in the east-west direction. This especially applies to the return wall line DE. I am aware that retrofitted drag bars were installed at a later date. However, the tensile strength of the slab with only 664 mesh would have been suspect at a position in the slab outside the saddle bar line and that is the line on line 4.

Retrofitting effective drag bars under the slab would have been a difficult task to achieve practically once the building had been completed. The drag bars would have needed to connect to the slab back to line 3 to be effective in my opinion. I could elaborate on that?

Q. Well if you would like to?

A. So I found in my experience a very difficult job to strengthen a building just a concrete building once it's been cast. The dilemma of engineering is that if you want to strengthen a building before the concrete is cast is a relatively easy job. You just put more reinforcing on the drawings then they get translated to the building. Once the concrete is cast then it is very hard to strengthen it and you often actually remove some of the existing strength of the concrete with your repair mechanisms – you may drill through reinforcing bars.

The other thing that I've found that the gluing of reinforcing rods into concrete especially about this time is not always a reliable mechanism. There is nothing on the drawings that I could see that said those bars should have been tested with pullout tests and that is what is usually done I have found. I had experience about the same time of reinforcing a large stone wall for earthquakes and building a wall behind it and we drilled in epoxy resin bars in and one day I went along and pulled one of these bars and it just came out. So one of the problems was that when you drill the hole you get a slurry in there which sets for a while and they glue on to the slurry but it hasn't got any real strength but looks like concrete so it can pull out quite easy and that is the reason I felt that this repair could well have been quite suspect.

Q. Thank you for that.

A. The design for torsion at the time was based on the predicted behaviour of the walls. If a wall was overstressed by torsional effects then it may become torsionally unstable, as described in the paper by Professor

Paulay in 1980 to which I have referred. The return walls on the north core wall line 5 were not likely to have provided significant torsional rigidity and ability to carry the torsional loads. They are slender walls,

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relatively close together and poorly connected to the floor.

Q. Now Dr Jacobs, that's one of the paragraphs which Dr O'Leary changed after you had prepared your reply brief?

A. Yes.

Q. If you've got that in front of you I just invite you to look at it and see whether the changes that have been made are something you want to comment on because the changes are reasonably extensive?

WITNESS REFERRED TO REPLY BRIEF

A. That's right, on paragraph 35 –

Q. Thirty-five –

A. This is where I think in the original one Dr O'Leary said there was only one wall that was not or inferred there was and he has since changed that he agrees that this particular shear wall has a notch at level 1 so it could possibly be classified as a slender wall.

Q. Yes but then you'll see that he goes on and it's the underlined addition?

A. Yeah.

Q. We're about another half dozen lines or so and he was, makes some further comments having made that concession to I think what you had said originally about the slender wall and it's that really which you might just like to take a moment to read and see if you want to say anything in response to that?

A. Yes well I would not agree with him especially the last sentence: "on this space I consider the wall spanning north south should be still, should still be classified as stiff because they are effectively stiff compared to the frames".

Q. They are relatively staff is what he said compared to frames.

A. Oh relatively sorry, I'm not going (inaudible 14:46:34) frames (inaudible 14:46:35). Well I did a drawing of those frames, of the frame in line F against those shear walls with a notch on it in my original statement, that's on page WIT.JACOBS.0001.12.

WITNESS REFERS TO PAGE WIT.JACOBS.0001.12

A. It shows the frames –

Q. Just wait a minute we'll bring that up.

A. They're supposed to be drawn to scale.

Q. Yes.

A. I did the drawings so I can't guarantee that I'm a good draughtsman.

Q. All right there it is on the screen.

A. I didn't consider that that was obvious that that frame was very flexible compared to the shear wall. The frame I think Dr O'Leary said the frame had long spans. Well the spans are I think about 7.5 metres from memory centre line to centre line. That is not a long span for a floor beam, I typically have designed buildings with 11 metre spans, 12 metre spans but 10 or eight metre spans is, is quite a common span for a building like this. The columns are, are skinny or narrow so they do make that frame more flexible but the beams are reasonably stiff and also the floor to floor height is quite small for an office building. It's 3.24 I think it is and it has a beam 550 millimetres deep so you get a resulting 2.7 clear height for the columns. That's relatively short so, I also think that the analysis undertaken and a graph shown I think it was in the Hyland and Smith report showed the shears taken from a push over analysis done on this building, it showed those columns were taking round about 20% of the shear, the total shear load in the building. I thought that was a significant amount of shear of the total shear to be taken. Also that notch even though some of the computer analysis don't show the rotation as occurring in my experience I think I mentioned in my main evidence has been that a lot of rotation will occur where that notch is it sort of forms like a hinge there. That's all I have to say.

Q. All right thank you Dr Jacobs. You were at paragraph 34.

A. Thirty-four.

Q. Or 24 rather. I'm sorry.

A. Twenty-four sorry yeah.

WITNESS CONTINUES READING STATEMENT FROM PARAGRAPH 24

A. In these paragraphs of his evidence Dr O'Leary sets out his reasons for concluding the non ductile detailing of the internal columns in the CTV building complied with the code. I do not agree with him. I understand from the code that the columns should have been designed for ductility. There are many instructions that this should be so. They are mentioned in the Hyland Smith report, the third statement of evidence of Ashley Henry Smith and in my first statement of evidence at paragraphs 27 to 37. The need for ductility in all elements is specified in 4203:1984 at the start of part 3 Earthquake Provisions, to which I have referred previously:

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

I have referred previously to the point that secondary elements are defined in NZS 4203:1984 as: *"elements such as partition walls, panels or veneers not necessarily for the surviving, survival of the building as a whole…"*

NZ3103:1982 is a code that states, sorry that relates to the design of reinforced concrete.

I consider that the north shear wall in the north-south direction was slender. The walls on lines C and CD had a severe notch out of them between levels 1 and 2. The other two walls are narrow all the way up the building. The frame line F would have found to be subject to significant seismic loads had it been included in the analysis. I think I've been through that (inaudible 14:51:13).

Q. Yes you have.

A. NZS 3101:1982 clause 3.5.14 states:

"3.5.14.1, Secondary structural elements.

Secondary elements are those which do not form part of the primary seismic force resisting system or assume not to form such a part and are therefore ***not necessary for the survival of the building as a whole under seismically induced lateral loads*** *but are subjected to* *loads due to acceleration transmitted to them or due to deformations of the structure as a whole".* The emphasis has been added.

I have stated early in my evidence that in my opinion the columns in this building were necessary for the survival of the building as a whole under seismically induced lateral loads.

NZS4203:1984 clause 3.5.14.3 states:

"Group Two elements shall be detailed to allow ductile behaviour and in accordance with assumptions made in the analysis".

A three dimensional modal analysis carried out by Compusof as required by the code 4203:1984 gave a set of deflections. When these deflections were imposed on the columns of the CTV building they indicated the columns did not remain elastic especially in the levels five to six and four to five. The shear stress generated by the moments was also greater than that allowed on the concrete without the required minimum shear reinforcement. The modal analysis was carried out for Hyland Smith report part 3 appendix E, referred to as the ERSA Modelling and mentioned on page 236 of the report.

The column design chart reproduced below from the design calculations of Alan M Reay Consultants at page G38 show the design point as a circle around 0.43 on the vertical axis and 0.028 on the horizontal axis. This position on the design chart indicates a heavily loaded column with a small capacity for moment. If the moment is plotted on this graph for the design moments calculated from the three dimensional analysis referred to above, the design point falls well to the right on the horizontal line between 0.4 and 0.5 ordinates indicating a yielding of a column by plastic action.

31. I'm not sure what Dr O'Leary means by the statement in paragraph 42:

"…the design and detailing of a secondary member should have satisfied the applicable clauses (a) and (b)".

It's not clear whether he means that they *would* have or *should* have satisfied these sub clauses.

Q. Right now you've turned to your second, or to the second of Dr O'Leary's briefs haven't you?

A. I have.

WITNESS REFERS TO SECOND BRIEF OF DR O'LEARY

A. These are a reply to Dr O'Leary's comments on my evidence.

Q. Yes.

A. I reply to Dr O'Leary's specific comments on my evidence as follows. Dr O'Leary states in paragraph 9:

"The question of irregularities, lack of symmetry, {paragraph 15(b) (ii)} needs in my opinion to be put in context

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of the understandings of the time and the understandable desire to have attractive and functional buildings".

In my experience at the time, the negative effects of eccentricity were well known. A professional engineer's role was to work closely with the architects to develop a building that complied with the code and at the same time was acceptable from a planning and aesthetic point of view. I do not believe that design engineers were forced into designing eccentric buildings, irregular in shape, because of client or architectural requirements. All the New Zealand architects I have worked with have been amenable to the requirements of sound seismic design once the requirements are explained to them. I have worked for a range of developers, from smaller developers through to large developers.

I have found the same situation in the requirements of the code in relation to seismic detailing. A client will usually accept that a building has to be designed with ductile detailing to conform to the acceptable code, ie, to resist seismic movements and to avoid the risk to life once the issues have been explained.

Q. Now you read "applicable" as "acceptable", "Conform to the acceptable code," you said, did you mean to say that or do you mean to say applicable code?

