HEARING RESUMES ON THURSDAY 9 AUGUST 2012 AT 9.30 AM

MR MILLS ADDRESSES THE COMMISSION

As the Commissioners will be aware what we're starting today is the session on Code Compliance and then in addition there's an aspect of Code Compliance but the distinct issue of the further ERSA that's been run by Mr Latham and issues around that. But I will just run through it in a little bit more detail with that broad overview.

On the question of what I would think of as the interpretation of the Code part as opposed to the ERSA part of this hearing which is scheduled to run today and through Monday morning, the first witness on that will be Murray Jacobs who of course is being called by Counsel Assisting and I don't know whether you've got the latest schedule in front of you, not just the daily schedule, but the hearing schedule. If you do, I'll just give you some batting order in that.

JUSTICE COOPER:

No. Commissioner Carter has got something he's printed off the internet.

MR MILLS:

I hope it's the latest one. On that one it's got Clark Hyland, second on that list. He's actually only going to be involved in the ERSA 'hot tub' so he isn't in fact the second witness to be called. The next witness after Murray Jacobs will be Ashley Smith who will read his brief number 3 which deals just with the code interpretation issue. He's actually here wearing two hats in effect. He's going to be giving that brief on interpretation of the code, then he'll also participate in the 'hot tub' on the ERSA issues.

Following Ashley Smith, Mr Arthur O'Leary will be called by Simpson Grierson and then following that, as on this schedule, Mr John O'Loughlin, also being called by Simpson Grierson and he will be dealing with the question of what a Council in the 1980s could have been expected to pick up in doing the permit review, what could be expected of a Council building officer at that time in relation to non-compliant aspects of the building application.

Following that, we will hear from Mr Latham and he will read his briefs in the normal way, take us through that and then we will convene a 'hot tub' where all the other witnesses listed from thereon down as far as David Harding and Alan Reay, their briefs will simply be taken as read and they will then go into a 'hot tub' responding to and discussing the issues that have been raised by Mr Latham and putting forward their various different views which of course also enable engagement with the Commission in a fairly comfortable way.

JUSTICE COOPER:
That sounds sensible to me. Is that an agreed procedure?

MR MILLS:
Yes it is.

JUSTICE COOPER:
One other thing that would be helpful would be if there could be an agreed list of the briefs that are to be taken as read. Can that be done please because there's been so many briefs, amended briefs, and third supplementary briefs that I would like the comfort of knowing precisely what it is that we are supposed to be....

MR MILLS:
Yes I will arrange that. It can pretty readily be done but I can understand why it's become rather an avalanche.

Finally, in this session on Code Compliance Mr Harding and Dr Reay will be called again. It's, of course, over to others as to what they do at that point but from my perspective it's principally to give them the opportunity to respond to the evidence that by then will have been given on code compliance and to deal with that in whatever way they wish to.

So, with that said, unless there's any other questions about that I will call Dr Jacobs as the first witness.

MR MILLS ADVISES THE COMMISSION THAT MR FAIRMAID WILL BE CALLED ON MONDAY FOLLOWING THE CONCLUSION OF THE CODE COMPLIANCE SECTION.

MR MILLS CALLS
MURRAY JACOBS (SWORN)

Q. Your full name is Murray Lionel Jacobs?
A. Yes that's correct.
Q. You are a Civil and Structural Engineer?
A. Yes.
Q. And a Director of Murray Jacobs Limited which is a Civil and Structural Engineering Consultancy Practice based in Auckland?
A. That's correct.
Q. You have a Degree of Bachelor of Engineering with Honours and a PhD in Engineering?
A. Yes I do.
Q. You're a Member of the New Zealand Institute of Engineers?
A. Yes.
Q. A Chartered Professional Engineer?
A. Yes.
Q. And an International Professional Engineer?
A. Yes I am.
Q. Now do you have in front of you, I think you should have two briefs of evidence. The first one is dated the 1st of June which was your principal brief. Do you have that in front of you? I know it's just been handed to you so you might just want to check. If you look at the last page of that brief which is page 18 on the numbering in the top right-hand corner. That will have a date on it and it should be signed by you. It's the brief itself rather than the appendices that I want you to read.

A. Yes on page 18 I see.

Q. And it's dated the 1st of June?

A. Yes.

Q. And then you've also done a reply brief which was a response to the evidence of Dr O'Leary. Do you also have that in front of you?

A. Yes I believe I have, yes.

Q. Again could you just check the last page of that which is page 13 and just ensure it's the signed copy that you have in front of you?

A. Yes I have the signed copy in front of me. It has my signature on it.

Q. Well if you could just go back to the first of those two briefs and just start reading your evidence at paragraph 3 and read it through. I may have one or two things to ask you on the way through and I understand you've got one or two points you want to elaborate on but, subject to that, if you could just start reading at paragraph 3.

A. "Paragraph 3 – I have over 35 years' experience in the design of structures in Auckland. Many of these structures have been in the CBD. Some of the buildings that I have been involved in are

(a) Vero Centre, Shortland Street, a 40 level office tower;
(b) PWC Tower, Quay Street, a 30 level office tower;
(c) ASB Tower, Albert Street, a 35 level office tower;
(d) Sylvia Park Shopping Centre;
(e) Quay West, Customs Street, Apartment building;
(f) BNZ Tower, Lower Queen Street.

I have read the Code of Conduct for Expert Witnesses. I agree to comply with the Code and I have prepared this statement in accordance with it.

Instructions:

I have been asked by Counsel Assisting the Royal Commission to provide evidence to the Commission that addresses the following issue:

Whether on 30 September 1986, being the date on which a building permit was issued by the Christchurch City Council for what is now referred to as the CTV building, the building complied to the

Christchurch City Bylaw number 105 (1985) and the relevant standards, standard specification and codes of practice listed in the second schedule to that bylaw.

In preparing this statement I have reviewed and had regard to the following documents:

(a) NZS 4203 1984 Code of Practice for general structural design and design loadings for buildings

(b) NZS 3101 part 1 1982 Code of Practice for the design of concrete structures

(c) NZS 3101 part 2:1982, Commentary on the design of concrete structures

(d) Structural drawings, office building, 249 Madras Street, Alan Reay Consultants, S 1 to S 39 (of the permit plans)

(e) Christchurch City Council Bylaw number 105 1985

(f) CTV building collapse investigation for the Department of Building and Housing 25th of January 2012 by Clark Hyland and Ashley Smith.

(g) Calculations seismic, Alan Reay Consulting Engineer.

The requirement and intentions of the three relevant codes of practice applicable to the time of the design will examined and compared with the design of the CTV building as shown on the permit plans. The questions asked will be;

(a) does the design comply with the codes and their intentions?

(b) if not what parts of the structure did not comply?

(c) how significant were any errors of non-compliance to the ability of the building to withstand an earthquake?

Codes:

The building design was required to comply with the Christchurch City Council Bylaw number 105. They required the building to be designed to the current New Zealand codes. The three significant codes all included in the Bylaw number 105 are: (and I've copied it there)

Code of practice for general structural design and loadings

3101, the design of concrete structures –

Part 1:1982, code of practice for the design of concrete structures

Part 2:1982, commentary on the design of commentary structures.

Q. And just to confirm that. I understand that what you've just read out was – is copied directly out of the Bylaw?

A. Yes it is, that is, yes it's copied and pasted on there. That's why it has the colour behind it.

NZS 4203 outlines the requirements for general structural design and gives design loadings to be taken for the design. It covers the gravity loads such as dead load and live loads, wind loads and seismic loads. NZ4203 states in the forward to the code on page 8 that:

"It aims at setting minimum standards for the general run of buildings".

Q. Now could I just ask you to pause there.

MR MILLS ADDRESSES THE COMMISSIONERS:

Commissioners would it be helpful as we go through so you can see the context given that a lot of these quotes are truncated, to actually bring up the relevant code pages from which the quote has been taken. I have thought it might be so you can see the context but it's –

JUSTICE COOPER:

No, I think I we want to we can follow it.

MR MILLS:

I suppose you've got copies of the code there haven't you?

JUSTICE COOPER:

Q. Except that I would just not mind finding that first one which is – that 1984 code Mr Jacobs was – there wasn't a commentary with that?

A. It is part of the code, the commentary's in the same code, if –

Q. It's in the same document?

A. Yes the same document, if you open up into a typical page you'll see on the left-hand half of the page is the column with the C in front of the clauses, so that's where the commentary is part of that.

EXAMINATION CONTINUES: MR MILLS

Q. Thank you Dr Jacobs, if you could just pick up reading where I interrupted you. I think you were however on page 9.

A. However, on page 9, it cautions that:

"Designers should recognise that the precise properties of construction materials and structural elements made from them are not clearly known. Furthermore, the interaction of these elements in the building frame under load is extremely uncertain, so that the total design technique is one of some degree of imprecision".

On page 33 under the section part 3 earthquake divisions, the first clause number 3.1 states:

"Symmetry. The main elements of a building that resist seismic forces shall as nearly as is practical be located symmetrically about the centre of mass of the building".

The CTV building does not comply with this instruction. The primary resisting elements in this structure are asymmetrical in the east-west direction. In the north-south direction the eccentricity is less. The main resisting element is the concrete core wall between lines 4 and 5 situated completely outside the main floor plate envelope, north shear core. There is a much smaller less stiff coupled shear wall on the south side of the building on line 1, coupled shear wall.

Q. Okay, can I just pause you there, when I think when you read that sentence you left out much when you read there is a smaller less stiff, do you mean to include that word much?

A. Yes I do mean to include that word much because I believe it is much less stiff.

Q. Paragraph 13.

A. The diagram shown below taken from the Hyland Smith report shows the large separation of the centre of mass from the centre of stiffness and consequently rotation. The building will rotate about the centre of stiffness during an earthquake and place a greater demand on some of the columns especially those further away from the centre of stiffness.

Clause 3.4.7, horizontal torsional moments:

"3.4.7(c) For irregular structures more than four storeys high, horizontal effects shall be taken account by three-dimensional modal analysis method of clause 3.5.2.2.2".

The commentary cautions in (C)3.4.7.1:

"It should be noted that even a three-dimensional analysis may not always give good predictions of a dynamic behaviour of very irregular buildings. It may indeed seriously underestimate earthquake effects in some cases".

NZS4203 states on page 33 in clause 3.2 ductility:

"3.2.1 The building as a whole and all of its elements that risk seismic forces or movement or that in the case of failure are a risk to life, shall be designed to possess ductility". I've got "note 3.4.8, but that is deleted in the 1986 version".

The columns in the CTV building were a risk to life if they failed and they should have been designed to exhibit ductility. They were not. Concrete is a brittle material, these columns are small in diameter and are not detailed to provide ductile action. That is they are prone to fail in a brittle manner when subjected to reverse cyclical motion such as in an earthquake. The usual failure mode is for the concrete outside the reinforced core to fall off the columns leaving a severely limited cross section of remaining concrete column to carry the load from the floors. Concrete is strong in compression but has limited reliable strength in tension. To make up for this characteristic the concrete columns and beams are reinforced with deformed steel bars. They bond with the concrete and carry any tensile loads developed from bending moments and, importantly, shear loads.

JUSTICE COOPER:

Q. Mr Jacobs, can I just take you back to the end of the quotation in paragraph 15 where you say clause 3.4.8.1 is deleted.
A. Yes.
Q. In the 1986 version, I don't understand the significance of that.
0950
A. No I agree with you I don't think it is very significant. I think what is significant is that in that code of 3.2.1 as I have stated there.
Q. Was 3.4.8.1 another clause dealing with similar subject matter?
A. Yes, to be – I don't remember why I put that in the brackets in there.
Q. For our purposes though as far as you are concerned can we just draw a line through it?
A. I would like to draw a line through that yes.

EXAMINATION CONTINUES: MR MILLS
A. And 17?
Q. Yes.

WITNESS CONTINUES READING BRIEF OF EVIDENCE FROM PARAGRAPH 17

In inductile columns these steel bars also serve another role. They confine the concrete inside the ties and contain it from breaking up and falling out of the column completely under the repeated cyclical loadings typical in an earthquake. Experiments have shown that if the ties are close enough and of sufficient strength they in conjunction with the vertical longitudinal bars are able to contain the concrete inside the area of the ties and thus provide a functioning, if reduced, area to carry the load of the column. If however there are insufficient ties to the column the concrete will fall out from within the inner core of the column during the reverse cyclical loading from an earthquake and the column will fail. The columns in the CTV building could be expected to fail an earthquake because of the – of insufficient ties. The columns were reinforced with six, should be diameter, longitude, oh, sorry the columns were reinforced with six longitudinal bars, 20 millimetres in diameter contained by six millimetre spiral ties at 250 millimetre centres. 150

radius inside. The spirals continued at 250 millimetre centres through the joint between the floor beams and the column. The six millimetre ties at 250 millimetres centres are not sufficient to provide ductile action in the columns and I have a diagram there showing these ties going through the beam column joint and they've been note effectively one six millimetre in diameter that is placed in the joint. The bent up bars from the pre-cast beams have limited anchorage in the joint zone. I don't believe they can be anchored fully in that zone because the anchorage length is too short and the distance between the end of the beam and the start of the hook is too short.

There is considerable congestion in the joint and it is difficult to see how the precast beam bottom reinforcement could be placed with a spiral in position as shown. The structural drawings number S14 show the spiral stopping under each beam and then starting again above the line at the bottom of the beam. There is no note in the drawings to lap the spiral bars as would be required to provide continuity of action.

NZS3101 part 1: 1982. Code of practice for the design of concrete structures and the commentary NZS3101 part 2 set out standards for the design and detailing of concrete structures.

(a) On page 15 under 1 General:

"1.1 Scope. This New Zealand standard code of practice specifies minimum requirements for the design of reinforced and pre-stressed concrete structures". The emphasis has been added.

This means that the designer needs to realise that there well may be extra design actions and forces to provide strength for if they see fit. It is not to be taken as the code that it specifies the maximum design actions that a building is designed for.

(b) The principles and requirements additional to 3.3.4 for the analysis and design of structures subjected to seismic loading.

"3.5.1.4:

The interaction of all structural and non-structural elements which, due to seismic displacements, may affect the response of the structure or *the performance of non-structural of elements shall be considered in the design of that structure.*

"3.5.1.5:

Consequences of failure of elements that are not part of the intended primary system for resisting seismic forces shall also be considered".

Clause 3.5.1.5 applies to the CTV building and is a warning that the internal columns shall be considered. The columns of the CTV building were small and heavily loaded. They were not detailed for ductility and as a consequence they would fail as subject to significant reversal cyclical moments such as occur during an earthquake. The consequences outlined in this clause do not appear to have been heeded. The central columns were also heavily loaded. I repeated myself there.

NZ4203 gives a various load cases so the structures is to be designed for on page 17.

Clause 1.3.2.3 the design load U for the strength method is U equals 1.4D x 1.7 LR.

Where D is the dead load, ie the self weight of the building,

LR is the reduced life load.

The live load for offices is 2.5 KPA from table 2 page 25. This can be reduced where the tributary area exceeds 20 square metres by R equals .3 times 4.5 over the root of B.

JUSTICE COOPER:

Q. Just, in those two equations that you have read out, there is a plus sign in there which you have read as times, is the plus right or should it be times?

A. Yes.

Q. So the first one U equals 1.4 D plus 1.7?

A. Yes plus 1.7.

Q. And then the bottom one, R equals 0.3 plus 4.6?

A. Yes that is correct, yes.

Q. You were reading times for some reason. Okay, thank you.

A. Yes I believe that is true.

EXAMINATION CONTINUES: MR MILLS

WITNESS CONTINUES READING BRIEF OF EVIDENCE

A. When I calculated the loads specified by the code for these columns to be designed for dead load and reduced live load, a value was obtained that was at the limit of their capacity.

The columns were fully stressed in axial load according to my calculations, allowing for a small superimposed dead load on each floor plus ceiling weights.

The graph below illustrates performance of highly loaded columns when subjected to rotations such as what occurred during a seismic event. The curve of P = 0.4 fca Ag reaches its load capacity then fails soon after with very little extra curvature. This diagram is taken from the NZSEE publication: "Assessment and improvement of the structural performance of buildings in earthquakes" report June 2006.

Page G41A of the CTV building calculations calculates the minimum size of stirrups in spacing required in accordance with NZS3101 clauses 5.3.29.2 and 6.4.7 and notes spirals six millilitres diameter at 250 millimetres centres. The designer then computes the hoop reinforcement required by the code assuming the columns develop plastic hinges taken, from taking part in seismic action and concludes R10 of 100 millilitre centres or R6 at 40 millimetres centres. The designer then notes on the calculations: "these do not apply as columns are non-seismic".

Q. Just pause there a moment to assist the Commission so they probably found that those words are at the bottom of that reproduced page of the calculations. You will see there these do not apply as columns are non-seismic. All right you are on paragraph 27 on secondary elements.

WITNESS CONTINUES READING BRIEF OF EVIDENCE FROM PARAGRAPH 27

A. Secondary structural elements. Secondary elements.

Page 26 of NZS3101 states:

"3.5.14 secondary structural elements:

Secondary elements are those that do not form part of the primary seismic force resisting system, or assumed not to form such a part and are therefore not necessary for the survival of the building as a whole under seismically induced lateral loading, but which are subjected to loads due to acceleration transmitted to them or due to deformations of the structure as a whole".

The columns of this building were classified as group 2 secondary elements. They are not detailed for separation and they are therefore subjected to both inertia loadings as for group 1 and to loadings induced by the deformation of the primary structure.

NZS3101 3.5.14.3:

"Group 2 elements shall be detailed to allow for ductile behaviour in accordance with the assumptions made in the analysis".

The question is, can the frames in the CTV building in lines 2, 3 and 4, the east-west direction, be assumed to be secondary elements by virtue that there are stiff shear walls running in the same direction that will protect these frames from any excessive deflections? Are deflections under earthquake attack small enough for frames to retain their integrity carry the floor loads as elastically deformed columns or are there deflections such that the columns and beams in these frames were stressed to pass the normal elastic limits. If they go into the post-elastic mode they are required to exhibit plastic deformations and therefore ductility will be demanded of them.

The same question applies to the frame on line F, which runs in the north-south direction, except these frames are even more likely to fall into the category mentioned the commentary to NZS 3101 (C3.5.14.1) as the shear wall in the north-south direction is more slender. Then a quote:

"Caution must however be exercised in assumptions made as to the significance of participation. Frames in parallel with slender shear walls shall be designed and detailed as fully participating primary members."

JUSTICE COOPER:

Q. Can I, you may go on to explain this, but I'm not understanding the shear wall in the north-south direction is more slender? What are you referring, what part of the structure are you referring to there?
A. If you look at the next page I have drawn a picture there.
Q. Yes.
A. Of this wall, what I'm referring to there in the, if an earthquake attacks in a north-south direction –
Q. Yes.
A. – the resistance is provided by the return walls on the north shear wall.
Q. Right so there's –
A. (inaudible 10:02:44)
Q. So why are you saying the shear wall. Is that, yes (inaudible 10:03:00)
A. It should be plural.
Q. It should be walls?
A. It should be walls, yes.
Q. Right, okay. Thank you.
A. And there's one correction there, in my notes underneath that picture I've shown there, I'm saying, "Note the notice in the base of the wall," it should be, "The notch in the base of the wall."
Q. Well just going back to the paragraph 30, that should read, "As the shear walls in the north-south direction are more slender," is that right?
A. Yes. Shear walls, correct, that is exactly right, the shear walls in the north-south direction are more slender.
Q. Thank you.

WITNESS CONTINUES READING BRIEF OF EVIDENCE FROM PARAGRAPH 31

A. "The reduced wall section of the north shear wall shown in diagram 4 between levels 1 and 2 will mean that the resulting rotation of the wall will be increased. The moment of inertia is under half for the wall at this level compared to the case if the notch was not present. I would consider this to be a slender shear wall in this direction and C3.5.14.1

would apply. I should amend that to be slender shear walls in this direction.

The frames in the building are elements of group 2 and NZS3101 clause 3.5.14.3 further mentions what are sometimes confusing tests especially in light of the statement in paragraphs 29:

"3.5.14.3(a) Additional seismic requirements of this code need not be satisfied when the design loadings are derived from the imposed deformations V-Delta, specified at NZS4203 and the assumptions of elastic behaviour".

My interpretation of this clause is that if the member is checked for its ability to accept deflections derived from V-Delta and the member is still within the elastic range, then there is no need for ductile behaviour to be provided for that element.

The results of modal analysis completed by Compusoft using the factors in 1984 loading code NZS4203 have been taken and the deflections of the core wall applied to the frame. The resulting moments introduced in the frame on line F and line 2 have been combined with the axial load in the columns. The elastic behaviour of the columns was exceeded and they would have possibly failed because of their lack of ability to sustain plastic action. My conclusion is that the frame did not comply with this requirement of the code and clause 3.5.14.3(b) would apply which states:

"3.5.14.3(b) Additional seismic requirements of this code shall be met when plastic behaviour is assumed at levels of deformation below V-Delta".

The columns should have been detailed for ductility.

Q. Now just pause a moment just so there's no uncertainty about the reference to the Compusoft modal analysis because in fact we've had two of them done?
A. Yes.
Q. I take it the reference you've got here is to the one that they did as part of the DBH report that was done by –
A. It is.

Q. – Hyland and Smith?
A. That one there, yes.
Q. Thank you. You were at paragraph 35.
A. I should further add that they did send me some results that they had done of those frames under that loading so I checked it all so as well from those results as well as the DBH report.
Q. Are those results derived from the more recent analysis that was done at the direction of the Royal Commission do you know?
A. The initial ones weren't but I reported it here.
Q. Yes.
A. Because they hadn't been done then but I have examined the more recent ones but these ones, they were done with a stiff base.
Q. Yes.
A. And I specially requested them to put a, sorry a fixed base –
Q. Yes.
A. – underneath it because I believe that’s what the code required in 1984.
Q. Yes, so nothing in what you've seen in the more recent data from Compusoft has changed any of this evidence you're giving?
A. No it hasn't changed the evidence, no.
Q. Paragraph 35.
A. 3.5.14.3(c) sets out the inertia loading requirements from NZS4203. 3.5.14.3(d) cautions that the secondary member may be subjected to more complex deflection and consequently loads in some localised areas.
3.5.14.3(e) appears to set a lower bound limit on the elastic behaviour of a secondary item of one quarter of the primary elements. This is to provide a minimum strength of secondary unit and points out that a secondary unit element that responds elastically to the total deflection may be too strong for the structure as a whole.
Despite the clause of 3.5.14.3(a–f) being difficult to interpret the overall lack of detailing for ductility of the critically important columns of the CTV building does not comply with the instructions of the code. These columns were critical to the performance of the building. The

calculations from the computer are so dependent on assumptions of stiffness, material properties and the mathematical model formed, that it is not wise to rely on the results as an accurate representation of what will occur in the building under seismic loading, especially with such an important element as all the columns in the building. These columns hold up the complete floor plate. I do not consider them to be secondary elements.

Principles and requirements additional to 3.3 for the analysis and design of structures subject to the seismic loading.

"3.5.1.6 Consequences of failure of elements that are not part of the intended primary system for seismic resisting forces shall also be considered".

This clause applies to the internal and external columns in the frames of the CTV building. They carry the major part of the weight of the building and are critical to its survival. The internal column and beam frames will take part in a seismic movement of the building. They are all connected by the floor acting as a diaphragm.

(a) Diaphragm action:

The floors of the building act as large in-plane ties and struts connecting all the various parts together when an earthquake occurs. They are designed to connect the critical elements such as shear walls to the rest of the building, sited away from the walls. The floor system in the CTV building was constructed of metal deck form work with a cast in situ 200 millimetre thick slab poured. The metal deck is ribbed to give an average thickness of 1.75 millimetres.

This is a heavier slab than normally is expected on a building with 7.5 metre slab– a 7.5 metre span. The reinforcing is principally 664 mesh. The area of this mesh is 185 millimetres squared per metre length, results in an under reinforced floor slab.

NZS 3101 10.5.6.2:

"Diaphragms shall be reinforced in both ways with not less than minimum reinforcement required for two way slabs in accordance with 5.3.32".

Q. Now it's just a minor point but you read directions as in both ways. I take it you meant to read in both directions?

A. Yes I'm sorry I did.

Q. That's all right, thank you.

A. Clause 5.3.32 outlines the minimum reinforcement for the various types of reinforcement. For mesh the following is given:

5.3.2 Shrinkage and temperature reinforcement specified that slabs where bars with fy (that means a yield strength) = 434 MPa, or welded wire fabric, deformed or plain, are used, the value is 0.0018.

The welded wire mesh does not meet this requirement. In one direction the metal deck does provide some reinforcement but in the other it is a series of discrete units joined together by friction. The slab design was not covered by the concrete code existing at the time. The typical – I meant by that the metal deck slab, the typical procedure was to refer to the manufacturers design charts and use them to select appropriate span and thickness including top slab reinforcement at the supports.

The Hi-bond literature current in 1985 contained a load/span chart for single spans, with a maximum of 6.6 metre span for 200 millimetre slab thickness, superimposed load 2.2 kPa. The manual did have a statement to the effect that larger spans could be possible if span continuity was introduced along with negative reinforcement, but no guidelines on design capacities were given. The manual also indicated 664 mesh was appropriate for a 200 millimetre deep single span slab. However this is in contradiction to the code requirements. With changes in the design code the current literature specifies a maximum span of 6 metres for a continuous internal span and 6 metres for an end span.

Q. Now you read 6 metres twice, I think the first time you probably meant to read at the bottom of page 14, 7 metres?

A. Yes I did I'm sorry, the current literature specifies a maximum span of 7 metres for a continuous internal span and 6 metres for an end span.

JUSTICE COOPER:

Q. Sorry what was the last comment, it should be something?
A. Negative steel has H 12 bars at 150 millimetres centres, or as would be central to. I'm sorry.

EXAMINATION CONTINUES: MR MILLS

Q. You all right?
A. Yes. It is apparent that the original design was –
Q. Sorry I think His Honour is still wanting to get the clear, the change you made in that.
A. Well I'm just reading that again. I think it is correct as it is stated a negative steel has H 12 bars at 150 millimetre centres. I think that's correct.

JUSTICE COOPER:

Q. Has is right?
A. Yes.

EXAMINATION CONTINUES: MR MILLS

Q. Paragraph 42.
A. It is apparent that the original design was from first principles and not to Dimond literature at the time, and going by the current literature, the design is beyond the criteria for maximum Hibond span capability. The concrete code at this time did not address the design of Hibond slabs. The CTV building has an end span of 7.5 metres and negative steel of H 12 at 120 millimetre centres over the central support beam. This light reinforcement may have contributed to a weakness in the slabs' ability to transfer loads from the structure to the resisting shear walls by

diaphragm action. The slab would have been subject to bending stresses as the shear walls moved back and forward during a seismic motion. The H 12 bars in the slab terminated 0.8 to 1.5 metres approximately from the edge of the shear walls. There is a point of weakness in the slab at the line at which the abrupt termination of the top slab bars occurs and only 664 mesh is available for negative moments. The mesh has a cross-sectional area of 186 millimetres squared per linear metre.