A. Applicable code, I'm sorry, I meant to say that. In my experience to aim to minimise the amount of reinforcing steel in the critical structural elements of the building is not the most important design criteria.

Dr O'Leary only selects a small part of the commentary on clause C3.1.1. The full commentary on clause 3.1 symmetry in NZS4203:1984 is very specific about the perils of designing non-symmetrical buildings and is reinforced by the paper of Paulay and Williams, page 118 I have previously referred to:

"For high buildings symmetry is one of the most basic requirements in achieving a structure of predictable performance. Simple geometry is essential for obtaining symmetry in practice. Notwithstanding the availability of modern computers, considerable uncertainty exists in selecting a mathematical model representing the true behaviour of complex arrangements such as combinations of geometrically dissimilar shear walls and frames. Shear walls and frames and unsymmetrical combinations of shear walls and frames.

Geometrically dissimilar resisting elements are unlikely to develop their plastic hinges simultaneously, and ductility demand may also be increased by torsional effects."

I do not believe that you can dismiss this clause. It is very specific and it is based upon the accumulated knowledge that buildings do not perform if they are unsymmetrical. As the commentary C3.1.1 outlines, the analytical tools cannot predict accurately the complex action of a real building. The ductility demand on dissimilar shear walls cannot be predicted and can lead to the overstressing and failure of the walls. This was outlined in 1980 in the paper by Professor Paulay, to which I have referred.

This is covered by my comments on the first statement of Dr O'Leary.

The columns are critical as failure would and did cause a risk to life. They support 90% of the building. Clause 3.2.1 specifically states that, *"All elements that in the case of failure are a risk to life shall be designed to possess ductility."*

I was referring to the placement of the spirals as part of the construction sequence as they had to stop below the bottom steel. The anchorage of the spiral, usually by welding to make them continuous, or by bending into the core of the column would be difficult. The bond length of a plain six round bar is around 48 d or 280 millimetres. This means that length of column is without effective spiral reinforcement.

Practice Advisory 8, Department of Building and Housing: Don't be

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undone - Anchor your Spiral, points out:

"inadequate anchoring or splicing can cause the spiral reinforcing to unravel under seismic conditions resulting in
loss of confinement to principal reinforcement,
premature and brittle failure,
failure of the whole structure".

JUSTICE COOPER:

Q. What's the date of that practice note?
A. That is about 2006.
Q. Thank you.

EXAMINATION CONTINUES: MR MILLS

Q. You're at paragraph 41.
A. My point in quoting this clause was that a designer should be cautious about aiming to look for ways of minimising the rules of this code as it is a minimum standard not a maximum.
The north shear core walls were either not at the base from level 1 to just under level 2 or they are narrower than the wall mentioned on line D. I do not understand what Dr O'Leary means by the statement:
"It would be inappropriate in my view to class a group as slender on the basis that only one of them could have had that characteristic". In fact two have that characteristic (lines D and E) and the other two are slender all the way up the building.
Q. Now that's the paragraph I took –

A. That's right.

Q. – you to prematurely and you'll see that he's taken out that reference that you have quoted from him. He's now deleted that from -

A. Yes I agree, I didn't realise that till I was half way through reading it.

Q. Yes.

A. Although I do say there that they have an aspect ratio of over five to one.

Q. Yes.

A. Which would make me consider them to be slender walls.

Q. Yes.

A. The critically important columns do not meet the requirements of elements that can be designed without ductile detailing as they are elements that a risk to life, clause 3.2.1 NZS 4203 1984, to which I have previously referred.

They talk about the mesh. I think there is a misconnect here. I stated that the 664 mesh did not meet the code requirements for shrinkage reinforcement according to my calculations. The calculation for topping steel required for a diaphragm action is another separate design requirement. My point was that this reinforcement did not comply with the very minimum required for temperature – and it should be "and movement."

Q. Temperature and movement?

A. No, no it's temperature movement I'm sorry.

Q. Temperature movement.

A. The requirements for diaphragm action, if they had of been provided for, would have ensured that the return walls on the north shear core were connected to the rest of the building by adequate reinforcing in the slab. Special reinforcing drag bars could have been provided within the slab. However, as designed the 664 mesh was the only reinforcing in the slab to resist the tension forces from the return walls of the north shear wall at the position of the slab where the short saddle bars finished. These walls were the only seismic resisting elements in the north-south direction. D12 saddle bars were placed over the beam lines but they were relatively short.

The 664 mesh was draped down to form support for the slab in case of a fire. I had not come across this method of providing fire steel before as it was very difficult in practice to accurately place mesh reinforcing at different levels, sorry depth in a slab. Usually in my experience extra bottom steel reinforcing bars are placed in the bottom of the metal deck profile to support the slab in case the exposed metal deck strength is affected by the heat of the fire. Draping of the mesh also means that a large area of the slab in the centre span has no reinforcing in the top portion of the slab.

The columns were required to be designed for ductility as I have explained in previous paragraphs as they were a risk to life. They were also subjected to reverse moments top and bottom of the columns due to the movement of the building during the earthquake motion. The calculated moments generated in the interior columns as a result of the 3D analysis referred to in my paragraph 30 of my evidence were above the yield strength of the columns. The resultant shears were also well above that followed by NZS 3101:1982 clause 7.3.4.3.

Q. Now you read the word “allowed” as “followed” but I imagine that was an error?

A. Yes it was an error. To assume that columns in a six storey building would not have significant deflection induced moments in shears and therefore not to reinforce them with a minimum reinforcement is difficult to understand. In my 37 years of structural engineering I have never seen a column with such little lateral reinforcing in New Zealand.
And 47. I think these objections to my conclusions by Dr O’Leary have been covered already.

Q. Thank you Dr Jacobs. Now I just have one other matter I wanted you to deal with before I sit down. [And if I can just, this is a reference from the transcript from the last time Dr Jacobs was here so if I could just have this brought up. It’s TRANS 20120809 and it’s page 25 of that transcript that I'm wanting]. Now just beginning, sorry – 27, try 27. Yes that looks more promising. Now if you look, you'll see the numbers of the lines down the left-hand side, numbered in fives, if you go to what is line 12, the answer, "Yes," if we look at that clause there, what I want you to do if you wouldn't mind Dr Jacobs is just read that page down from that line 12 and then I'm afraid I'm going to have to ask you to read over the next page to where Justice Cooper addresses Mr Reid and what, this involved a series of questions by my friend Mr Reid about your views on ductility and you gave an answer to that at the end which as I read at any rate is different from what you gave in your evidence-in-chief. I just want to be sure that the answer you've given is the one you want to give. If there's any change in that then I'm inviting you to state it but if there isn't that is just fine.

A. Yes if we look at that clause there, C3.5.14.1 the definition of a secondary element is more particular than that in NZS 4203, 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 earthquake, to lateral loads.

JUSTICE COOPER:

Mr Mills were you intending the witness to read this aloud?

MR MILLS:

No I wasn't, no, no.

EXAMINATION CONTINUES: MR MILLS

A. I'm sorry I thought you were.

Q. Sorry I thought - no, no I just want you to read it to yourself.

A. Sorry. I thought you (inaudible 15:08:47).

Q. Just to get you to refresh yourself on this line of cross-examination and you'll see where you get to, where at the end you say, "Yes that is correct," and I just want to be sure that the Royal Commission has got your considered view on this correctly.

A. (witness reads). Yes I've read down to 20.

Q. All right and so does your final answer there, "Yes that is correct," does that on reflection correctly state your view on that?

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A. My view on that – yes it does because I believe that what he was asking me there was that, if you were to design a building and you looked at the deflection according to this code and it was within the V delta then you would be correct to accept that that didn't need ductile detailing but I did not infer from that that the columns in the CTV building were, conformed to that.

Q. I see.

A. It was just looking at that specific clause which says if you have V times delta and the column had not gone into its plastic design action then that clause says that you are able to do that but to me it contravenes all the other clauses that I think are very important and also when I start to think of, can I think of a hypothetical building that would conform to that, I am hard pressed to really think about a hypothetical building.

CROSS-EXAMINATION: MR PALMER

Q. Dr Jacobs, could I just take you back to paragraph 11 of your evidence?

A. Is that the evidence in reply?

Q. Just the one that you have read?

A. Yes, paragraph 11, yes?

Q. It is a short paragraph. When you came to the view that you express here about the building having a high degree of eccentricity in the east-west direction, did you do calculations to make that assessment?

A. I did calculations yes, I based it on a number of things. First, I did calculate – I don't know whether it was first or not, on the stiffness of the two walls, one was 11.65 metres deep and the other was, I assume the other one was a rigid wall of about five metres. I neglected the effect because it is easy to do that and that would be conservative.

Q. Are they the calculations that you might be expected to do under the commentary to clause 3.4.7.1?

A. I expect so, it is certainly not the calculations I would do as an engineer looking at that building.

Q. When you did those calculations did you take into account soil stiffness as determined by Ian McCahon?

A. No I didn't take into account soil stiffness because also in NZS4203 I think it says you neglect deflections due to soil, due to ground.

Q. Regardless of what it says you did, you certainly –

A. I certainly didn't take –

Q. You didn't –

A. – I didn't try, I was doing a hand calculation so it would be quite difficult to (inaudible 15:13:08) into the soil –

Q. Now in your earlier brief of evidence you talk about, and this is, I will read it to you, it is a very short passage it is from paragraph 12 of your original brief. It's a sentence in the middle of the paragraph, have you got your earlier brief there?

A. The original one, yes.

Q. In that case –

A. I will just look that up.

Q. Have a look at paragraph 12?

A. Yes.

Q. You are picking up here this issue of symmetry?

A. Yes.

Q. You mentioned line 2 that this, the resisting elements in the structure are asymmetrical in the east-west direction and then you say, "In the north-south direction the eccentricity is less."?

A. Yes.

Q. You don't mention eccentricity north-south in your reply evidence that you gave today. Is my assumption correct that in the north-south direction the eccentricity is not high but moderate?

A. Just trying to think of – yes, if the earthquake was coming in the north-south direction I believe it would be moderate (inaudible 15:14:23), yes. That is if – mind you, in effect in this building I don't believe it is moderate because I don't think it is connected to those shear walls but had the drag bars been put in and those walls been working effectively then I believe that would have been so.

Q. So making the assumption that everything is connected as it should have been –

A. Yeah if the diaphragm had conformed with the code then I think it would have been reasonably symmetrical.

Q. However, in the real world there are times when matters of detail do need correction, aren't there?

A. Yes I would think so yes.

Q. And in the case of the CTV building of course the Holmes Consulting Group identified in 1990, 1991 the need for better connection to the north shear wall, didn't they?

A. Yes I believe they did, yes.

Q. And the solution that was adopted, what, really was one which had to be used when a problem of that type was found, wasn't it?

A. I am not quite sure what you mean by, "had to be used," the exact solution had to be used or a solution post, as I was pointing out before, once the concrete is set you are limited in what you can do about it. I don't think that actual solution they used was the only one that had to be used, I think there were other solutions but that was the solution they chose to use.

Q. And would you say that in the circumstances that was an acceptable solution?

A. From my point of view, I don't think so, no. I don't think that did connect that, those return walls adequately to that building. I think the enormity of what you are trying to do then, to connect that slab through that, you have a 300 millimetre wide wall butting up against the slab like that and you are trying to get a very good connection between that, that is a fairly difficult thing to do. It would have been a lot easier, say for instance if you had put, before the concrete was cast perhaps you could have put four 32 diameter bars, cast them in there, that would, that is a typical solution you would use.

Q. But as we know the concrete had been cast –

A. Once the concrete set you have quite a lot of problems. I have said in my everything I thought those drag bars should have gone right through to line 3 in other words, so they spread out within the slab especially seeing we only had 664 mesh in the bottom, drooped in the bottom in a mid span. So I thought they weren't quite adequate.

Q. But nevertheless a drag bar style of solution perhaps designed as you've indicated you would have done it would be an acceptable solution, is that correct?

A. If it could have satisfied the requirements yes although I noticed that the drag bars solution put forward in that Holmes report was not used in practice. A lesser degree of drag bars were put in, in the final solution.

Q. I think – did you, wasn't that done after calculations were done regarding the –

A. I don't know what the reason was for. I think a different engineer did those but they were a lesser, I am not sure why they were lesser, they cost quite a lot less –

Q. Did you analyse the Holmes calculations?

A. I didn't analyse the calculations but I did look at it and I looked at, it rang a few alarm bells for me. One was this problem I have with the drilling in and the epoxy resin in, especially the techniques were not very developed there. The other thing that these bars were quite close centres in the Holmes report. I think from memory they were 100 millimetre centres and you do get a problem with putting bars in too close to each other because they start to influence each other and you could, you don't get the full strength of a bar in isolation.

Q. But in the end, problems like this do happen don't they?

A. Oh, they do happen yes.

Q. And presumably they will have happened in your practice at certain points?

A. I have seen them happen in my experience too I am not admitting to them in my practice of course but I have seen them when I worked for other people. I used to be a bridge engineer and I saw a lot of problems in bridges.

Q. Are you aware that drag bars were put into the building design by your firm the Sun Alliance building after the design plans were finished?
A. No I am not aware of that, no.
Q. Would seeing a plan of the building help you in that respect?
A. Yeah, yeah, yeah.

MR PALMER:

I don't have a copy in the system Sir but I have a copy that I can show the witness.

JUSTICE COOPER:

Where is the Sun Alliance building?

MR PALMER:

It is in Auckland Sir.

JUSTICE COOPER:

A. It is now called the Vero building.
Q. In Shortland Street?
A. In Shortland Street.
Q. Yes we will have a look at it.

CROSS-EXAMINATION CONTINUES: MR PALMER

WITNESS REFERRED TO PLAN OF SUN ALLIANCE BUILDING

A. Who were they designed by, were they designed by me?
Q. Well, your company?
A. My company?
Q. But you have a large company don't you it is Murray Jacobs –
A. Oh, it is not very large.
Q. – Limited, is that you?
A. Yeah that is my company yeah.
Q. It’s – what you are about to see is a plan which is stamped with the Auckland Council’s stamp and it shows what appears to be a late

installation of drag bars. I am afraid you will read the plan better than I, so it would be better if you have the plan.

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A. I have no memory of that. What's the date on the plan?
Q. The plan is dated 7 May 1998 it's signed. And it's the level 7 floor plan. It shows the late installation of drag bars.
A. Is it on one floor is it? How many floors is it?

JUSTICE COOPER:

Q. You'll get it in a minute. This one is a plan of level 7.
A. Right.

WITNESS REFERRED TO SLIDE – PLAN

CROSS-EXAMINATION CONTINUES: MR PALMER

Q. The plan's stamped, "Producer statement," so I assume it's at a relatively late stage in the process and you'll see that there are several drag bars put on there.
A. Yeah but they would've been put in before the building was finished.
Q. Yeah I wasn't suggesting otherwise Dr Jacobs?
A. Oh, well that's probably fair enough, I mean I think I could comment on relation to checking that the Auckland City Council had a very rigorous checking department when, in my experience, especially when was this designed, this was later, and they concentrated on seismic checking and so they well could've gone through my drawings and said, "We think you should put some drag bars in extra," and I might have to relook at that and put some drag bars in. I thought you were inferring that these had gone in at the later date after the building was finished?
Q. No, no I was just suggesting that it's not uncommon and for the late installation of matters of detail, and I think you're acknowledging now that you have in the past subscribed to that practice?
A. Yes I would do, but, well I don't know. These just happen to have been put in by hand in ink drawings but it doesn't say here when they were put in really does it?

Q. No, but it would indicate, wouldn't it, that after you'd finalised the plan there had been some attention to this and that drag bars were a necessary addition. Would you agree with that?

A. Well they probably, they weren't put in on the original draughtsman's drawing if that's what you mean, but they have been put in before the building was built, and I would say before a consent was issued for it.

Q. And presumably they will have formed part of the final as built structure?

A. I would hope so, yes.

MR PALMER:

I don't know what the procedure is for introducing this into the system but if the Commission wants it in it should be given a number?

JUSTICE COOPER:

Yes, well we'll receive it and it should be given an appropriate electronic life after that, including a number. Mr Reid?

MR REID:

I have no questions but I wonder whether we might see the plan before this witness departs Sir?

JUSTICE COOPER:

Q. Yes. Mr Jacobs.

A. I think it's a very different sort of thing to what's happened on the CTV and I mean these are just, that quite often happens where somebody, when they're checking through their drawings does put some extra bars in them. I mean I don't think it's, if they had been bolted on after the building was finished then I would agree with you more but I think what you've got there is just a something that probably with a recheck it may have happened within our own firm, with a known firm because I notice the person is a very senior engineer that had signed those drawings. Maybe I had checked them, maybe somebody else had checked them.

Q. Can you tell from the dates whether it would've been before a building permit was issued?
A. I can't tell that. I haven't looked specifically for that issue, but I would doubt it because they look like – no I don't, I can't tell you that.
Q. Would you like to have another look at it?
A. Yes I would, would have a look.
Q. So that building was erected, what, in the mid 1990s?
A. Yes it would've been erected about then yes.
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Q. Just have a careful look at the dates and so on I know it's difficult when these thing's sprung on you but…?
A. Well it seemed to be drawn, I mean it's very hard for this to be exactly right, it's got a drawn date there of the 11^{th} of the 16^{th} of '98.
Q. Ninety-eight did you say?
A. Yeah, yes '98 so 11^{th} of the 2^{nd} of '98 it was when it was actually drawn by the draughtsman but then these, it was signed which probably means is when it was signed for permit at the 7^{th} of the 5^{th} of '98 by a senior engineer.
Q. In your firm?
A. In my firm yes, but it doesn't have, it doesn't say when that was, when those bars, I don't think we can tell from that when those were put in there. It hasn't got anything here in the, there's a revisions list down the side. Normally when you put something on to a drawing post the building consent date it's listed as an amendment and it usually has a little cloud round it.
Q. Yes I've seen them.
A. The idea is that so the quantity surveyor can say well the building cost more than it did the day I tendered it you know so they can get some reward for what they're doing is extra but this, this drawing has got no revisions apart from revision (a) building consent tower so it hasn't got a revision post the building consent date.
Q. So what's that tell you?