(b) Connection to north shear wall

The connection to the north shear wall is limited. In the east-west direction the rear wall of the shear wall is 11.5 metres long and provides a shear capacity in this direction for the wall. However the connection to the rear wall is only by a slab approximately 3.75 metres wide and 4.5 metres deep. There is also a hole in this slab adjacent to the rear wall resulting in only a 2.35 metre slab connection directly to the wall. This slab has one layer of 664 mesh top throughout as reinforcing plus short starter bars from the return wall on line C and line CD. This is below the code requirement for steel in the slab.

In the north-south direction the two walls on line C and CD are connected by 19 D12 diameter bars in the slab. The two return walls, D and DE of the north wall do not appear to be connected to the main floor slab. The effect of this would be to induce further eccentric behaviour in the wall under north-south seismic action. I have been advised by counsel assisting that some attempt was made to connect these walls to the slab at a later date.

The Hyland Ashley Smith report suggests from the examination of the collapsed state of the building that the north shear core was not stressed into the plastic range as a result of the earthquake. Normally I would expect the wall to show signs of large plastic deformation for such seismic loading. I would infer that the wall was not stressed as expected because the wall was not loaded from the main weight of the building. Either the building had collapsed or the attachments to the core had been insufficient to transfer the seismic loads. And the diagram shows the colouring, the position of the slab in that core wall and the attachment to the rear on line 5.

Q. That diagram I take it is taken from the Hyland Smith report is it?

A. I think it's actually taken directly out of the design drawings of the original Council approved drawings and I had just coloured in the concrete so it's clear what is an opening and what is concrete.

Q. Thank you.

A. In chapter 7 of the concrete code NZS3101 part 1 1982, clause 7.3.4.3 gives the minimum requirements for shear steel:

"Where shear reinforcement is required by 7.3.4.1 or by analysis, minimum area of shear reinforcements for pre-stressed (except as provisions in 7.3.4.4) and non pre-stressed members shall be computed by

Av = 0.35bws/fy"

Q. Are you able to explain that in narrative terms that non-structural engineers would get some grasp of?

A. Yes I can attempt to. That the AV means the area of steel that you should provide in these columns to take horizontal shear, should be equal to .35 times the beam width and "S" is usually the depth of the beam from the effective steel to the compression area of the concrete. So it's really, it gives you a requirement of how you calculate your shear.

WITNESS CONTINUES READING BRIEF OF EVIDENCE

A. "This requirement applies whether the member is a primary seismic resisting element or not. The spiral reinforcing provided in the columns of the CTV building did not meet these requirements. The R6 spirals would have been provided at much closer centres had this minimum requirement been satisfied. The Hyland Smith Report calculates the spirals required R6 at 900mm centres.

JUSTICE COOPER:

Q. 90?

A. At 90mm centres, yes.

WITNESS CONTINUES READING BRIEF OF EVIDENCE

A. "This is 2.7 times as much air steel as that provided.

NZS3101 Part I 1982, Section 9, applies to the design of the beam column joints.

9.4.1 General:

"Provisions in this clause 9.4 apply to beam column joints where gravity actions govern. If the joint is also subjected to seismic reversals, it shall be checked for compliance with the provisions of 9.5".

I have already stated in my opinion the beam column joints in the CTV building were subject to seismic load reversals and my reasons for this conclusion. In my view clause 9.5 applied.

NZS3101 9.5 – Principles and requirements additional to 9.3 for joints designed for seismic loading.

Clause 9.5 outlines design requirements to protect the joints from failure.

"9.5.1: General. Special provisions are made in this section for beam column joints that are subjected to forces arising as a result of inelastic lateral displacements of ductile frames. Joints must be designed in such a way that the required energy dissipation occurs in potential plastic hinges of adjacent members and not in the joint core region".

The joints shown in my diagrams S.2 and 3 have not been designed to meet these requirements. The provisions of one 6mm diameter spiral does not provide the shear resistance needed to transfer the internal forces generated in a beam column joint.

The CTV building design did not comply with the code or the intent of the code in respect of the following critical structural elements:

The building was not designed to be symmetrical despite several instructions in the code to design buildings with symmetrical resisting elements. Although there is no absolute criteria specified in the code the instruction is clear in NZS4203 1984 3.1 and in the Commentary on this clause 3.11.

"For high buildings symmetry is one of the most basic requirements in achieving a structure of predictable performance".

Buildings designed with seismic resisting elements placed to provide a symmetrical resistance to earthquake loads have been found to suffer less damage than asymmetrical buildings. Hence the instruction in the code.

The columns internal and external to the buildings were not designed for ductile behaviour under earthquake loadings. This is despite several clauses in the code which specify that they should have been designed for ductility.

The designers may have assumed that they could predict accurately that the columns would be subject to a certain amount of reverse moments from an earthquake and they somehow would not be stressed for that little bit extra that would cause complete catastrophic collapse. This is despite several cautions from the code pointing out that the limitations of assumptions for material properties in theoretical analysis.

The small diameter columns were heavily loaded and this made them further unable to resist, to accept post elastic deformations without failure.

The minimum shear steel required by the code was not provided in the columns.

There was limited connection to the major shear wall on the north side of the building situated outside the main floor plate of the buildings."

Q. Now Dr Jacobs I understood there were some points you might have wanted to add to that brief. Are there?

A. Well I could emphasise that I thought that the shear walls and the slender ones in the north/south direction on, that form the returns are slender. They have an aspect ratio that's over five to one and the effect of that notch I have found in my engineering experience does cause more a rotation than a computer analysis would show but the stresses when that wall is bending outwards away from the building have to flow back round the notch and then back out again to the wider part of the wall and that can often cause a concentration of cracking I have found in that sort of part of the wall.

Q. Any other points you want to make?

A. I think that I could probably emphasise again probably the limitations of a computer model to represent an actual building in practice. I think that a building such as this built on the site is quite hard to model. For instance the diaphragm actions in the computer model in 1984 would have been taken as rigid diaphragms. The diaphragm in reality is not rigid, especially where this one connects to the north core wall. The main resistance is the wall on line 5 which is 11.65 metres, I think, long and that is very hard to connect to being outside really the building line to try and connect that back to the middle of the building is a very difficult process. Now the computer analysis, in my opinion, would assume that that's rigidly connected to that floor plate when in fact it probably was not.

Q. So is it your view that if one was using a computer model there would then be a need for some judgement to be exercised?

A. There certainly would be a need for judgement in that and I would look back to my experience that at that time in designing buildings where I think that, I did not rely on one computer analysis as being the absolute design basis for the building that I would approach. Usually the way a building like this is designed you do what was called a preliminary analysis. You work with the architect and you design what is going to become your resisting elements in that building. First of all you do the gravity loads, the columns and beams, and then you look for your seismic resisting elements and you do what is called a hand analysis. In other words you don’t use your computer for a start usually because you haven’t got time and also because you want to get a good feel. A practising engineer with experience wants to get a feel for the capacity of the seismic resisting elements that take the loads that he would have worked out from the code. So that gives you a good feel for where the loads are going to go into the building. Then I would typically do a two-dimensional analysis on that building looking at each wall in turn, estimating the loads that wall was going to take and that would help me further understand how the building was going to behave. And then finally I would do three-dimensional analysis, a modal analysis, to give a more accurate picture and I would have been checking that analysis all the time to make sure that there was correlation between the three types of analyses.

MR MILLS ADDRESSES THE COMMISSION

Yes, yes, thank you for that. Commissioner do I sense you have something you'd like to..., just by all means break in if there are questions.
Now thank you Dr Jacobs so that concludes the principal brief. Now he's got a reply brief and because it is a reply brief it's potentially somewhat disconnected not having heard from Dr O'Leary so there's really two ways of dealing with it and I invite some guidance from the Commission. We could hear from Dr O'Leary first and then have Dr Jacobs give his reply brief or I could invite you to have Dr O'Leary's brief open in front of you to which Dr Jacobs is replying and in that way I think you would be able to understand.

1030

JUSTICE COOPER:

No, we'd prefer to hear Mr O'Leary and then Dr Jacobs.

MR MILLS:

In that case if you could just stay there and answer any questions and then I'll call you back again later for your reply brief. I'm aware you've got a time limitation but I will work within that.

JUSTICE COOPER:

Q. What is the time limitation?

A. 4 o'clock.

MR MILLS:

4 o'clock so we'll be fine.

CROSS-EXAMINATION: MR ALLAN – NIL

CROSS-EXAMINATION: MR CLAY – NIL

CROSS-EXAMINATION: MR REID

Q. Dr Jacobs, I am counsel for the Christchurch City Council.
A. Yes.
Q. And I'll shortly be calling Dr O'Leary.
A. Yes.
Q. I was going to put some passages to you from Dr O'Leary's evidence but given the way that the Commission's indicated it will be proceeding I won't do that at this stage. So I just have some other questions for you.

THE COMMISSION ADDRESSES COUNSEL – NOISE OUTSIDE

CROSS-EXAMINATION CONTINUES: MR REID

Q. Dr Jacobs I just want to try and understand a bit more, particularly the relationship as you describe it between the, between 4203 and 3101, the two standards?
A. Yes.
Q. Now do I take it from your evidence that if the columns stay elastic at V delta they would not be required to be detailed for ductility?
A. From my evidence I would say that the clause, I think it's 3.5.14 in 3101 does say that, but it also, there are other clauses in 4203 which caution against using that clause. They say that if the elements, I'm quoting here by memory, are critical to the performance of the building then they should be detailed for ductility. So before I would accept that clause as

being the only criteria to design those columns I would look at those other clauses. Also in 3101 Part I. I think in the commentary close to 3514 is it 14.

1035

Q. This is just to assist us, is this ENG.STA.00116A.32, if that could be brought up. Dr Jacobs just on the screen in front of you on the small screen right in front of you.

A. Yes if you look at clause –

Q. Yes the small computer screen in front of you Dr Jacobs.

A. Yes.

Q. Yes, that's the document that I've asked to be brought up.

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 and includes such primary gravity load resisting elements as frames which are in parallel with stiff shear walls and do not therefore participate greatly in resistance to earthquake, to lateral loads", but it goes on to give a caution, it says "caution must however be exercised in assumptions made as to the significance of participation". I think that's a very important caution. "Frames in parallel with slender shear walls should be designed and detailed as fully participating primary members". And I believe the frames in parallel with this – the shear walls D in the north-south direction, do have a slender shear wall supporting them and they should be designed as primary elements. That's specifically what that sentence says. It just goes on to say "for convenience of reference and for specification requirements secondary elements have been subdivided into groups, that is group 1 and group 2 element". So I would take that commentary and say be very cautious before you don't detail your columns for ductility.

Q. Yes, so with that caution, if that caution accepted, if the columns in the CTV building stay elastic at V delta, isn't it true that they would be able, under the code to not be designed for ductility?

A. Under the clause I think that would be correct, yes. I'm not saying that I would personally do that, if I was designing a building like that but I think that clause does a bracket, the clause does say that yes.
Q. Yes and so that's an important distinction isn't it, because you're drawing a distinction between what you might have done and what it might be prudent to do and what the code allows. That correct?
A. Yes I suppose that's correct, yes.
Q. And that proposition that arises out of this clause 3.5.14(1) and its commentary, the proposition that columns may need not be designed for ductility if they stay elastic at V delta, that applies regardless of the directions in 4203 regarding ductility. Isn't that correct?
A. I don't know the exact legal requirements that you have, but if one code says you should do something and the other code says it's an out then I guess, I suppose it is correct because 3101 does apply specifically to concrete.
Q. Yes, yes, so just so that we're clear that's your evidence about how the interrelation between the two codes works, is it, that as we've been discussing columns stay elastic at V delta therefore no ductility required and that's true regardless of the direction in 4203?
A. Yes that is correct.

JUSTICE COOPER ADDRESSES MR REID

This ultimately I think is going to be resolved as a question of law. Normally expert witnesses are here to give evidence about matters to do with their expertise such as matters of structural engineering. In fact there's only one exception when an expert can be called to give evidence about matters of law as I understand it and that is about foreign law though sometimes some of these structural issues may appear to be relating to matters in a foreign language, not I'm sure the right approach so I think it's inevitable that counsel will end up putting questions of that nature to witnesses but unless I'm persuaded to the contrary I think these are matters that will need to be the subject of legal submission.

MR REID:

Yes Sir and I –

JUSTICE COOPER:

And as I say I'm capable of being persuaded to the contrary.

MR REID:

Well Sir I accept that and the reason for my questions is because they're really germane to the matters that he put in the evidence.

JUSTICE COOPER:

Well as I say I'm not going to stop questions of this kind but I just thought I should lay out the way I see it and also the converse of that approach is that I wouldn't see that in the deliberation phase the Royal Commission would say, oh well such and such a proposition of law was not put to a witness, therefore it hasn't been raised. I think all these issues must be matters which are important addressed by counsel in their submissions.

MR REID:

Yes I accept that Sir. I can frame the question slightly differently.

JUSTICE COOPER:

I'm not criticising it I just thought I'd take that first opportunity to put a flag in the sand.

MR REID:

Thank you Sir.

CROSS-EXAMINATION CONTINUES: MR REID

Q. Just moving on from that Dr Jacobs, we've heard evidence from Mr Henry who is a practising engineer in Christchurch and was in the 1980s, about a method of design which he describes as a shear wall protected gravity load system, and are you familiar with that description of a design method?

A. I did hear him mention that in when I was just looking at one of the videos of his statements, yes.

Q. And is that a design method that you're familiar with?

A. Well you say it's a design method. I see it more as a, I don’t think it’s a design method, it's with using this clause here as a means of not putting reinforcing steel in a column and I guess you could call that a design method.

Q. Yes, well that was what I was going to ask, is that approach, it's based is it on clause 3.5.14?

A. Yes I would expect it was, yes.

Q. Now on the basis that as you've said, 3.5.14(1) governs the requirement for ductility or not, in the CTV building, the issue comes down as I understand your evidence to the Compusoft analysis as to whether or not –

A. It does.

Q. Yes.

A. Now I do, talking about Mr Henry’s evidence. I understand the building he was talking about had a – quite a stiff shear wall in it which was changed to be a tube rather than a C shape wall. Is that correct, from my memory?

Q. I think –

JUSTICE COOPER:

Broadly speaking that's correct, yes.

1045

CROSS-EXAMINATION CONTINUES: MR REID

A. Yes, so he, he satisfied himself, I can't remember the name of the building now, that he was dealing with a building which had a very competent set of shear walls in it that gave him a lot of assurance that he was not going to get into this problem with V-delta, so to me just looking at it philosophically, what do you do if you design your building and you do your analysis, whatever that might be, and you believe in it, and that says I've got 98% of the deflection to break that rule so I'm not going to put any reinforcing to resist earthquakes in it. But then by taking a slightly different value for your stiffness or something like that you get 101%. So you then suddenly go from having no seismic resistance to those columns to having seismic resistance so I see that as a weakness in that approach.

Q. Yes, and that's the reason I take it you would say or the direction as to caution?

A. That's right, that's why they had the caution here and there's other statements, I think it's in the 4203, about your analysis which I mentioned in my report there that the analysis, it must be an approximation because the materials that you are trying to represent in your computer analysis are at best an approximation.

Q. Yes, yes, so just going back to –

JUSTICE COOPER:

Can I just ask a question if I may? I'm interrupting your flow am I?

MR REID:

No Sir, that is no problem.

JUSTICE COOPER:

Q. In this provision of NZS3101 which is still displayed, commentary 3.5.14.1 it says the definition of a secondary element is more particular than that in NZS4203. Where is the definition of secondary element in NZS4203?

A. I can answer that if you'd like.

Q. Yes.

A. It's in the front part of it and if you look in 4203 there is on page 13 –

Q. Page 13?
A. – in the front it has a definitions clause 1.1.3. On 13, page 13?
Q. Yes.
A. Secondary elements.
Q. Right, I see that.
A. Yeah.
Q. 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.
A. And it mentions such things as partition walls, panels and veneers, not necessary for the surviving of –
Q. It's just that the requirement for ductility in clause 3.2.1 of NZS4203 does not seem to utilise the expression secondary elements, is that right? It doesn't turn on that at all.

MR REID:

I think that is correct Sir. It seems to be an over-riding requirement which takes no account of whether the element concerned is primary or secondary.

JUSTICE COOPER:

And if there is an element that might fail and pose a risk to life in doing so, going back the other way that might in itself not be so easy to fit into the expression secondary element, such as partition walls, panels or veneers?

MR REID:

Well, yes, that is true Sir in terms of the definition of secondary element and –

JUSTICE COOPER:

Thank you anyway, I was just looking for that because it seems to me the clause in the commentary is talking about secondary element and the requirements of ductility doesn't turn on that language at all.

MR REID:

No.

CROSS-EXAMINATION CONTINUES: MR REID

Q. So just go back to Mr Henry and his description of his design approach as shear wall protected gravity load system. I think we got on to talking about a building that he'd been involved in which I think was Landsborough House but his comments about that design approach were more general. He was talking about a generalised system at paragraph 27 and 28 of his evidence and you're familiar, in general terms, with that approach to design?
A. Yes, although I have not read his evidence. I have not read that.
Q. No. Well was that a common approach to design in New Zealand in the 1980s?
A. Well not that I was aware of, in my practice it wasn't. Certainly was not. I remember, I do remember looking at this issue and together with the issue is whether you should model the frames in a building which had shear walls in it.
Q. Yes.
A. Should you neglect them or should you put them in, and I sort of thought about this quite a lot. In the end I used to put them in to my analyses because I thought they, who said they shouldn't be part of it. They're going to take part in it and they may have a small percentage of the load but to me they were part of the resisting element of the building. I also thought at that time really, I think an engineer has to take an overall view of his structure and has to be happy with what he's designing and I could not bring myself to design a column without a small amount of extra stirrups at each end to give it security that it wouldn't fail in an earthquake and extra costs I believe was very small in doing that if you were not. The overall costs of these buildings is it usually made up of the land value, the total cost of the building and the holding costs from the interest rates on the building. Now the costs of a little bit of extra steel is very, very small –
Q. Yes.

A. – so an engineering judgment has to be made because an engineer's responsible for this building for its lifespan of at least 50 years and maybe more so I felt that it was a basic engineering principle that why would I not put in a little bit of extra steel in the top and bottom.

Q. Yes. Do you accept thought that that's a slightly different issue from whether or not the building strictly complies with the code?

A. Well it's subject to different interpretations of the code, I agree with that, but the code does say quite strongly that you should any element that's to, is dangerous to the life of the building you should detail for ductility so –

Q. Yes. Mr Henry also says and this is just in relation to your comment about whether the frames should be included in a computer analysis.

A. Yes.

Q. Mr Henry says and this is at, perhaps it can be brought up, WIT.HENRY.0002.4. This is in relation to his commentary on or this is his proposed evidence on Mr Latham's work. Just wait for a moment while it's brought up. Paragraph 9, he says that, he notes that both David Harding, the original designer, and Hyland and Smith had carried out their analysis excluding secondary elements and he regarded the frames as being secondary elements and he says that excluding these secondary elements from the analysis was normal practice in the 1980s.

A. I can accept that it may have been fairly normal practice, yes, but I was just saying that I can remember thinking, it is very hard for me to remember exactly what happened in the 1980s, but I can remember thinking about this problem quite a lot on a number of buildings I did include the secondary elements in them.

1055

Q. But if it was normal practice to exclude them, that would suggest wouldn't it that this approach to design based on 3.5.14 was widespread. Do you accept that?

A. No I don't accept that. I don't know enough for a start about the widespread practices in those times. I haven't studied them but I wouldn't accept that it was widespread not to detail columns that say sat in a building with shear walls, not to be detailed for ductility. I don't know that that, no I'm not aware of that. Certainly didn't seem to be in Auckland that I was aware of. I mean I've never seen a column like the columns in the CTV with such little reinforcing in my experience and I've had considerable experience and I've just never seen a column like that. With that number 6 diameter bars they don't look like they should be there.

CROSS-EXAMINATION: MR RENNIE

Q. Now Dr Jacobs if you turn to the start of your brief. At paragraph 5 you set out the question that you were asked to answer in your brief and that was put on the basis of whether the building complied with Christchurch City Bylaw number 105. Do you see that?

A. Yes I do see that.

Q. Would you accept that your brief is actually directed to the question of whether the building complied with 4203 and 3101 rather than with the Bylaw itself?

A. Yes it probably is, yes. Well I understood at the time that was the effect of that Bylaw.

Q. It's just that for example and His Honour has referred to the matter of law being a matter for submissions, Bylaw 8.4 which relates to compliance in relation to matters of concrete design provided that concrete elements designed in accordance with the requirements of NZS3101 or recognised equivalent standard shall be deemed to comply with the requirements of this Bylaw. Did you note that when preparing your brief?

A. I did not note that specific clause, no.

Q. So that I'm not asking you for the answer but just pointing to you the tension for a designer in 1986 designing concrete elements but on its face if they meet 3101 they've met the Bylaw although on your analysis they may be in difficulty under 4203.

A. I certainly think they would be in difficulty under 4203.

Q. Now is that a factor in compliance that stands outside from your evidence and is not a matter on which you are putting forward a particular finding?
A. If I understand your question, that is correct.
Q. And then in relation to the summary which you present in your brief at the conclusion of the brief which you have read and I appreciate you have a reply brief yet to come, paragraph 53 you say, “The CTV building design did not comply with the code or the intent of the code.” Do you see that?
A. Yes.
Q. So, and this is not a criticism, I’m simply trying to clarify the significance of the evidence you’ve given. What you’ve put forward is essentially your interpretation of the code and the intent of the code in expressing your opinions?
A. Yes it is but I believe it is mainly the code.
Q. And the source of the intent of the code. Do you get that from the commentary in the code or do you get it from other sources as well?
A. I mainly get it from the commentary in the code.
Q. Now am I right in thinking that an engineer designing in 1986 would be guided by both the code and the commentary?
A. I would expect so, yes.
Q. And in fact on occasion for example the way in which moderate eccentricity is to be interpreted you find the method for that in the commentary rather than in the code don’t you?
A. I was aware there was, yes.
Q. So can you see an additional dilemma for a designer in 1986, namely that the Bylaw which I’ve read to you and there’s a similar one in Part XI for the general construction, required compliance with the code but your evidence is directed to compliance with the intent as defined by both the code and the commentary?

JUSTICE COOPER:

Q. He may not understand that question.

A. I don't know whether precisely I'm answering two things here at once.

CROSS-EXAMINATION CONTINUES: MR RENNIE

Q. I'll try and put it a little more clearly. What I'm putting to you Dr Jacobs is that a designer in 1986 complying with the code simpliciter, by itself, maybe open to criticism by you because the designer has not paid attention to the commentary when trying to achieve that compliance?

A. I would not say he's open to criticism. I think that if you look at 1986 the code was not as defined and is, shall we say, specific in all aspects as the code is now. It had been developing so a designer at that stage one of his main aims was to design a building for earthquakes. He would have looked at the letter of the law of the code. He'd also when things weren't that clear to him, the code perhaps wasn't written so that every word and everything was specific. He needed to use his judgement and the judgement was influenced by the commentary. So I think that the commentary is a very valid support for the letter of the law of the code.

Q. Please understand I'm not offering a criticism, I'm trying to identify the difference between the legalistic approach, complying with the code, and your approach, as I understand it, which is to look at practice and intent as well as the code wording and express an opinion as to how the compliance should have been assessed?

A. Yes I did use the commentary as a help in assessing it.

Q. You drew our attention in your evidence to a couple of passages in the Foreword to the Code at page 9 and –

A. Is this 4203 now?

Q. This is 4203 that's correct, and the second of those was a paragraph beginning, "Designers should recognise the precise properties of construction materials." Do you recall that?

A. Yes I do.

Q. I'm just going to read you the last part of that paragraph which you left out of your quote and then ask you a couple of questions about it because the paragraph ends as follows, "In fact the design result depends so much on the nature of the mathematical model of the building as envisaged by the designer but the use of more advanced techniques of earthquake analysis can easily lose validity." Do you see that?

A. No I can't see that but I can understand it. What page is that?

Q. It's page 9 of the 4203 code. I'd mistakenly thought you'd got to it. It's halfway down that page and it's the second part of the paragraph beginning, "Designers...." It's just the last sentence in that.

A. Yes I read that.

Q. So the caution that's been given there is that in the state that the modelling by mathematical means the buildings had then reached, the outcomes could in some cases not be valid outcomes. That's the caution isn't it?

A. That is what the caution says, yes.

1105

Q. If we come back to page 8, the second paragraph in the Foreword, the Committee said, "The loading committee's task in draughting this standard was seen mainly to be one of providing a set of minimum design criteria of an effective and economic nature which would not be too difficult for the designer to apply but at the same time would leave him, as they said in those days, "scope for innovation and imagination". Do you see that?

A. Yes I do.

Q. Yes, and again what we're looking at there is paralleled on page 9 which we've looked at, the limitations which then existed as to the ability in the code to prescribe the weight of C?

A. Yes it does. Well your limitations of sophisticated computer analysis I think is all C.

Q. Yes that's the point I'm coming to so that, and your point a couple of questions back was that there have been substantial advances since then such that it is now possible to have a code which is more prescriptive as to what has to be met?

A. Yes it's probably more defined, yes I agree.

Q. Yes. Now you go on in your brief of evidence at paragraph 11 to discuss symmetry?
A. Yes.
Q. And that is, as you may or may not know, a requirement which appears not only in the standard but which is quoted verbatim in part 11 of the Council's bylaw at 11.2.5.1. So the Council clearly attached the importance to this in design terms?
A. Yes I could accept that.
Q. Now the definition of symmetry therefore in both the code and expressly in the bylaw was, "The main elements of a building that resists seismic forces shall, as nearly as is practicable be located symmetrically about the centre of mass of the building". That's exactly it isn't it?
A. (no audible answer 11:07:50)
Q. Now given that that's taken from the code would you agree that we can apply the term "elements" from the code, the definition you've already pointed to, in understanding what is to be assessed in symmetry. We can use that meaning?
A. I would expect so, yes. This is in 4203 we're talking about now.
Q. Yes this is 4203. And "elements" in 4203 is a definition at page 13 which includes both primary and secondary elements. In the symmetry provision that we're looking at, that has two extra elements attached to it. The main element is the first and second that resist seismic forces?
A. Where is that, sorry? That's in page 13?
Q. Symmetry in the code appears at –
A. I've got that on page 33.
Q. Yes?
A. But what, that last statement of yours. I didn't quite follow that?
Q. It's taken directly from the definition. We're looking at elements which are main elements and we're looking at elements that resist seismic forces. That's the two requirements of an element?
A. Does it specifically mention main elements? Is that on page 13?
Q. No, I'm sorry, I'm not trying to confuse you but I'm doing quite well. I'm referring now to the definition of symmetry as it appears in 4203?

A. Oh, it's the symmetry definition, yeah I thought we were still on elements.
Q. Sorry if I confused you on that.
A. I can't find it on that.

JUSTICE COOPER:

Page 33, clause 3.11.