A. That tells me that these drag bars were put in there before the building consent was done. Was given.

Q. And consequently they would have been part of the original construction?

A. Oh yes definitely yes you couldn't, you can't fit them in there, they're in the topping slab. There's no way you could fit them in afterwards. They are just extra steel that has been put in there to connect the returns in the shear wall to the floor slab.

Q. Right oh.

A. This building incidentally is, has got a moment frame round the outside of it, it was very heavily reinforced with seismic detailing and it has been recently checked and found to comply with the modern code so it has had design check on it very recently, independent.

CROSS-EXAMINATION: MR ELLIOTT AND MR REID – NIL

RE-EXAMINATION: MR MILLS – NIL

QUESTIONS FROM THE COMMISSIONERS – NIL

WITNESS EXCUSED

HEARING ADJOURNS: 3.28 PM

HEARING RESUMES: 3.51 PM

JUSTICE COOPER:

Now, Mr Palmer.

MR PALMER:

Yes Sir I'm going to ask Mr Latham to read his evidence which is contained within two, two evidence statements dated the 25th of July and the 1st of August and he's also prepared a PowerPoint presentation to assist in an understanding of the evidence that he's about to present because of its technical nature, but just before I do that I would like to make some introductory comments which are important because it's equally important that Mr Latham's evidence must be understood in context. I just want to make the following observations about Mr Latham's evidence.

First, he's done an exercise as part of his role on the ERSA panel.

Secondly, in doing this exercise he explores the available interpretations under the code as it was in 1986. Nobody else on the panel has undertaken the same exercise using the information that Mr Latham has used, that is to say the information that would have been available at the time of design in 1986. That's an important contextual issue. The work Mr Latham has done, however, has stimulated considerable further work by other experts many of whom have filed briefs now in relation to it and those experts are not all in agreement over the issues. Mr Latham does not necessarily disagree with the other experts but his work does show that there are other ways of interpreting the code. Mr Latham has asked me to say that it's unlikely that he would undertake the exercise that he's undertaken in a practical situation and with those introductory comments I now would like to introduce Mr Latham.

MR PALMER CALLS

DOUGLAS ALEXANDER LATHAM

Q. Is your full name Douglas Alexander Latham?
A. Yes.
Q. Sorry.

DOUGLAS ALEXANDER LATHAM (SWORN)

JUSTICE COOPER TO MR PALMER:

Q. Mr Palmer just to make sure I'm understanding what you're saying Dr Latham is going to present an approach which it will be argued could have been taken to comply with the relevant codes but not one which he would advocate.
A. Correct.

Q. Nor one which was in fact taken.
A. That would appear to be the case Sir, but it is relevant to the issue of compliance with the code.
Q. Yes but it's not an approach in which Dr Latham himself has any confidence. Nor is it one that Mr Harding took. How does it, what's the point of it?
A. Well Sir Mr Latham I would argue is competent to undertake such an analysis because –
Q. Well I have no doubt he it but my question is not as to whether he's competent to carry it out but as to what its point is.
A. The point Sir is to understand how the code could have been complied with at the time. It is certainly an issue before the Commission as to whether or not there was compliance with the code and I think to put that all in context it's helpful to the Commission to understand how compliance could have been achieved in the circumstances which Mr Latham has identified. It's certainly been the subject of considerable further consideration by other experts, many of whom have different views and I think that fact in itself is rather interesting and perhaps if I could jump forward to where I want to be at the end and that is that arising out of Mr Latham's work he has prepared a list of issues which, in respect of which I don't think there is unanimous agreement amongst any of the experts that are before you in the panel and it's in, his work is valuable in understanding how each of those issues is considered relevant to the code as it was at the time.

COMMISSIONER CARTER TO MR PALMER:

Q. Just by way of starters I think it might be helpful for those that are about to give evidence to us to see the way that the Commission is looking at dividing up the issues that are before it. So if those that talked to us subsequently would be able to sort of have in their mind the thoughts that we are trying to exercise in considering the matter which I'm sure everybody can see is a number of opinions and complex in itself.

First of all we do want to know about what would comply with the code in 1986.

Second, we want to know what would be best practice in regard to the codes, acceptable practice.

Errors that are present, were present in the code or difficulties in regarding to interpreting the code as it was in 1986 and

If any of those matters still continue into the present code.

So first of all the code as it was in 1986 and what would comply, the best practice that would have applied at that time, what would have been considered acceptable even if it fell short of best practice, what are considered to be errors within the code of difficulties in interpretation and, finally, do we still have any of those matters resident in our current codes.

A. Well thank you for that indication and I think that you'll find the evidence that Mr Latham is about to give does identify the key issues and examines each of them in turn to determine what are the bounds of each element in the code and what those elements were using the known information at the time. I suggest it will be of assistance to you, the discussion that ensues.

Q. Thank you.

A. With that I'll continue.

EXAMINATION: MR PALMER

Q. Is your full name Douglas Alexander Latham?

A. Yes.

Q. Do you reside in Christchurch?

A. Yes.

Q. Are you a structural engineer?

A. Yes I am.

Q. Could you please read your second statement of evidence commencing at paragraph 2.

A. In accordance with the requirements of rule 9.4.3 of the High Court Rules I confirm that I have read the code of conduct for expert witnesses and that my evidence complies with the code's requirements. Matters on which I express an opinion are within my field of expertise. I am employed by Alan Reay Consultants Limited (ARCL) an affected party in this Royal Commission hearing.

I hold a Bachelor of Engineering with Honours. I am a graduate member of the Institution of Professional Engineers New Zealand.

I have been employed by ARCL since January 2010 after completing my studies over which time I have worked on a number of analysis and design projects. My full resumé is annexed to this statement.

Q. So do you need me to take you to that?

A. No.

Q. Carry on please with paragraph 8.

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A. Involvement in CTV Building Analysis.

Together with Chris Urmson, another structural engineer employed by ARCL, I have been working with Dr Alan Reay on an investigation into the collapse of the CTV building. This work has included:

(a) Reviewing the draft reports by the Department of Building and Housing (DBH) in December 2011, and preparing comments on the draft report, a number of which were picked up and reflected in the final report.

(b) Carrying out a retrospective analysis of the building's compliance with the building code at the time of the design, and when the drag bars were fitted in 1991 as well as reviewing the DBH collapse scenario of the building using analytical tools designed for these purposes.

I have already filed three statements of evidence before the Commission, being a statement of evidence dated 31 May 2012, and two affidavits dated 5 and 6 June 2012.

In preparing this evidence I have referred to and relied upon the following principle sources of information:

(a) CTV building structural drawings,
(b) CTV building structural specification,
(c) the calculations,
(d) the reports prepared for the DBH comprising:
 (i) the CTV building collapse investigation for the DBH prepared by Dr Clark Hyland and Mr Ashley Smith, and
 (ii) chapter 5, CTV building of the expert panel report on the structural performance of Christchurch CBD buildings in the 22 February 2011 aftershock.
(e) NZS3101:1983 "Code of Practice for the Design of Concrete Structures,"
(f) NZS4203:1984, "Code of Practice for General Structural Design and Design Loadings for Buildings,"
(g) other information referred to in my evidence including the seismic analysis report I referred to below.

Following an interlocutory hearing in which such information was sought by ARCL and Dr Reay from the DBH and Compusoft Engineering Limited in early June 2012 ARCL received copies of the full input files used by the authors of the DBH report for the ERSA analyses. I've used these files to assess the inputs for the DBH ERSA analysis and to carry out further analysis.

Royal Commission ERSA panel.

I have been a member of the ERSA panel constituted as part of the Royal Commission into the collapse of the CTV building pursuant to the order as to directions in relation to the elastic response spectra analysis evidence dated 18 June 2012.

The outcome of that panel was reported in a joint report prepared by Professor Carr. Areas of agreement and disagreement are noted in that joint report.

I am in a disagreement with a number of issues relating to the ERSA. Most of those issues have been noted in the joint report to the Commission.

Due to my disagreement over some of the ERSA panel issues, I have carried out a further ERSA analysis. I have prepared a seismic analysis report, a copy is attached to this brief, which sets out the analysis I have taken and the result of the analysis.

I wish to explain how this report came about and its context in relation to other work that I've done. In particular I have undertaken three tasks arising out of the ERSA panel work. They are:

(a) First, I've run a static analysis of the CTV building to determine the forces and displacements that could be used in the design of the structure in 1986. In running a static analysis I did so with the assumptions that I consider are justified on a straightforward interpretation of the relevant information available in 1986.

(b) Second, I performed an ERSA using the static analysis to carry out the scaling required by the code. Of course in doing so I adopted the same assumptions used in the static analysis.

(c) Third, utilising the subsequent results of my analysis I am in the process of completing a design review of the columns with a view to ascertaining whether there was compliance with the code as it was in 1986.

Q. And is that work now complete and the subject of your third statement of evidence?

A. Yes.

Q. Carry on.

A. For the reasons explained below the report accompanying this part of my evidence only includes the first and second task noted above.