1110

CROSS-EXAMINATION CONTINUES: MR RENNIE

A. Clause 3.11.
Q. Top right-hand corner of the page?
A. Yes I see that there is the symmetry in 3.11, yes.
Q. So that –
A. The main elements of the building – I just struggling to find what the point was about elements. In elements on the definition it just says includes primary and secondary elements doesn't it?
Q. Correct. So when we are working out whether a building is symmetrical we need to identify what an element is. We know that. We now need to work out what a main element is and it also has to be an element that resists seismic forces. We have those to consider?
A. Yes I accept that, the main element or do you mean a primary element by that or a main element?
Q. Much as I would like to help you Dr Jacobs I doubt that my opinion is worth as much as yours?
A. It just the definition because some of these things are quite defined, you know. Primary is defined but main, I haven't come across that –
Q. I am not aware, sorry –
A. – as an engineering definition -
Q. – I understand your point –
A. – I can understand what it is.
Q. I am not aware of any guidance as to what –
A. Okay.

Q. – main means?

A. Fine.

Q. When we are coming to assess the symmetry of any building we are looking to work out what the elements are so we can decide the extent to which they have been symmetrically arranged, don't we?

A. Yes, I agree.

Q. And would you agree that we now encounter this difficulty that if the columns are in fact a primary element in this building, that is to say they resist seismic forces, then they have to be considered as well as the shear walls?

A. Just repeat the question, are you saying that –

Q. If the columns in this building are primary elements that resist seismic forces?

A. Yes.

Q. Then they will have to be considered as well as the shear walls when assessing symmetry?

A. If you consider they are primary I would say so yes.

Q. And I am putting to you therefore which way you treat the columns affects how you assess the symmetry?

A. It could affect, yes, in some (inaudible 11:12:30).

Q. Because my understanding is that you expressed the view that this building was not symmetrical?

A. I did yes.

Q. But if I have it right you expressed that view based on the position of the shear walls in relation to the centre of mass?

A. Yes I did.

Q. Have you considered whether the building should be seen as symmetrical if the columns were also considered?

A. I haven't spent a lot of time looking at that but I would consider by inspection that because they are very flexible and they would only take part in the small amount of the stiffness of the building that there would not be a great variation of the centre of rigidity if those frames were taken into account.

Q. Well we are really looking at the centre of mass aren't we rather than the centre of rigidity, in the definition?
A. I mean okay, yes sure.
Q. So that if we now look at this issue of symmetry in relation to this building in those two ways, the first relates to the positioning of the shear walls?
A. Yes.
Q. Taken by themselves in relation to the centre of mass it is your opinion that the building is not symmetrical?
A. It is my opinion, yes.
Q. Do you accept that others have and could validly reach a different opinion?
A. I accept that others have reached a different opinion. The reason why it, I thought it was very asymmetrical is the stiffness of the wall has influence on whether it is symmetrical or not and the stiffness of a wall is proportional to the fourth moment of its area, so if you take the depth of the north wall, it is 11.65 metres, if you take that to the power of four, you get somewhere exceeding 18,000. If you look at the southern shear wall which is roughly three metres, no five metres overall, you take that to the power of four, you get 625. So you are comparing 18,000 relative to 645, or 25, so that to me is quite a marked different in stiffness so that is why I would consider that, it is not a balanced design.

COMMISSIONER FENWICK:

Sorry I didn't mean to interrupt, can I just intervene now. I think there are two factors which are slightly being confused here. I think probably between the engineers we know what we are referring to, but there is symmetry and there is eccentricity and you really need to consider both of them, and so considering one without looking at the other can I think be slightly misleading so if you excuse me just on that. The values that Dr Jacobs was just recording, my calculations just to save you trying to do the sums, but the stiffness to the north wall and the east-west erection compared to the south wall is about 25 to one.

MR RENNIE:

Sir I acknowledge that and indeed I may be going a little too pedestrian but, because I was going to proceed to introduce the relevance of eccentricity in considering this as the next stage.

COMMISSIONER FENWICK:

Right then it becomes vitally important to consider the centre of stiffness.

MR RENNIE:

Yes, yes but the difficulty that I am addressing Sir is what a designer in 1986 would understand to be required by 3.1.1 rather than what an engineer would see looking at it holistically as the appropriate way to approach the issue.

COMMISSIONER FENWICK:

In 1986?

MR RENNIE:

Sorry in 1986 Sir, yes.

COMMISSIONER FENWICK:

I would hope that it would be the same in 1986 as is in 2012.

MR RENNIE:

Indeed Sir.

CROSS-EXAMINATION CONTINUES: MR RENNIE

Q. Just before we pass to eccentricity Dr Jacobs and I have taken on board your previous answer. There are words in the reference to symmetry, "As nearly as is practicable."?

A. Yes I saw that.

Q. And at an earlier point in the hearing and you may or may not know this, I asked Mr Henry about a building Landsborough House, which he didn't remember the name but that was the building?
A. Yes.
Q. And asked him about that and his response essentially was that the words, “As nearly as is practicable,” provided some basis for designing buildings that were not wholly symmetrical?
A. Yes, are exactly symmetrical yes, no I’d agree with that yes.
Q. It then becomes a matter of degree as to how far the words, “As nearly as is practicable,” can be taken before you know whether you have reached the point of having a highly unsymmetrical building?
A. Yes I agree with that.
Q. So would you accept that in the differing views that we have in evidence about whether this building was symmetrical we are seeing – and are all engineering views, we are seeing different opinions brought to bear in that respect?
A. Yes I could accept that different engineers would have different interpretation as to what is near as practical but still does seem to me to imply as near as practical as what it says. It doesn’t mean wildly eccentric or very eccentric.
Q. No.
A. It means as near as practical to symmetrical. This is an aim that an engineer is admonished here to have.
Q. And you have now taken us to eccentricity because essentially symmetry and eccentricity and the other word which is used is regularity, are interrelated in this assessment aren’t they?
A. They are interrelated yes.
Q. And the point that you were making to me as to the relative contribution to resistance of seismic forces of the two shear walls is part of the overall weighting or assessment of their contribution when you come to eccentricity and regulatory?
A. Yes it is.

Q. And therefore do I understand you correctly to say – I will just change that. Do I understand you to say that the correct way of looking at 3.1.1 and symmetry is in respect of all those aspects symmetry, eccentricity and regularity?
A. Yes I suppose you should look at them all, yeah.

1120

Q. And in assessing compliance with 3.1.1 am I right that you've not made an assessment as to whether if the columns were treated as being part of the resistance of seismic forces that would affect the degree to which the building was asymmetrical?
A. I haven’t mathematically done that. I've only done that by judgement, by inspection.
Q. In paragraph 14 of your evidence you make a reference to very irregular buildings. Is that intended to include the CTV building as such a building?
A. Yes I thought it was regular vertically, but I thought it was irregular in plan, so I felt that you could not ignore that clause because of the plan shape of the building because it had re-entrant angles from the core wall on the north side and the fact that the line 5 shear wall was outside the main part of the building. I thought that was irregular.
Q. And they are the features that led you to that conclusion?
A. They are the main features yes?
Q. And again that's a matter of opinion and interpretation?
A. I would agree with that, yes.
Q. Then in paragraph 15 you come onto this issue about, in the case of failure, are a risk to life, these difficult words?
A. Yes.
Q. Am I right in understanding your opinion on that, that you assess that as meaning that in the event that an element failed that directly or indirectly would create a risk to life, or is it alternatively a situation where it is the element itself whose failure must cause the risk to life directly?

A. Could you just repeat those two definitions before. I'm just trying to get a picture of it.

Q. I'll try and give you a simple, and being a lawyer's one, probably not all that helpful illustration. If a column falls over on you, it's a risk to life?

A. Yes certainly.

Q. If a column falls which causes a beam to fall which causes a slab to fall and the slab is the risk to life are you saying that the second brings the column inside the definition?

A. It's a bit tricky for me that question but I think if the column fails the whole building's going to fall down isn't it, so I mean, are you saying that if the column fails and hits somebody on the head that's sitting next door to him, but the building doesn't fall down, then that's a different – if the column falls down and the whole lot comes down, I suppose that's correct but I can't see how one follows without the other, but fair enough.

Q. I think you're assuming in relation to my first illustration that the fall of a column is accompanied by some other failure, whereas my intention was to try and have two illustrations. One where the immediate failure causes the risk, and then the other where the immediate failure causes a chain of events where the risk arises somewhere else. Those are the two points that I'm looking at.

A. Yes, I – to be honest –

JUSTICE COOPER:

Did you say causes a chain of events, Mr Rennie?

MR RENNIE:

Yes Sir.

JUSTICE COOPER:

Because if you did, I don't understand if it's a question worth asking I'm sorry, I mean if a column fails causing some other failure, and there is that causal relationship, and the result is a loss of life, then the failure of a column –

MR RENNIE:

Sir I agree with that and –

JUSTICE COOPER:

There's a risk to life.

MR RENNIE:

The first scenario was where the element fails but the only risk is from the failure of that element. The difficulty that I'm aiming at here Sir is where you draw the boundary between what is inside the definition and what is outside.

JUSTICE COOPER:

Well I understand that but you've got to actually put your premise to the witness. He's got to be asked to imagine, was it one column or a number of columns that might fail with no structural consequences other than for a person who happens to be beside the column and it falls on him. That's your scenario isn't it?

MR RENNIE:

That was my example.

JUSTICE COOPER:

So once you've established all those premises that's why I come back to is it a question worth asking. Look it's a matter for you, I don't want to (overtalking 11:25:51).

MR RENNIE:

No, no Sir but as a question I think it's proved to be unhelpful Sir, but as an issue I'll put it this way.

CROSS-EXAMINATION CONTINUES: MR RENNIE

Q. Ultimately there is some element of risk in every part of the building. Where do you draw the boundary between what you pay attention to and what you don't?

A. I think it's a matter of judgement but I think as far as the main columns that hold up about 90 percent of the building concerned, that's what struck me as being very, very important for the loss of life and that's what I would certainly include underneath that. If you're talking, well I leave it to you about other things that may have fallen. I can think things on the top floor that's about all I can think of or maybe some ceiling or a bookcase or something like that that might be individual.

Q. Sorry I just didn't catch the last bit of what you said.

A. Oh, I thought there maybe some other elements that fit into your first definition but I'm just having trouble thinking of some of them.

Q. At paragraph 39 of your brief you make reference to the adequacy or perhaps more accurately in your view the inadequacy of the reinforcement of the slab?

A. Yes I do, yes.

Q. There has been evidence that in addition to the 664 mesh which was placed in the slab there was a second lot of mesh that was placed there to meet FRR requirements, Fire Resistant Rating requirements, were you aware of that?

A. I wasn't aware that there was a second layer, I thought the same layer was supposed to – I wasn't aware of that. I thought they deflected that layer of mesh, the 664 mesh. Was there somewhere on the drawings that says there's two layers?

Q. I'm based on what was said in evidence but –

A. I couldn't find evidence on the drawings but maybe I didn't see it.

Q. I'll see if I can find you a reference after the break.

A. Because I was concerned about that issue. I've mentioned it in my reply to Mr O'Leary's that in my experience it would be hard to – I'd never come across that before. Normally the practice I'd seen done at that time was to put another straight steel bar in the bottom trough of those metal decks and to try and deflect your reinforcement down to act as a sort of a little suspension bridge to pull up that is quite a difficult thing to do. I spent most of my time on site trying to stop big construction workers deflecting that and putting those things out of place because normally you want it close to the surface to stop cracking. That mesh is quite light. It's only one heavier than what you'd put in a concrete footpath, so it's not a heavy mesh so I was concerned about that.

Q. Well given that it comes up in Mr O'Leary's evidence it might be that we will come back to that?

A. Yes, I was unaware there was two layers on mesh in that slab.

Q. And it's possible that I'm wrong. I'm wrong on all sorts of things Dr Jacobs so I may be wrong on this but I'll have a check. You also referred to the span in relation to, the span I should say of the Hibond.

A. Yes I did, yes.

Q. In relation to the 1985/86 documentation –

A. Yes.

Q. – of the Dimond organisation.

A. That's right.

Q. And are you aware that Hibond was initially developed in Australia and then licensed into New Zealand by the Australian owners?

A. I don't know that, I couldn't say that I wasn't aware but I probably was not aware, but I don't know that, I can't remember that.

1130

Q. I just wondered whether the documentation which you've consulted in relation to the permissible spans where you noted an inconsistency with the code, whether that was Australian sourced documentation or New Zealand sourced documentation?

A. I've got it here. On the last page it's got, "Dimond Industries, Head Office, Auckland, Wellington, Palmerston North." It doesn't mention but I was aware I think it was Lysaght's was the name of the company to bring it over but that document, that purports to be from that time. The reason why I was interested in that. I first of all spoke to the engineer that worked for Dimond at this stage who has been there for many years about this because I had used Dimond HiBond too and its primary design was steel structures where you have close steel secondary beams, maybe three metres, and the idea is you don't need to prop it so, and that's what it was very helpful for but in New Zealand we tended to use it in concrete buildings and stretch it, push it, and there's an issue with the bond between the concrete and the metal deck because there's no actual stud interference, so you're relying on almost the adherence or the glue or whatever you like to call it, the bond between the concrete and, now if you push this relationship too far, that is the factor that causes a failure if you were to get a failure from these things. So when you push it out to seven and a half metres it just, it rang a bell in my mind saying are we pushing it too far here, till we do get problems with this. That's why I brought that HiBond up.

Q. And the long serving Dimond person that you spoke to, did they, he or she have a view on the spans?

A. Yes he told me that he thought that they don't do it anymore, that six metres is more they recommend because he was worried about this bond between the HiBond and the concrete. He said that would be something that we wouldn't rely on so much as we had in those early cases.

Q. So the, both the span and the reliance may well have been something commonly done in the mid '80s but would not represent current practice?

A. That's as I understand it yes.

Q. And to be clear, when you refer to failure are you talking about the slab delaminating from the HiBond or are you talking about the slab failing in the sense of breaking and falling?

A. Well strictly they'd be the same thing because if it de-bonds then the reinforcing is gone from that slab. The only real reinforcing in that slab is the HiBond and if that was to fail you've only got that light mesh in there, maybe the two, I'm not sure whether it has two or not but that would be struggling to hold it up, or even though the fire calculations show that it would, but you would have to say that that slab wasn't functioning any more.

HEARING ADJOURNS: 11.33 AM

HEARING RESUMES: 11.48 AM

CROSS-EXAMINATION CONTINUES: MR RENNIE

Q. Dr Jacobs, I'm going to ask that on the screen in front of you is put paragraph 30 of Mr Harding's evidence, it'll come up on the TV screen, you can look at the little one in front of you?

A. It hasn't come up yet. Yes, it's up now.

Q. If you have a look at paragraph 30 which can be enlarged I think if we can...This is what I was asking you about in relation to the additional mesh in the floor before.

A. So I wasn't aware that that was extra reinforcing as above what was there already.

Q. So in terms of an overall assessment in slab strength we don't therefore know what, if any, significance that may have?

A. That's correct. No. The only thing, well it does have some effect on the slab strength. I've had no evidence there's extra reinforcement but because the reinforcement is drooped, draped shall we call it, in the centre of the slab that means the top of the slab has no reinforcement in it so that was of concern to me that if you have reversals of moment on that slab you could get into problems with that.

Q. Well are to hear from Mr Harding later on so no doubt it can be clarified but –

A. Certainly.

Q. – I at least have understood that evidence to indicate that there was mesh, hibond and something in the nature of fire reinforcement as well?

A. Well it'd be interesting because I have not located that on the original drawings that went through the Council.

Q. If you turn to paragraph 28 of your evidence you say that the columns of this building were classified as group 2 secondary elements. It's not clear to me whether you are actually adopting that as your view in paragraph 28 or simply reciting something?
A. Well my view overall is that they shouldn't have been adopted as secondary elements but I think they were. I think what I meant, what I meant there was they were adopted by the designers.
Q. Yes, because in fact if we come to paragraph 36 of your brief.
A. Yep.
Q. You say that notwithstanding the difficulties with the code clauses you don't think they should have been seen in that way?
A. No I don't think they should have been seen in that, well the difficulty with the code clauses I also thought that the predicted moments generated in those columns exceed that clause 14. – 3.5.14.
Q. The view of several of the other engineers who filed briefs appears to be, and of course we have yet to hear them present those briefs, that the column and frames could be considered as secondary elements are we again in an area of engineering opinion?
A. I would expect so, yes.
Q. Yes. Next and this touches on a point that my friend Mr Reid asked you about but in paragraph 33 as I understand it you agree that if the columns did remain elastic then they did not require ductile detailing. Is that right?
A. That's what I believe the clause says, yes.
Q. Yes, and so a designer who had reached the conclusion that the members would remain elastic under the V-delta drift does not need to proceed further?
A. If he, if he takes that as a sole reason that clause then that would be correct.
Q. Do you agree that there's also an issue as between the concrete code and 4203 as to the extent to which concrete elements need to possess ductility where they might present a risk to life in the event that they fail?

A. If I understand your question I don't think there's a disconnect there as to the importance of it.
Q. Mhm.
A. I believe that clause there is meant to, is specifically related to concrete but in that concrete code I think before we went through the commentary where it said there, just let me look at those clauses again, what I got from that –

JUSTICE COOPER:
Q. What are you looking for?
A. I'm just looking for clause 3.5.14.3: "group 2 elements shall be detailed to allow ductile behaviour". That's a statement in the code and in accordance with a, I believe that they put that there for a reason, group 2 elements shall be detailed to allow ductile behaviour. Well how can you then say that you shouldn't, the next clause in (a) you go against that recommendation. That's what I found hard to accept so I think that is in the same vein as 4203 which is telling you to detail for ductility. It may be confusing but that's the statement in 3.5.14.3.

CROSS-EXAMINATION CONTINUES: MR RENNIE
Q. Yes, and you say in paragraph 33 that taken in totality 3.5.14.3 is difficult to interpret?
A. I think it is difficult to, it's confusing, yes.
Q. Yes.
A. Because if you read that statement you would detail all secondary elements type 2 for ductility.
Q. And that doesn't seem a logical outcome does it?
A. Well it doesn't seem to me I suppose to relate to (a). But nevertheless though I find it hard to ignore it.

JUSTICE COOPER:

Q. Is the expression "additional seismic requirements of this code" one which has an obvious meaning? Is it defined somewhere? A term of art under the –
A. I know what it means to me, it means to look at the ductility requirements because but –

COMMISSIONER FENWICK:

Q. When you look at the code going through the different chapters you will find that there's the normal design criteria then there are additional seismic requirements?
A. Yes that's correct.
Q. And they appear at the end of each chapter, so I think, sorry answering the question for you –
A. Yes.
Q. – but probably easier for me to do it –
A. Thank you very much for assisting me there.
Q. Yes, there's specific title in each chapter which indicates that.

CROSS-EXAMINATION CONTINUES: MR RENNIE

Q. The way that you apply this essentially in any particular building is going to determine upon the deflections that you have computed or obtained from using ETABS or whatever your design programme is and then applying the code, isn't it?
A. That's what I understand yes.
Q. Yes. So that a designer who had done those calculations and rightly or wrongly concluded that the deflections were not at a level where ductility was required does not need to proceed further?
A. If you strictly stick by that code requirement (a) that's correct.
Q. Now going back to 4203 if you could turn to 3.8.1.2.
A. Yes.
1158

Q. You indicated in your evidence when it was being led I think that you'd proceeded on the basis effectively of hard foundations. Is that correct?

A. That's correct, yes.

Q. And 3.8.1.2 would not come into play if that is your basic assumption would it?

A. Well I believed I'd conformed with that clause, yes.

Q. Yes, well if you take a hard foundation then you have no foundation rotations to consider do you?

A. Yes, if you don't put springs in your foundation you assume where the columns or the foundations beams are supported that's a rigid support there, yes as I understand that.

Q. Now we know that in 1986 in relation to the CTV building a report had been obtained in relation to the soil conditions done by Mr McCahon and based on I think a total of 13 bores of various types. Have you examined those?

A. No I haven't examined that report, no.

Q. So is it that the evidence that you have given simply proceeds on the basis of an assumption of a hard foundation and you've not looked beyond that?

A. I have looked at flexible foundation results which were provided to me by Compusoft and they showed a more flexible building because of that but I thought it was a more stringent requirement and more forgiving for the columns and the deflections to take a rigid foundation which I believe this clause says because it says there, "Computer deformations *shall* be calculated..." It doesn't say *may* be or *should* be. It says *shall.* I'm not a lawyer but I would understand that is a command rather than a request.

Q. It's expressly stated in the Bylaw of the Christchurch City Council that shall is a requirement so you...

A. And I think it was general practice at that time not to put in springs underneath your foundations because division of ETABS which I used later in that didn't have an easy to use facility to do that. You could do it but it wasn't easy to use.

Q. I understand it involved putting a notional basement in an allocating column values to try and equate the values for springs?
A. Yes, I understand so.
Q. The code though didn't actually prohibit paying attention to the actual soil conditions did it?
A. I can't see a clause there that says "thou shalt not put in springs".
Q. Although in paragraph 34 you appear to be proceeding on the basis that that was what the code required?
A. Code required which – rigid foundations?
Q. Stiff base?
A. Yes, well that's what my interpretation would have been on that clause 3.8.1.2 and also the fact that that is what was standard practice I believe in those days.
Q. And that standard practice driven in part at least by the factor you've mentioned of the limitations on computer modelling at that time?
A. Yes, yes I suppose so. That was the reason.

MR RENNIE ADDRESSES THE COMMISSION

I was about to say that the other matters that I have essentially relate to matters in issue between Dr O'Leary and Dr Jacobs and if I have the sequence right it's intended that Dr O'Leary will give his evidence and then Dr Jacobs will come back and it seems to me that I should desist at this point because the points I have may either be dealt with, invalidated or become triumphant Sir so at this point if it's suitable I will sit down.

CROSS-EXAMINATION: MR ELLIOTT

Q. Dr Jacobs I remain conscious during my questioning of His Honour's comments about legal issues. I am going to ask you one or two questions though about the Bylaw and just ask you about your engineering view. Now Mr Rennie referred you to clause 8.4.1 and I'll just bring that up on the screen, ENG.CCC.0044A.72. And if we could have clause 8.4.1 enlarged please. Do you see this is the clause in the Bylaw that Mr Rennie referred you to?

A. Yes that's the clause he referred me to.

Q. And it refers to concrete elements. Do you see that?

A. Yes I do.

Q. Firstly concrete is a construction material isn't it?

A. Yes it's right, yes.

Q. And an element in a building can be, for example, a beam. Is that right?

A. Can be a beam.

Q. Or a column?,

A. A column.

Q. Or a wall?

A. If it's a concrete wall, yes a concrete wall, yes.

Q. And the code referred to there NZS3101 sets out the visions about the design of such elements doesn't it?

A. Yes it is. I believe it's the concrete code.

Q. I'll now refer you to a separate section in the Bylaw which is at page 86 and if clause 11.1.5 in its entirety could be enlarged please. Do you see there it says, "The general structural design method as distinct from detailed design appropriate to particular construction materials as required elsewhere in this Bylaw....." et cetera. So firstly do you see there appears to be a distinction drawn between general structural design method and detailed design appropriate to particular construction materials. Do you see that?

A. Yes I do see that.

Q. So the reference to design appropriate to particular construction materials, would you interpret that as being relating to the design of concrete elements as referred to under NZS3101 as we've discussed just now?

A. Yes I would think concrete would be one of the construction materials, yes.

RE-EXAMINATION: MR MILLS

Q. I have just one point I want to raise by way of re-examination Dr Jacobs. Don't know if I've got the wording quite right but I think I've got the essence of it. You were asked a question by my friend Mr Rennie which was along the lines of a designer who has done the calculations to determine deflections and rightly or wrongly concluded from those calculations that they were all right, would then proceed on that basis. Do you recall that being put to you?

A. Yes I think I do.

Q. I just want to make sure that that is clearly understood by you as to this question about a designer doing calculations and rightly or wrongly coming to a conclusion on deflections. Is it your position that if a calculation is done and it shows a certain result that the calculations are demonstrably wrong that somehow that immunises the conclusion that's been drawn by the designer from criticism in terms of code compliance?

A. Well no I wouldn't say. It's not as simple as that. A designer is expected to be an experienced designer and to use his overall experience and judgement to look at these matters, and also all the other clauses in the two codes which apply, the loadings code and this concrete code to make that decision, he would realise the gravity of what he was doing if he's going to ignore that with a, if the calculations could be not correct, that he is exposing himself to a high risk I'd say based on a mathematical analysis which has been cautioned especially in 1984/'86 as being probably suspect.

Q. Just so I can be clear on this, in terms of code compliance questions which is what we're fundamentally dealing with in this part of the hearing process. A designer does a calculation that to him says whatever it is I'm checking is code compliant and proceeds on that basis but that calculation or assumption is established to be incorrect. What's your position on whether the code compliance is satisfied by what the designer may have believed rightly or wrongly?

A. Well I'd expect that it was since shown by quite rational and backed up by other people's opinions to be wrong then it hadn't conformed with the code.

QUESTIONS FROM COMMISSIONER FENWICK:

Q. A structural engineer, practising structural engineer in high rise buildings would have to go through a series of courses at university or some other and then act in a manner where he was detail supervised for a number of years before he could be registered or be considered competent for structural design. Would you agree with that?
A. I would agree with that.
Q. Would you say that this knowledge that he gains in that process would teach him how to interpret the code and where to ignore the code?
A. Yes I would think it would, I think it, yeah your experience is very, very important. Especially when we're talking about these analyses and I think I mentioned before how it seemed important to me to approach it from hand analysis, to two dimension and work up to the more complicated analysis. Not just to accept one run of a complicated analysis as being the solution to the problem.
Q. Yes I like the approach you've got there. Now just to illustrate one case where one actually should ignore the code. Imagine you have some columns like the ones in the CTV building where the columns as you come down are getting more highly loaded?
A. Right.
Q. Would you agree the more highly loaded are going to be more brittle than the ones at higher up?
A. They certainly are, yes.
Q. Especially if they've got the same reinforcement arrangement. Now if we go to an analysis and at this stage we'll make the assumption that the ground's assumed rigid which is what was done as I understand it, right through, you know, through the '80s. It's very unusual to allow for soil deformation. We'll come back to that one in a minute. If we did that analysis, do an equivalent static or a modal analysis, you would get a deflected shape which would be a curve like that wouldn't it?
A. Yes for a shear wall yes.
Q. So the highest drifts would be at the top of the building?
A. That's correct, yes.
Q. And the lowest drifts at the bottom?