Q. I don't think you need to read the last paragraph of that sentence. Finish at, continue with 17?

WITNESS CONTINUES READING BRIEF OF EVIDENCE FROM PARAGRAPH 17

A. It is important for me to record that as a member of the ERSA panel I have always promoted the dual viewpoints that:

(a) first an ERSA was not required in the circumstances of the CTV building design, and

(b) second, if the ERSA was going to be run it should be re-run using inputs derived from 1986. I note here that any such analysis, including static analysis, should also be run using inputs derived from 1986 circumstances.

I understood from the, "Order as to Directions," dated 18 June 2012 that

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if agreement could not be reached amongst the ERSA panel on the Compusoft's ERSA reliability that a further ERSA would be carried out. Reference clause 4.4 of that order.

From the outset on the ERSA panel I have challenged Compusoft's ERSA reliability and as noted above have promoted the need for a separate ERSA to be completed using 1986 derived inputs. Notwithstanding the possibility contemplated in clause 4.4 of the order as to directions no separate ERSA was run by the ERSA panel. Accordingly, I determined that I would do an appropriate analysis myself.

In considering running an ERSA with the 1986 derived inputs I came to the view that running an ERSA was not required under the code. It was only once I got to the point of assessing the CTV building for running an ERSA that I reached a definitive conclusion on this issue. I concluded that instead it was feasible and appropriate for a static analysis to be run for the CTV building and that analysis was all that was required.

In the process of running the analysis it occurred to me that I then had all the information I needed to undertake a design review of the columns, measured against the code as it was in 1986, so I was in the process of completing that review but at the point that this brief was prepared had not finished.

Q. Now you have given the fact that you have now achieved what you discussed in the rest of paragraph 21, could you just continue with paragraph 22 please?

WITNESS CONTINUES READING BRIEF OF EVIDENCE FROM PARAGRAPH 22

A. Summary of seismic analysis report.

In summary my seismic analysis report on my static analysis and ERSA concludes:

(a) The CTV building is only of moderate eccentricity using the McCahon soil stiffnesses and therefore a static analysis can be used exclusively to determine the design forces and displacements compliance with the 1986 code.

(b) Under both the static analysis and ERSA the CTV building complies with the drift limits in the code.

It can also be noted that the design drifts determined from both the static analysis and ERSA are such that the columns appear to comply based on the criteria used in the DBH CTV building collapse report by Dr Hyland and Mr Smith.

Q. Thank you Mr Latham, just a couple of housekeeping matters. The seismic analysis report that you refer to in your evidence which is document WIT.LATHAM.0002.11. Do you have a copy of that with you?

A. Yes I do.

Q. Could you please go to page – what is page 15 in the Commission's reference and it is page 3 of your report under the heading, "Modelling inputs."?

A. Yes.

Q. There is a table, table 2 at the bottom of the page. Do you wish to make some corrections to the first paragraph below that table?

A. Oh, right, yes the reference for the coordinates of the building used in this report are mirrored that, of the DBH report. The calculation of the building mass used a different set of coordinates in the appendix so where it says in table 2 X and Y measured from the intersection of grid A and grid 1, that refers to the coordinates used in the appendix not the report.

Q. So should that be, should those references be changed from A to F and from 1 to capital A?

A. Yes.

Q. And further – when it carries on with positive X in the east direction, should that really mean north direction?

A. Positive X is in the north, positive Y towards the west.

Q. In the west. And if you could just turn the page of your report to page, what is page 16 of the Commission's version page 4 of the report, underneath paragraph 3.3 is the – that report that is dated there that is dated the 5th of June 2012 from Mr McCahon?

A. Yes.

Q. The reference for that is BUI.MAD249.0460.1. Now with those housekeeping matters out of the way, could you please turn to your third statement of evidence which is dated the 1st of August?

A. Yes.

Q. Could you read that report, beginning at paragraph 2 please?

WITNESS CONTINUES READING THIRD STATEMENT OF EVIDENCE PARAGRAPH 2

A. I refer to my second statement of evidence dated 25 July 2012 for full details of my qualifications and experience. I again confirm that I have read the code of conduct for expert witnesses and that my evidence complies with the code's requirements.

As signalled in my second statement of evidence I have carried out a design review of the secondary frames in the CTV building relevant to code of compliance.

I attach my report on secondary frame design review report dated 31 July 2012.

Summary of secondary frame design review report.

In summary my secondary frame design review report concludes:

(a) Consideration of gravity frames as secondary frames. It is reasonable to expect the gravity elements of the CTV building such as the beams and columns to be considered as secondary elements and detailed accordingly to the requirements of NZS3101:1982 outlined in clause 3.5.14. This was the basis of the lateral analysis carried out in my seismic analysis report dated 25 July 2012.

(b) Requirement of ductile detailing. Based on the equivalent static drifts determined in my seismic analysis report, the additional seismic requirements of NZS3101:1982 were not required to be satisfied as the imposed deformations on the secondary frame elements did not result in plastic behaviour.

(c) Column design. The design of the columns appears to be consistent with the provisions of the required codes and standards for load combinations involving earthquake loadings.

(d) Beam design. The design of the beams appears to be consistent with the provisions of the required codes and standards for load combinations involving earthquake loadings.

(e) Beam column joint design. The design of the beam column joints does not appear to be consistent with the provisions of the required codes and standards for load combinations involving earthquake loadings.

I elaborate and explain each of these conclusions in my secondary frame design review report.

Q. Have you prepared a short Powerpoint presentation which is BUI.MAD.249.0583.1?

A. Yes.

Q. If that could be brought up please. Now if you can take control of the mouse and, or what is the easiest way to do it. It is easier if you do it if you click your way through it and if you just explain each of the slides in the way that you wish to?

A. Right.

WITNESS REFERRED TO POWERPOINT PRESENTATION

A. Right so I will just start by explaining the context of the review. So obviously the context has been taken with a 1986 context, obviously when the building was designed. That involved the application of the standards NZS4203:1984 and NZS3101:1982. Those were the relevant standards in the Christchurch bylaw. The review does not intend to replicate what was done in the original design. It looks at what could have been done in 1886 so that is an important point.

The assessment has basically been undertaken in two phases: the first phase involves a lateral analysis of the building as a whole using ETABS to determine the drifts and displacements of the building under the code loads. From that then we can then look at the assessment of the columns under those drifts determined above and the assumptions must be consistent between the two phases. The two phases there, one and two are the two separate reports that I've prepared.

So it was part of the DBH report a 3D ETABS model was developed by Compusoft and one of the features of this model is that it used flexible foundations using the upper bound soil stiffness recommended by Tonkin and Taylor.

As part of the Royal Commission work a panel was set up to look at the ERSA and the areas of agreement and disagreement and to essentially determine whether that model made for the DBH report was the most reliable and I was a member of that panel and we requested that the foundation stiffnesses used in the model should reflect the recommendations by the original geotechnical engineer Mr Ian McCahon. Another area that came up was that the masses of the building should also reflect the 1986 information and assumptions in full. As part of that work there was no further ERSA analysis undertaken by Compusoft so I took that upon

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myself to carry out that extra work so I, we, we received the ETABS models and I adjusted the soil stiffnesses to reflect the recommendations from Ian McCahon. There was some other minor changes made to the model but the soil stiffness was the major change and then subsequently that work was presented in the reports attached to my briefs. In response to this analysis Compusoft have now presented further analysis and they've covered three cases: one, the first one a fully rigid base, the second using the most probable soil stiffness values recommended by Tonkin and Taylor, and the third the lower bound soil stiffnesses recommended by Tonkin and Taylor.

One of the effects of these varying foundation stiffnesses is a change to the natural period of the building so as the soil becomes more flexible the natural period lengthens. If we look at the fixed base model the period was about .8 seconds and if, for the Tonkin and Taylor stiffness ranges the period was in the order of one to 1.4 seconds and with Ian McCahon values the period lengthened to 1.2 seconds in one direction and two seconds in the north south direction. One of the other key changes was that it affected the relative stiffness of the north core wall to the south coupled wall and I've borrowed this figure that Mr Smith has prepared and we can see the centre of mass in the middle of the building there and the various positions of the centre of rigidity, so for the fixed base case the centre of rigidity is to the north of the building near line 5 as the, some soil flexibility is introduced that centre of rigidity moves towards the south and there's a range there indicated by Mr Smith and then using the softer soil stiffness recommended by Mr McCahon the centre of rigidity moves further south where it's shown in red there. This is an example at level 4 only and the implications of that are that it can be, the building can be considered using the definition in the loading standard to be of moderate eccentricity with the McCahon stiffness as opposed to a high degree of eccentricity with a rigid base or the Tonkin and Taylor stiffness and again the implications of whether it's of moderate or high degree of eccentricity relate to the type of analysis that can be used on the building. If it has a high degree of eccentricity then a ERSA or a 3D ERSA was recommended, not required but recommended, and that was in fact done in the original design as well.

Now one of the issues with using a flexible foundation model is how you treat it with, in relation to this clause 3.8.1.2 of the loading standard which stated: *"computer deformations shall be calculated neglecting foundation rotations"* and so a flexible foundation model is in conflict with this clause. If we used a fixed base model then there is no such problem. Now one interpretation of, if a flexible foundation model is used is that the columns still undergo the drifts and therefore they should be included. If you interpret this clause literally you are required to neglect the foundation rotations and so that is the basis of the analysis report that I have done which neglects the foundation rotations.

I've just got a quick summary of the different models. I'm not going to go into the numbers but I'll just highlight a couple of trends. The Compusof model with the Tonkin and Taylor upper bound stiffness which was that presented in the DBH report generally has the highest drifts. If we then look at the Compusoft model with the fixed base the drifts are reduced and then finally the ARCL model which uses the McCahon soil stiffnesses and neglects the foundation rotations the drifts are lower.

And again on, this is that previous one was line F, if we go to grid 1 we see the same trends the Compusoft Tonkin and Taylor upper bound stiffness has the highest drifts, the fixed based model reduced and the ARCL model with the foundation rotation components neglected has the lowest drifts.

And this is line 2, the internal columns, same trends again.

So then we can then go on to move to the second phase of assessment which is looking at the detailing requirements of the columns. And so clause 3.5.14.3 of NZS3101:1982 outlined the requirements for group 2 secondary elements such as the beams and columns and we've heard this a number of times before but essentially if the columns remained elastic then the additional seismic requirements of the code were not required to be met and if the columns did not remain elastic then the additional seismic requirements were required. They are fully ductile or limited ductile. So we can then ask ourselves well how do we determine if the columns remain elastic?

And there have been a number of approaches put forward so the first method there is to carry out moment curvature analysis, this was carried out by Dr Hyland and was presented in the DBH collapse report. A second method has been proposed by Dr Hyland which is the working stress method in Appendix B of the concrete standard and one thing to note with this method is it required different loading factors to be applied in the, the loading standard 4203 provided different factors and if, to use the working stress method only required 80% of the earthquake specified in 4203 so that just needs to be consistent if you're comparing. The third method there is an elastic frame analysis using uncracked properties and then assessing the demands on the column against the dependable strength and that's the method that Mr Smith appears to have used, and finally using an elastic frame analysis but using cracked properties and then assessing it against the dependable strength and that's the method that I have presented.

So talking about the degree of cracking one option to a designer is to use a simplistic assumption of uncracked properties for all columns. Another option available was to carry out a more detailed assessment equation 4.4 in the concrete standard provided a method for getting a more accurate picture of the cracking, now that if, essentially what that equation does for members that had a high degree of axial load on them such as the lower floor columns you'd expect less cracking because of the high axial loads and so you end up using the gross stiffness. For members, columns towards the higher end of the building with lower axial loads then you can expect more cracking and the stiffness is reduced to reflect that.

So if we just make a comparison of some of the methods and models that have been used to date, we have on, we're looking at grid F in the north south direction, the demands that have come out from the ETABS model and then scaled accordingly are presented on the left. There we show the Compusoft fixed based model and ARCL model with the McCahon soil stiffnesses with the foundation rotation component removed. And then on the right there we have the elastic capacity as determined by these, those different methods for assessing the columns which I outlined above. Essentially if the elastic capacity is greater than the demand then the columns are remaining elastic. If the elastic capacity is less than the demand then the columns are not remaining

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remaining elastic. So if we assess the different criteria that have been put forward to date with the different models we can see that using the

Hyland criteria then the columns are remaining elastic for both the Compusoft model and the ARCL model. If we use Mr Smith's criteria they are not remaining elastic under either models apart from the ground floor and using the criteria I put forward they are remaining elastic at all floors under the ARCL model and not at one floor under the Compusoft fixed base model.

Looking at that same scenario but on gridline 1 in the east-west direction the Hyland criteria remains elastic at all levels under the ARCL model drifts and it does not at one floor under the Compusoft fixed base model. Again, Mr Smith's criteria, they do not remain elastic for either model apart from the ground floor and then using the criteria I have put forward they remain elastic at all floors for the ARCL drifts. They do not for the Compusoft fixed base model drifts.

And, finally, on grid 2 which represents an internal column the Hyland criteria remains elastic for either model at all floors as does the criteria that I have put forward. Mr Smith's criteria does not at the upper floors.

So in summary the columns remain elastic using the ARCL drifts in either the Latham or the Hyland column criteria. Similarly for the Compusoft fixed base model drifts they remain elastic for most floors but not all. One of the key points to note is that there are different methods of analysis available to the designer. There are different assumptions that can be made and there are different interpretations of the code clauses. And the conclusions on compliance are dependent on the above methods and assumptions and as the slides before showed we can get some different results.

Q. Thank you Mr Latham and finally have you prepared a one-page sheet with a list of issues that you consider arise out of your evidence?

A. Yes, yes I have.

MR PALMER:

I think has been provided to the Commission Sir. I'm not sure if everyone else here has a copy but I've got additional copies here for those that don't. I provided it to counsel assisting. I thought it was to be provided to you.

JUSTICE COOPER TO MR PALMER:

Q. Yes we've seen it. I'm not quite sure what it's, this is just a sort of aide memoire is it?

A. Yes. Mr Latham thought that analysing all of the other evidence that has come in on this issue he's summarised in this one page the topics that, where there seems to be divergence of opinion and so it's a useful reference point to consider the issues that arise from the evidence that he and others have presented on these issues. That's all Sir.

JUSTICE COOPER:

Thank you. Now can I just ask have members of the expert panel seen the summary that we've just been through, that Mr Latham has just taken us through, the various slides?

MR BRADLEY:

We've seen his briefs of evidence but we haven't seen the summary just discussed.

JUSTICE COOPER:

The summary.

MR BRADLEY:

No.

JUSTICE COOPER TO MR PALMER:

Q. So it would be better if that was made available too wouldn't it?

A. Yes Sir I'll make sure that is. Sir it does have a number. It's in the system as BUI.MAD249.0583 at 1.

Q. Yes but in order for people to know what they're wanting to call up they'd have to be reasonably familiar with this wouldn't they?

A. I've got some extra copies here Sir.

Q. Yes well I think they should be made generally available because we're obviously not going to finish this discussion tonight are we.
A. No. If that could be circulated to the other members of the panel.
Q. It needs, are they colour photocopies? Have a look at the last –
A. Two colour photocopies Sir. It probably needs to be in colour.
Q. It does doesn't it. So in due course if you could make coloured copies available to all the members of the panel.

JUSTICE COOPER:
Now Mr Henry's just joined us. What's happened is that Mr Latham has read two statements of evidence, successively called his second and third briefs, to which was attached reports setting out his conclusions following the exercises that he's done. He's then gone through a series of slides which purport to summarise the contents of the two reports that he's given in evidence and that's brought us to this point.

MR MILLS:
Sir I've just realised that Dr O'Leary is supposed to take this aspect as well. It seems to be that now that Mr Latham's done his presentation as we've done in previous panels he could create another seat up there and we could –

JUSTICE COOPER TO MR MILLS:
Q. Well it depends upon the answer to my next question.
A. All right.
Q. Is it not intended that Mr Latham be cross-examined on what he's presented?
A. My understanding was that the –
Q. You'll need to be speaking into a microphone obviously.
A. My understanding was that the intention was that there would be the normal sort of panel discussion and then at an appropriate time, and I hadn't thought it would be initially, that the opportunity would be given to any lawyers who are here who wish to question on this but at least initially what I think counsel assisting had contemplated was that it

would simply go into the opportunity for these other panel members who are lined up here, all of whom have provided briefs responding to Mr Latham, which are being taken as read, that there would be an engagement in the normal way on the issues that Mr Latham has raised. Whether the lawyers present do want to ask questions, that's a matter for them.

JUSTICE COOPER ADDRESSES COMMISSION:
Well is everybody content to proceed in that way?

UNKNOWN SPEAKER:
Yes Sir.

JUSTICE COOPER:
Yes, right, Commissioner Fenwick.

COMMISSIONER FENWICK:
Q. Just to start things off. You have done a degree course?
A. Yes.
Q. You've spent three years or thereabouts specialising on civil engineering and design.
A. Yes.
Q. And the same was true in 1960s. So Mr Harding would have gone through the courses as well. Mr Harding would have gone through the courses in 1970 I think, but about that time, but over that time there has been a three-year course and it's necessary would you agree for engineers to go through that training and to have some experience before they're capable of design. Would you agree with that?
A. Yes, yeah.
Q. It's essential isn't it do you feel, it's essential they have that training so they understand the code and how structures work so they can work out how to apply that standard?
A. Yeah, it's important working with someone experienced, yes.