A. Yes.
Q. But that's not what happens is it? Because you form a plastic hinge at the bottom in reality and it rotates at the bottom and though you've got slightly higher drifts at the top you've now got a very much higher drift at the bottom than your analysis?
A. Yes you would do because yes because it's an elastic analysis.
Q. The wise engineer would know the elast – well sorry, the acceptably competent engineer would know that he could not take the storey drift off a modal or a cognistatic analysis. He would have to make an allowance for the inelastic deformation increasing the deformation of (inaudible 12:12:41) do you agree?
A. Yes I do agree. I think that probably is backed up by I think it's in the commentary on 4203 which says one of the aims in giving the philosophy of design is to have eight repetitions, I think it is, at four times the elastic deflection. So that's what your aim is when you're designing them and that's a pretty severe aim, really. It means that you have to take that into account to take four times the elastic deflection eight times.
Q. If we bring it back, yes I agree with you there but if we bring it back to the CTV, you cannot in fact take your modal analysis or your equivalent static analysis to give you a realistic assessment of what that column will be subject to near the base, the brittle column would be subject at the base, can you?
A. No I don't think you can, no.
Q. Now just one other thing on this soil flexibility. Now if you have a raft foundation or something and the whole thing rotates, it has no effect on the wall does it, essentially in terms of actions, slight increase in P-delta actions but in terms of –
A. No, the walls still would be –
Q. Inter-storey drifts don't matter, they all increase so – but if you have a wall and then you have secondary frames alongside it, the soil deformation is now going to be different for the secondary frames than it is for the structural walls isn't it?

A. Yes it is, certainly yes.

Q. So in that case ignoring the, computing your deflections, ignoring the soil deformation, in that particular case where you have secondary frames alongside, wouldn't an acceptably competent engineer say no I can't do that?

A. Yeah, but they give the – they don't give the what you'd call the lower bound. I think we mentioned before that if you had the flexible you're going to get more drift from the flexible foundations put onto the building than you do if you assume it's rigid.

Q. Yes and that would reduce the forces on the structural walls but increase the deformations on your frames?

A. Yes, certainly would yes.

Q. And any engineer who's presumably been through university and perhaps one or two years after would recognise that straightaway and would not go down that track would they, of assuming you had soil deformations and then removing those soil deformations from your structure?

A. Yes, no I think that would be true.

Q. Yes, so would your interpretation of that clause, sorry I can't remember the number, 3.8.1.2 I think, but that clause would actually say when you do your analysis do what we've been doing for the last 10 years, modal analysis, I'm talking about the 80s now and you assume your foundation is rigid?

A. Yes.

Q. And partly I suggest, I'd like your comment on this, this might have been a problem also in knowing what the stiffness, the short term seismic stiffness of ground would be (overtalking 12:15:46).

A. It would be, yes, yes I think there's – the geotechnical reports which I think was mentioned done for this building and I haven't sighted it, but a lot of those gave settlement, gave deflections that were really settlement deflections. They were long term deflections that you would get in and you should just be aware of these in the design of the foundations so you don't get unacceptable settlement.

Q. Yes.
A. I'm not aware at that state that I had soil reports that would give me dynamic characteristics of the soil.
Q. Very difficult too isn't it, because when you put a building on a site it will settle and it will consolidate, it will change the character of the ground and therefore it might start of as flexible but it could go stiffer?
A. It could change, yeah, the core pressures could evaporate or -
Q. So I mean there was quite a few problems I think, geo-mechanics is probably advanced since the 80s but at that stage that would have been one of the reasons why we would have stuck to a –
A. Yes I think that would be –
Q. Why at that stage they would have stuck to the rigid foundation.
A. Mhm.
Q. One other very quick question, the connection of the wall, the floor to the north wall complex was through that one bay.
A. That's right.
Q. Did you look at the sort of stresses that might have been induced in the reinforcement near that bay?
A. This is in the slab?
Q. In the slab.
A. Yes I did. That little slab there if you draw a plan picture of it and draw on it the starter bars that come out of the wall.
Q. Yes.
A. And the saddle bars that go over the line 4 and the little bit, you end up with quite a large area in the centre, that only has 644 mesh in it.
Q. That's right.
A. And if you’re taking diagonal tension each way, depending on which way they are – you get that those are severely stressed mesh in there.
Q. Yes.
A. Yeah, I couldn't see how that worked really but that was a first – that worried me quite a lot that.

QUESTIONS FROM COMMISSIONER CARTER – NIL

QUESTIONS FROM JUSTICE COOPER – NIL

WITNESS STOOD DOWN

MR ALLAN CALLS

ASHLEY HENRY SMITH (SWORN)

Q. Good afternoon Mr Smith.
A. Afternoon.
Q. You've appeared before this Commission on a few occasions already in this section, but the first of those occasions was to give evidence along with Dr Hyland as to the content of the building collapse report that you co-authored with Dr Hyland.
A. That's correct, yes.
Q. And in that report or rather the report, makes comment and draws certain conclusions with respect to design and compliance issues associated with the CTV building?
A. Yes, that's right.
Q. Including compliance issues associated with inter-storey drift limits for the building itself, the drift capacity of columns, minimum shear requirements of columns?
A. Correct.
Q. Now at page 12 of that report, it's noted Mr Smith that interpretations by the authors varied on a range of issues and one of those is interpretation of the requirements for design of secondary structural elements?
A. Yes.
Q. And as a result of that particular variation in interpretation, have you been asked by the Commission to prepare your own statement of evidence setting out what you understand to be the requirements for compliance and your conclusions with respect to compliance?
A. Yes I have.
Q. Was that your third statement of evidence which has been lodged with the Commission under the code WIT.SMITH.0003?
A. Yes that's correct.
Q. Now do you have a copy of that with you this morning or this afternoon Mr Smith?
A. Yes I do.

Q. Well perhaps if you could read through that please, commencing at paragraph 3.

A. I have been asked to provide evidence to the Canterbury Earthquake Royal Commission relating to my interpretation of the structural design codes of the day, ie
NZS 4203 1984, code of practice for general structural design and design loadings for buildings, and
NZS 3101 1982, the code of practice for design of concrete structures as they apply to the design of columns and beam column joints in the CTV building.
In particular:
the requirement to design the columns to possess ductility and
the requirement to design the columns to withstand deformations due to earthquake loads.
I have read and agree to comply with the code of conduct for expert witnesses, a copy of which is attached.
I confirm that the matters I am giving evidence about are within my area of expertise.
The title Code Interpretations Varied, referring to the DBH report.
Interpretations of these code requirements varied between myself and Dr Hyland, Dr Clark Hyland, the co-authors of the CTV building collapse investigation report dated 27 January 2012, (the Hyland Smith report) as stated on page 12 of that report. The interpretation described on page 20 and page 109 of the Hyland Smith report, and in the Appendix F titled “Displacement capability analysis to standards,” was Dr Hyland’s interpretation.
It is worth noting that other members of the Department of Building and Housing expert panel also had different interpretations of the codes in relation to column design. This is an indication to me that the design codes of the day were not entirely clear.
Although there were variations of interpretation, neither myself nor Dr Hyland and also none of the DBH expert panel members thought that the design of columns, of the columns would have complied. The

variations of interpretation were about the extent of non-compliance only and which of the criteria in the codes was most critical. My interpretation of the design codes follows.

Under the heading "The requirement to design columns to possess ductility."

The CTV building was designed with the reinforce concrete north core walls and the south wall as the primary bracing system to resist lateral loads from earthquake. These walls were designed to be ductile and were therefore required to be designed to be capable of dissipating seismic energy by flexural yielding.

Further, the north core walls and the south wall were required to be subject to capacity design which is defined in NZS3101:1982 as follows:

"Capacity to design definition. In the capacity design of earthquake resistance structures elements of the primary lateral load resisting system are chosen and suitably designed and detailed for energy dissipation under severe deformations. All other structural elements are then provided with sufficient strength so that the chosen means of energy dissipation can be maintained".

In the case of the CTV building "all other structural elements" would have included the columns.

There was an overriding requirement in clause 3.2.1 of NZS4203:1984 that was applicable for all buildings, ie. not only those constructed of concrete, that's stated:

"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…"

The CTV building columns were elements that had to resist seismic movements and they were also a risk to life if they failed therefore under this clause 3.2.1 they were required to be designed to possess ductility.

As defined in NZS4203:1984:

"Ductility means the ability of the building or member to undergo repeated and reversing inelastic deflections beyond the point of first

yield while maintaining a substantial proportion of its initial maximum load carrying capacity".

Applying this definition to the CTV building columns, I would interpret it to mean that there was a requirement for the columns to be able to undergo repeated and reversing inelastic horizontal deflections or inter-storey drifts beyond the point of first yield while maintaining a substantial proportion of their initial maximum vertical load carrying capacity.

Circular concrete columns are generally designed to possess ductility by providing a number of vertical steel bars around the perimeter of the section and wrapping around those vertical bars a sufficient quantity of spiral reinforcing spaced at sufficiently close centres so as to:

first prevent shear failure and

second to provide confinement to the concrete core of the column to ensure that it can maintain its vertical load carrying capacity.

There are separate sections in the code of practice for design of concrete structures NZS3101:1982 for design for shear which is covered in section 7 and for design of flexure and axial load including confinement which is covered in section 6. Shear is generally considered separately from confinement in the design of reinforced columns. Spiral reinforcement is required to resist shear and to provide confinement but according to different rules for different regions of each column. The final design of reach column needs to have at least the minimum quantity of spiral reinforcement required for shear or for confinement at each section whichever quantity is greatest.

Under the heading, "The requirement to prevent shear failure."

Shear force in a column refers to the horizontal force which is generally constant over the height of each column at each storey in the building. If shear forces become large they can result in the formation of inclined cracks followed by shear failure if those inclined cracks become too large or separate altogether. This is a non-ductile or brittle type of failure which is to be avoided. Therefore an essential first step in the design of a column is to ensure sufficient shear strength by including an appropriate minimum quantity of spiral reinforcement to provide tension across any potential inclined shear cracks thereby preventing such cracks from opening.

The minimum requirements to prevent shear failure of columns were clear in the standards of the day for the CTV building and there was no variation of interpretation on this aspect by myself, Hyland or any others in the DBH expert panel.

NZS3101:1982 clause 7.3.4 states that *"A minimum area of shear reinforcement shall be provided in all reinforced… (I haven't included all the words) …concrete, where shear stress vi required to resist Vu (which is the applied shear) exceeds half the shear strength provided by concrete…"*

And I've, just defining those terms:

vi is the total shear stress and

Vu is the factored shear force at the section.

NZS3101:1982 commentary clause C7.3.4 provides further guidance as follows:

"When repetitive loading might occur on flexural members the possibility of inclined diagonal tension cracks forming at appreciably smaller stresses than under the static loading should be taken into account in the design. In these instances it would be prudent to use at least the minimum shear reinforcement … even though tests and calculations based on static loads show that shear reinforcement is not required".

My interpretation of this NZS3101:1982 clause 7.3.4, taking into account the point in items 11, 12 above, that there was a requirement for the columns to be able to go, undergo repeated and reversing inelastic horizontal deflections, is that at least the minimum shear reinforcement was required over the full height of all columns is that at least the minimum shear reinforcement was required over the full height of all columns. I calculated the minimum shear reinforcement as R6 spiral at 90 millimetre centres or R10 spiral at 150 millimetre centres as explained on page 110 of the Hyland-Smith report.

Under the heading, "The requirement to provide confinement."

There were variations of interpretations by the authors and by others on the expert, DBH expert panel over the requirements for confinement of columns in NZS3101:1982. The reason why different interpretations existed can be explained partly by the structure of the standard as explained in the foreword to the standard which is attached.

Paragraphs 3 and 4 of the foreword state:

"The arrangement of clauses represents a significant change in format from the previous code with the aim of producing a more workable document.

The intended order of usage is that after proceeding through notation, scope and general principles and requirements which apply to all structures the designer then either, goes either to: the principles and requirements additional to clause 3 for members not designed for seismic loading, or to: the principles and requirements additional to clause 3 for members designed for seismic loading. That is, only one of the last two clauses is used but not both".

There is a diagram attached which would explain that.

If we consider that the above arrangement of clauses in relation to section 6 of the standard where the provisions for confinement of columns are contained we see that there are no general requirements for confinement under section 6.3 and there are different requirements for spiral confinement in clause 6.4.7.1 the additional requirements for members not designed for seismic loading, and under clause 6.5.4.3 the additional requirements for members designed for a seismic loading and under clause 6.5.4.3 the additional requirements for members designed for seismic loading. Also within clause 6.4.7.1 there is the option for either, using either a strength reduction factor of 0.9 which is subsection (a) or a strength reduction factor between 0.7 and 0.9 which was subsection (b), each with different requirements for minimum confinement.

Further under certain conditions structures and members including columns could be designed for limited ductility, in which case the additional requirements of section 14 of the standard would apply. The provisions for minimum confinement for columns in section 14 are contained in clauses 14.6.2 and 14.6.3.

The rationale for limited ductility section 14 in the standard is explained at the bottom of page 12 of the foreword attached as follows:

"Section 14 give the design and detailing provisions for members in structures of limited ductility subject to earthquake induced loading. This section recognises that less stringent ductility requirements are appropriate because of the larger lateral design loads applicable to such structures".

Thus, if one is designing confinement for a column in accordance with NZS3101:1982, the requirements for the minimum amount of spiral reinforcement are contained in four separate clauses of the standard as follows:

6.4.7.1(a) under the additional requirements for members not designed for seismic loading when a strength reduction factor of 0.9 is used, or

6.4.7.1(b) under the additional requirements for members not designed for seismic loading when a strength reduction factor between 0.7 and 0.9 is used. The third option is:

6.5.4.3 under the additional requirements for members designed for seismic loading with full ductility, and the fourth option is under:

14.6.2 and 14.6.3 under the additional requirements for members designed for seismic loading with limited ductility.

It is evident from a review of the structural drawing S14 titled "Columns" for the CTV building that confinement for those columns was designed on the basis of, should be of the minimum requirements to NZS3101:1982 clause 6.4.7.1(b) only, because it was not satisfy any of the other three clauses referred to item 26 above.