Q. You don't think the initial training is, the initial under-graduate degrees, courses in geomechanics and so on are important?
A. The, the under-graduate degree is important? Is that what you're asking?
Q. Yes.
A. It's very important, yes.
Q. So it's important they know some geomechanics, they know structures –
A. Yes, absolutely.
Q. – they know how they're designed, okay, right, and so when we're looking at what you're proposing –
A. Mmm.
Q. – I can't myself see it. I just wonder how you've, I can't see it in terms of just a code because someone's got to have background experience to be able to apply that code.
A. Yes.
Q. And therefore isn't a mixture of two items we're looking at? What would an acceptably competent engineer do in this case?
A. Mmm.
Q. If you agree it's more than just what the code says and the way you interpret it 30 years later.
A. Yes well I mean the approach, the approach that we've undertaken in this exercise is to then look at what the code says and what the code requires.
Q. Right. Well just to illustrate the point.
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Q. Well just to illustrate the point –
A. Mmm.
Q. – you have used the recommended soil stiffnesses from McCahon and Co in 1986 and those were derived for structures for long-term gravity loading, is that correct?
A. Apparently yes.
Q. Yes, and we know that what happens, because you've done geomechanics, two or three courses of it, we know that occurs because

the water drains out and settlement occurs gradually. On the sand it's relatively fast, on silts it's slower, it takes a few days and so on. So they are slow period times, aren't they? Now we know if we look at the record of the letter you got from, I've forgotten the name, but the letter you got recently asking about the properties, he says in that letter does he not, that okay it may be that liquefaction occurs or partial liquefaction occurs and the stiffnesses values determined for the long-term loading might possibly have been appropriate for earthquake loading?

A. Yep.

Q. Now what I'm saying, as an acceptably competent engineer, could you possibly design or analyse a building only on those soil stiffnesses? Is it possible and conceivable that anyone who had had two years' experience had known that normally the dynamic stiffness is very much higher than the other stiffness, would use that on the basis that it may have been possible that the soil was more flexible in liquefaction, in fact they didn't know much about liquefaction until the 1990s but would that have been a possibility, you could have possibly designed it only on that case or that being an extreme case that you would've looked at, now what's the answer?

A. Yeah I mean it's important to understand the range, and I mean I think initially if we look at this, we had one model with one value presented and one of our requests was to look at the effects of what would happen if other values were inputted, including those that were put forward by Mr McCahon as recommended in 1986.

Q. For long-term loading?

A. Yeah, well so now we, now we've looked at that and we've also got these additional models now, so I mean we're in a much better position to actually assess what the structure should be designed for or assessed against. I think it's important to look at the range though and not just have this one model that we had to start with which, which I mean –

Q. So your judgement then on whether this building, the columns and members – and we'll talk more about that later, go inelastic is going to based then not on just your minimum soil stiffness. It's going to be on the likely higher stiffnesses as well, is that correct?

A. Yep, well that's correct yeah I mean and in say my presentation that's why I've presented both you know the rigid case and also the most flexible case. So I mean we in effect have that range there.

Q. So wherever the rigid case controls and says this is critical, that's the value we use?

A. Yeah.

Q. And that's what is acceptably right? Now I've got a second question too. You've taken the drifts that you're checking for off your ERSA analysis?

A. Correct.

Q. Now would a rationally competent engineer do that? Now, the reason I'm saying this is you have done an elastic model and you've predicted the deflection on that. But we know that plastic hinges form don't they?

A. Yes.

Q. All right, so your elastic model at the base would have very little of that deflection because of the stiffness of the ground, but we know in fact, do we not, when it forms a plastic hinge it's going to rotate?

A. Correct.

Q. Correct, right, thank you. So the inter-storey drift you get off your ERSA analysis, how are you going to adjust for that? I mean I'm saying would not an acceptably competent engineer recognise that you can't just take and ERSA value and use it?

A. Well I mean the numbers that come out of the ETABS analysis are scaled.

Q. What way are they scaled?

A. Well they're, there's, in the 1986 or the 1984 loading standard they were scaled by factor K over SM which in effect allowed for the ductility.

Q. Well I'm going to say to you I don't think a reasonably competent – I agree that's what it says there –

A. Mmm.

Q. – but I think a reasonably competent engineer would recognise that when you form a plastic hinge at the base of a structure that's not going to give you the correct scaling because we know that when you form a plastic hinge it's going to rock over. So you've got your elastic

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Q. deflections plus your plastic deflections. So I've a question, any engineer who took that value as that, can I judge that as a reasonably competent assumption? A reasonably acceptable assumption? Now would you agree or not?
A. I would agree in today's terms. In 1986 I couldn't tell you what the standard procedures were.
Q. You don't think in 1986 they knew about plastic hinges?
A. Oh, I'm sure they did, yes.
Q. And you don't think they knew about the way deformations occurred?
A. Yes I'm sure they did.
Q. So if they knew that a plastic hinge was going to form, or your coupled wall was going to lean over like that, wouldn't it have been grossly incompetent to ignore that when you're assessing the deformations on your columns?
A. Well they're not, they're not really ignored because they're taken into account by this increased, increased factor that's dependent on the ductility.
Q. So for simplicity –
A. Mmm.
Q. – when you've got zero rotation of your column at the base –'
A. Yep.
Q. – when you multiply K over SM does it increase from zero?
A. No.
Q. So it's not correct is it?
A. No.

WITNESS EXCUSED

DEREK BRADLEY (AFFIRMED)
BARRY DAVIDSON (AFFIRMED)
CLARK HYLAND (AFFIRMED)
ASHLEY SMITH (AFFIRMED)
JOHN HENRY (AFFIRMED)
ARTHUR O'LEARY (AFFIRMED)
DOUGLAS LATHAM (AFFIRMED)

JUSTICE COOPER:
The first point that has arisen I think is as to well in two ways I think. What Commissioner Fenwick's "reasonably competent engineer" might have done to apply this code in 1986, and I wonder if Commissioner Fenwick could just in a brief way put those two propositions that he's discussed with Mr Latham to the general panel for response?
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COMMISSIONER FENWICK:
The propositions I put were how do you treat the soil information from 1986 where you are given a soft soil parameter, a comment that this may possibly apply for dynamic analysis, the person that wrote that said they are not an expert in this area. That was the first proposition. Whether that is sufficient to do an analysis just on and draw conclusions on that.
And the second one was the inter-storey drifts. Is it rational to scale ERSA ones or should one be making what are acceptably competent engineer actually have made some allowances for the inelastic deformation knowing that scaling ERSA values does not give you the correct reflected shape particularly in the lower regions of the building where the axial loads are higher? So those were the two points I was really discussing with Mr Latham.

JUSTICE COOPER:
Can we start with you Dr O'Leary, somebody who was probably designing buildings at the time.

DR O'LEARY:

The first question related to the treatment of the soft soil stiffness, is no, it is not a legitimate stiffness to use. I might point of course that it does lengthen the inter-storey drift because it lengthens the period but neglecting that which isn't an insignificant point, no the soft soil stiffness is not legitimate. We all know that stiffnesses under a dynamic situation tend to get pretty high. It is the first question.

JUSTICE COOPER:
Well we will deal with them one by one shall we.

DR O'LEARY:
Yeah.

JUSTICE COOPER:
Is there anything else you want to say on that one?

DR O'LEARY:
Not on that one, no.

JUSTICE COOPER:
Mr Henry?

MR HENRY:
So you want just the first question about the soil that we are talking about?

COMMISSIONER FENWICK:
Yes the use of the soft soil and the analyses Latham carried out, Mr Latham has carried out and then he has used those to analyse the structure and then remove the soft soil deformations out of the structure. So the question I had was it legitimate to use the soft soil only knowing that this was an absolute – well from what I interpret as being a likely lower, a possible lower limit of stiffness.

MR HENRY:

Okay, well in the 1980s it would essentially be unheard of to even consider modelling the soil underneath the building. The standard procedure was a fixed base model which tended to keep the periods short and the load at the highest level and therefore produce a stiffer structure but I don't, I mean the programme ETABS it couldn't really deal with an equivalent stiffness for the soil properties, I mean it could be done by the dummy storey as it was called but really it would just be an approximation would probably be a reasonable word for it, you'd have to try all sorts of different values and then try and correlate them with what is on the site to even consider that you had something realistic so that's, I mean really it was just not done. If you were, if you did have soft ground and you really thought it through, in terms of using the, I mean I should just take one step back. We did do a job once with soil properties which was the Union House base isolated building, that was 1982 where we had the piles, essentially a building on stilts in tubes and restrained it ground level by the dissipaters, energy dissipaters and we had to model the ground to make sure they didn't come into contact which was done with the two dimensional programme ReMoKo at University of Canterbury. It was a very expensive, drawn out sophisticated analysis. Professor Carr helped us through that, so it was really quite an extreme thing to even consider it. Anyway, taking a step back from that, in terms of what it actually means though if you are using the soil rotations if you were to. What the code was really saying you don't need to take into account these flexible or the soil flexibility for building deformations because it is really, considering that the whole building is going as a whole and all the relative inter-storey deformations are the same providing the whole building moves as one unit but, then that would be fine if you had a one big foundation and everything sits on one pad but when you have stiff foundations on the ends and isolated pads in the middle with flexible columns on them then you are going to get a difference where the flexible columns want to stay behind in the ground storey at least and, but the upper storeys they'd be constrained the diaphragms holding everything together. So the ground storey, the rotations of the end walls in the case of the CTV would drag the ground storey columns with them and enforce them to take, undergo the rotations. So if you have calculated I think as in the case of Mr Latham has, rotations that were three times the fixed base deflections, there is four times the deflections would be imposed on the ground storey columns.

COMMISSIONER FENWICK:
Can I summarise that -

DR O'LEARY:
Yes.

COMMISSIONER FENWICK:
- by saying that except the competent engineer would recognise that you could not remove the soil deformations where the structure was separated on, supported on individual pads?

DR O'LEARY:
Yes, yes.

JUSTICE COOPER:
Dr Davidson, do you have a view on this issue?

DR DAVIDSON:
No my view is more or less in line with, I guess Commissioner Fenwick's, it is very difficult –

COMMISSIONER FENWICK:
I have no view I am just asking questions.

DR DAVIDSON:
Oh, okay. Okay, your summary then shall I say. No, no I was involved with many analyses though I am not a structural designer so to some extent I was guided by the designers that I work with and –

JUSTICE COOPER:
What was the practice in the 1980s?

DR DAVIDSON:
The practice was as Mr Henry said, it was a fixed base. Typically the analysis was done with a fixed base.

JUSTICE COOPER:
Dr Hyland.

DR HYLAND:
Yes, there is a number of points. I think the first thing is when we are looking at code of compliances to consider how these codes are developed and the process of development and that there's the code committee takes a body of knowledge that has been derived through research and through academic processes and then that is transformed into a set of principles that are then agreed and then prescribed qualitative, prescribed quantitative measures are then developed by a code committee that it is assumed an experienced engineer will then apply and so when one approaches interpretation of these things should used a fixed base shouldn't you – the requirement is that you interpret the full body of the standard, the qualitative and the quantitative proportions of the clauses. I think we have quite an interesting contrast between the last two witnesses in that you have Dr Jacobs and you have Mr Latham taking probably quite different approaches, one perhaps looking at the qualitative and the one looking at the quantitative only and you know, it is not the intention of a standards committee to just restrict interpretation just to quantitative clauses. I know through my involvement with steel structure standard that we were asked, could you make everything very quantitative when we develop the revision to the standard and we had to say, no you can't do that, because every design is different, every situation is different, you are expecting an experienced engineer to use the principles which have been set down in a way that another experience engineer could look at it and say, yes,

you have applied this code in an appropriate manner to a situation, a specific situation which couldn't be totally quantified by the quantifiable requirements. So in answering the question of, should you allow for soil deformations, the experienced engineer would say yes you must because that is a principle of design. So I don't think it is appropriate to just take a clause and say, that is all you have to do because that is a convenient quantitative measure.

COMMISSIONER FENWICK:

The question Mr Hyland really was, could you base, rationally base a design on the low soil stiffness derived from a settlement foundations, with the

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qualification from the person that knows they're not an expert in this field but may be if there was liquefaction or partial liquefaction that might be appropriate. My question is would it be appropriate to base an analysis on that soft soiled or would you need to do one at the other extreme where you took a more realistic stiffness?

MR HYLAND:

Well the first answer to that is that well first of all you would get advice from an experienced geotechnical engineer and they would give you a report and they would give you limits and they'd give you say, "here's, here's a bound, here's a lower bound, here's an upper bound". And you would then use that as your, as your basis for a design. The – my understanding of the, the advice that Mr McCahon gave is that he said it was qualified just for settlement purposes and not for dynamic uses so he wouldn't be using that for a compliance check in my view. The other, the other issue related to this is the complication with the, the Harding model is that they've used a, they say they've used a fixed base structure yet the period of vibration was 1.05 seconds which doesn't seem to conform with what you would have for a fixed base. If you used the fixed base then the south wall would be much less compliant with the standard if you use the 1.05 seconds you're getting closer to compliance but there's a, there's a, there's an inconsistency in the approach that has been taken in the calculations that either they used a fixed base or they used some sort of intermediary approach it's not clear because we don't have the, the analysis results.

JUSTICE COOPER:
Yes Mr Smith anything you wish to add to that?

MR SMITH:
Yes I'd just like to, yeah my recollection being a practising engineer in Wellington at the time was that we modelled things with a fixed base, which was the accepted practice at that time. I would like to refer to a paper that was included in Dr, Professor Carr's report for the ERSA panel that if I can give a reference, WIT.CARR.0002B, page one initially. Can we put that on the screen?

WITNESS REFERS TO REPORT

So we did discuss this in the ERSA panel and I think we agreed at that time that the fixed based model was common practice. However this, there's this paper that I referred to dated 1980 in the Earthquake Society Bulletin that was directly relevant so, so we're looking at, if we can zoom up on the paragraph 2 on the right-hand column please.

JUSTICE COOPER:
Well what page is this.

MR SMITH:
This is page 46.

JUSTICE COOPER:
WIT.CARR.0002B.46.

MR SMITH:
Forty-six. Sorry about that, okay the, the title of the paper is The Analysis and Design of the Evaluation of Design Actions for Reinforced Concrete Ductile

Shear Wall Structures. So that is directly relevant for the CTV building and on that top right-hand paragraph, paragraph 2 I'll just read it out:

"Deformations of the foundation structure in the supporting ground such as tilting or sliding are not considered in this study as these produce only rigid body displacement for the shear wall superstructure. Such deformation should however be taken into account when the period of the structure is being evaluated or when the deformation of a shear wall is related to that of adjacent frames or walls which are supported on independent foundations."

So this is relevant to what we're talking about and I would consider this paper dated 1980 whilst it may have been common practice to model things with rigid foundations because that was the way the software worked in a, in most cases, this paper would have been common knowledge and I would consider that to be best practice shall we say so that we should be aware that when we're modelling it with a fixed base there are limitations to that and that would have been known at the time. Just going back to the, to the, our, the Smith, the Hyland Smith report modelling with the upper bound soil stiffness that we were given by Tonkin and Taylor the reason for that is that we knew it was critical that the column drifts we were assessing would determine whether the columns complied or not and the most, the least onerous drifts shall we say would come from the case with the upper bound soil stiffness so that's the reason we used the upper bound stiffness rather than the lower bound because the lower bound stiffness would give higher drifts. So that's probably the points I had.

JUSTICE COOPER:

Thanks you, now Dr Derek Bradley, have you anything to add?

DR BRADLEY:

I'd just on the first question I don't believe it would be appropriate to base a dynamic analysis on a long term settlement stiffness and I'd like to add that if there is uncertainty on a parameter something like a soil stiffness then and it, then an engineer would be expected to do a sensitivity analysis on that particularly if it did affect the results considerably. On the second question.

JUSTICE COOPER:

Well we haven't arrived at that yet.

DR BRADLEY:

Oh yeah, sorry.

JUSTICE COOPER:

Now Dr O'Leary at one stage you looked as if you wanted to say something more, this is your opportunity.

DR O'LEARY:

Thank you. From maybe '85, '84, '85 in our practice in Wellington we did actually use foundation flexibility but it was always used for pile supported buildings rather than pad supported buildings so that rotation at the head of the pile was insignificant apart from the ground first floor.

JUSTICE COOPER:

You'll have to, I'm sorry, you'll have to stay on contact with the microphone. Could you just repeat the last sentence please?

DR O'LEARY:

So we used foundation flexibility but the rotation that it imparted to the head of the pile was as relatively minor influence on, well it was a very minor influence on anything above that because we had to develop plastic hinges at ground level anyhow and the reason we used foundation flexibility was it lengthened the period of the building so something more appropriate to real life. It also increased the interstorey drift a little which for separation of window systems was again more appropriate to real life so yes we, I don't agree that it was a totally uncommon practice but by default we complied with 3.8.1.2 which says computer deformation shall be calculated neglecting foundation rotations. The way we used it it didn't actually make any difference.

COMMISSIONER FENWICK:

Right so did you remove the deformation due to the foundation deformations?

DR O'LEARY:

No this is only piled structures. What we used to do was pin the pile at an appropriate depth and use soil springs horizontally I think to try and have a realistic view of the flexibility of the soils above and we used for that particular exercise a ASCE paper from about 1956 which got so dog-eared and until Professor Pender's paper came along on horizontal reactions on piles then we could use that but in the mid '80s it was this ASCE paper.

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COMMISSIONER FENWICK CONTINUES TO DR O'LEARY:

Q. So your analysis was purely stiffness in the horizontal direction.
A. Yes.
Q. The vertical was very high because it was just the axial stiffness of the –
A. The axial stiffness of the pile was that –
Q. Down to four, four pile diameters?
A. Well usually it ended, sorry, usually it ended up about six pile diameters if you pinned them but the vertical flexibility of greywacke is pretty high, pretty low sorry.

JUSTICE COOPER TO MR LATHAM:

Q. Mr Latham is there any response you wish to give to the discussion on this point?
A. No, again, I mean some of the other issues about, you know, sensitivity being raised, I mean again we were presented with one model. We went to the original geotechnical engineer to see what his thoughts were and he gave us his thoughts and that's what I used to put in the model to see what effect it would have.

COMMISSIONER FENWICK TO MR LATHAM:

Q. He gave you, he said "might," "may," "possibly." He indicated, he didn't say this was a value you should use. If you look at his document he's saying, I'm not an expert in this field, it may be, it's possible, very different from saying this is a value you should use. That's what, I think if you look at it, go back, I think you will interpret this as likely to be an extreme value but I'd, perhaps you can answer that tomorrow, go back and have a look.

A. Yes I'd have to have another look at the, look at the letter.

HEARING ADJOURNS: 5.06 PM

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