Q. Item 26 Mr Smith being the various options for spiral reinforcement requirements that you've just related?

A. That's correct.

Considering the clauses listed in item 26 above it would have been sufficient to detail the columns of the CTV building with full ductility in accordance with clause 6.5.4.3 without any further checks. However, for any of the other three options for design of confinement further checks were required to ascertain whether which clauses were appropriate and which were not appropriate.

If we start by considering the choice between the clauses 6.4.7.1(a) and 6.4.7.1(b) it is helpful to read the commentary clause, sorry the commentary NZS3101 part 2 1982 clause C6.4.7 to understand the rationale behind these code provisions.

Q. Just before you do that Mr Smith. The difference between these clauses turns on strength reduction factors that are applied. Are you able to explain that concept for us please?

A. Yes it's like a safety factor. The strength reduction factor. The normal method is to apply factored loadings and then also which shows one aspect of the safety factor and the other method is, and then when we assess the capacity of a member to sustain those factored loadings we apply strength reduction factor which accounts for variation in material strengths and other issues, so it's part of the safety factor in design.

Q. So a strength reduction factor of 0.7 and 0.9 would be a more conservative?

A. A 0.7 is more conservative. That as far as I'm aware, that clause related to the clause that had shall we say detailing not for ductility and it meaning a higher safety factor had to have been applied to that level of detailing.

Q. Yes, all right, thank you, now you're about to take us to the commentary in relation to 6.4.7 which sets out these options?

A. Yeah, okay, so as explained in the foreword to the standard:

"A comprehensive commentary is published with the code and it is strongly recommended that the two documents should be read together".

That's the code and the commentary.

Commentary clause C6.4.7 states that:

"Columns may be designed using a strength reduction factor (ø) of 0.9 … if the quantity and arrangement of transverse reinforcement is

adequate to ensure ductile behaviour. Clause 6.4.7.1(a) specifies the required spiral or circular hoop reinforcement considered necessary for using ø = 0.9. The amount of spiral reinforcement required by equation 6-3, (ie. clause 6.4.7.1(a)) is intended to provide additional load-carrying strength for concentrically loaded columns equal to or slightly greater than the strength lost when the shell spalls off".

The shell, I've defined the shell in the next paragraph.

The rationale behind clause 6.4.7.1(a) is therefore consistent with requirement of NZS4203:1984 clause 3.2.1 for the columns to possess ductility and the definition of ductility in that standard outlined in items 11 and 12 above.

In item 30 the 'shell' refers to the cover concrete around the perimeter of the column section outside the line of the reinforcement. This is particularly important for the CTV building columns because they have a, they are relatively small 400 millimetre diameter sections with 50 millimetres of concrete cover to the inside face of the spiral and so the concrete core contained by the spiral is only around 56% of the total section area. Consequently the strength lost when the concrete shell spalls off is a significant proportion of the total strength.

The final paragraph of commentary clause C6.4.7 is also relevant and that states:

"Note that when the axial load on the column is low (that is P_u/f'_cA_g is relatively small the dependable strength of the column with transverse steel for low ductility ø = 0.9 may be adequate, however, the ductile design case using ø = 9 is a useful means of increasing the dependable strength of the column when the axial load on the column is relatively heavy. This is important in relation to the CTV columns because the axial loads in the columns are relatively heavy and I calculated the ratio P_u/f'_cA_g up to 0.58). In relation to the CTV building my interpretation of the last paragraph of clause C6.4.7 is that the axial loads on the columns were high and so the dependable strength of a column with transverse steel for low ductility (where ø < 0.9) may *not* be adequate. For this reason I consider that the design of confinement based on the lower limit of NZS3101:1982 clause 6.4.7.1(b) for members not designed for seismic loading and with a strength reduction factor between 0.7 and 0.9 would not be appropriate for the CTV building.

NZS3101:1982 clause 3.5.14.3 lists the requirements for group 2 secondary structural elements which include the columns of the CTV building. The first paragraph in this clause states that:

"Group 2 elements shall be detailed to allow ductile behaviour and …"

then it goes on to other state other requirements.

This statement is consistent with NZS 4203:1984 clause 3.2.1 which required the columns to be designed to possess ductility as outlined in item 11 above. We also need to keep in mind the definition of ductility outlined in item 12 above. In particular the need for the columns to maintain a substantial proportion of their initial maximum load-carrying capacity.

NZS 3101:1982 clause 3.5.14.3 then goes on to outline the various conditions where the additional seismic provisions of the code need or need not be satisfied including subsection (f) when the requirements of section 14 for limited ductility may be applied. However, in my view for the reasons explained in item 34 above, design of confinement for columns based on the lower limit of NZS 3101:1982 clause 6.2.7.1(b) for members not designed for seismic loading and where the strength reduction factor between 0.7 and 0.9 would not be appropriate in any case under clause 3.5.14.3.

Overall my interpretation of the minimum requirements for spiral reinforcement in the CTV building columns, according to the standards of the day would be:

R10 at 150 millimetre centre spiral in the mid height regions of columns between the potential plastic hinge regions which are governed by shear, and

R10 at 75 millimetres or closer spacing spiral and the potential, in the potential plastic hinge regions at the top and bottom of each column, at each storey, and this is governed by the requirements for confinement. The 75 millimetre spacing would be appropriate only if designed in accordance with section 14 for limited ductility was applicable. That would be debateable in my view, but may be relevant for the lighter loaded columns in the upper levels.

I calculated the confinement that would've been required in the potential plastic hinge regions of the indicated columns, which were those at grids T2 and F2 in the Hyland Smith report, according to NZS 3101:1982 clause 6.5.4.3 the additional requirements for members designed for seismic loading with full ductility. I calculated those as follows:

For the indicator column at grid F2 at level 3 you would need R10 spiral at 50 millimetre spacing and

For the indicator column at grid D2 at level 3 you would need R10 spiral at 40 millimetre spacing.

This is recorded in my email dated 3rd of February 2012 which was attached and marked “C”.

I also calculated the confinement that would be required in the potential plastic hinge regions of the indicator columns in the Hyland Smith report according to NZS 3101:1982 clause 6.4.7.1(a), (the additional requirements for members not designed for seismic loading when a strength reduction factor of 0.9 is used) and I found that to be more onerous than the confinement calculated in item 38. So, yeah, there’s two reasons. It was slightly closer spacing but also is required over the full height of the columns as opposed to just in the top and bottom portions.

It’s informative to read the paper titled, “The valuation of a 10 storey building using alternative structural systems,” which was written by D K Bull, who at that time was structural engineer for the Cement and Concrete Association of New Zealand, and the paper was dated October 1991. This paper was written after the CTV building was designed and so would not have been available to take account of in the design. It is referred to here only to demonstrate the debate that had occurred about some of the NZS 3101:1982 code provisions. The paper by Bull is attached and marked “D”.

Section 4.2.4 on page 11 in that paper by Bull is titled, "Design philosophies: gravity frames," and in this section Bull discusses methods of analysis and interpretations of NZS 3101:1982 clause 3.5.14 for secondary structural elements. This first paragraph in Bull 4.2.4 states:

"At the inception of the study there was much debate as to whether gravity frames should or should not be carrying lateral load? And were the designs to be limited ductility approaches or full ductility approaches".

Also in the paper by Bull, under section 4.2.4.2 titled Gravity frames: full ductility or limited ductility?

"The first reaction after deciding that the gravity frames were secondary was to start designing on the basis of members not designed for seismic loading and go to a limited ductility approach. This proved to be not completely appropriate."

(according to Bull).

My interpretation of these statements in the paper by Bull is that there would have been debate about the various options for detailing confinement to gravity frames including columns under the rules of NZS3101:1982 as outlined in item 26 above.

In relation to NZS 3101:1982 clause 6.4.7.1(b) my interpretation of the 1991 paper by Bull of that particular clause in the standard was intended only for gravity frames with negligible lateral load capacity as per section 4.2.4.3 by Bull, or for the mid-height regions of columns between potential plastic hinge regions as per 4.2.4.4 by Bull.

In the case of the CTV building this would be consistent with my interpretation as outlined in items 34 and 36 above. However in my opinion NZS3101 could have provided further explanation to simplify and clarify the intended uses and limitations for clause 6.4.7.1(b) if that was the intention.

It is significant that the structure of the standard NZS3101 was changed from 1995 as explained in the forward to that standard attached and marked "E" as follows:

"This standard features an organisational structure which is essentially the same as for NZS3101:1982. However, for the majority of sections which contain seismic provisions, there is no longer a separate clause covering the requirements for members and structures not designed for seismic forces. Such requirements are now included in clause X.3, (X being the any of numbers relating to the chapters in the standard). Such requirements are now included in clause X.3, general principle and requirements for design, with seismic provisions being addressed in clause X.4, additional design requirements for earthquake effects".
Accordingly one of the key changes in 1995 was that minimum requirements for confinement of columns were stated in the general section, thereby removing the options outlined in item 26 above in relation to the minimum requirement for confinement. The other key change was that the option to use a strength reduction factor between 0.9 and 0.7 and the associated reduced quantity of confinement was removed.

Under the heading beam column joints,
According to NZS3101:1982 clause 9.4.8:
"The horizontal transverse confinement reinforcement in beam column joints shall not be less than that required by 6.4.7 with the exception of joints connecting beams at all four column faces in which case the transverse joint reinforcement, (it's meant to be may), may be reduced to one half that required in 6.4.7, but in no case shall the stirrup tie or spiral spacing in the joint core exceed 10 times the diameter of the column bar or 200 millimetres, whichever is less".
None of the CTV building columns have beams connecting at all four faces and so the confinement had to be not less than that required by 6.4.7. Because the beam column joints are an integral part of the columns my interpretation is that it would have been necessary to provide confinement in accordance with 6.4.7.1(a) and not 6.4.7.1(b) for the same reasons explained in item 34 above.

CROSS-EXAMINATION: MR CLAY – NIL

CROSS-EXAMINATION: MR REID

Q. Mr Smith, do I take it from what you say about the disagreement between yourself and Dr Hyland that that relates to the interpretation of section 3.5.14 of NZS3101?
A. That was one aspect, yes.
Q. Was that the principal aspect in which the two of you disagree?
A. Yeah, I believe so, yeah.
Q. And as I understand it, appendix F drafted by Dr Hyland, on the basis of his interpretation of the applicability of that clause?
A. That's correct.
Q. And so you've distanced yourself from that analysis?
A. Yeah, I think what happened in the process we were co-authors and we were also working with the expert panel and it was necessary to come up with a report that everyone could live with that was not necessarily reflective of our individual opinions but Dr Hyland's interpretation was a more lenient interpretation which if we were saying things did not comply we felt we really had to go with the most lenient interpretation because – so that was one of the reasons we ran with this, yeah.
Q. You will have read the evidence of Professor Holmes?
A. Yes.
Q. And he gives an interpretation of –

JUSTICE COOPER:

Q. Who's Professor Holmes?
A. This was William Holmes.
Q. He's not a professor as far as I know, just Mr.

MR REID:

Just mister is it Sir, my apologies.

CROSS-EXAMINATION CONTINUES: MR REID

Q. Mr Holmes?

A. Yes.

Q. And his analysis broadly accords with Dr Hyland's, correct?

A. I wouldn't say that. I thought there were differences between them also.

Q. Yes, well there may be differences between them but I think my reading of both pieces of evidence is that they place some reliance on 3.5.14. Do you agree with that?

A. Okay, yes.

Q. So would you agree with me that what that really indicates is that there is room for genuinely held different professional points of view as to how the code should be interpreted in this regard?

A. I know there are different views. What was your wording again? I don't consider they are valid myself, so I have a different view that I believe if you analyse it correctly it cannot be interpreted differently so, I know they have different views, yeah.

Q. And you accept that they're reasonably held?

A. Reasonably held, reasonably held in their minds, not in my mind.

Q. But you don't agree with them?

A. No.

Q. Would you accept that at the time in 1986 there were a variety of views about how these provisions should be interpreted?

A. Yes I do. I recorded that in the paper by Bull. He's recording those discussions. I am aware that there were different views yes.

Q. So there was no settled position in 86 was there amongst the profession?

A. I, at that time I wasn't aware of alternative views in the profession. I had a certain interpretation which wasn't in line with what you see in the CTV. I wasn't aware of what other interpretations were at that time actually.

Q. So you can't comment really on what the prevailing interpretation was in '86, is that what you're saying?

A. I was based in Wellington at that time, designing substantial concrete buildings, multi-level buildings and I had a different interpretation that would not accord with what was seen in the CTV, so.

1258

Q. Yes but there were clearly, there was a body of opinion within the profession that took the view that, clause 3.5.14 allowed for the approach that we see in the CTV building.

A. Apparently there was. I wasn't aware of that at the time.

Q. Now just talking about the way that you interpret the code, in particular the concrete code. You refer in your evidence to the diagram and the discussion on page 12 which is ENG.STA.0016.14, if that could be brought up?

WITNESS REFERRED TO SLIDE

A. Yes.

Q. Yes, and this discussion on page 12 discusses, doesn't it, the general approach to the interpretation of the code, the concrete code?

A. Yes that's correct.

Q. Yes, and if you read under the heading, "Diagram indicating order of usage of clauses," section 3, you see that part?

A. Yes. Yes.

Q. That reads, doesn't it, the general design requirements has a particular importance in the code for two reasons. Do you see that?

A. Yes.

Q. And general, the expression "general design requirements" that refers to items headed, "Clause 3," doesn't it?

A. That's correct.

Q. So just so that we're clear going over to ENG.STA.0016.6 which is the index, the first page of the index, .6?

A. .6 down the bottom, yeah?

Q. My numbering's slightly different. It's the page previous to that, page 3. So the heading, "General design requirements". That's the reference that we were just looking at on page 12 isn't it?

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

HEARING ADJOURNS: 1.03 PM

HEARING RESUMES: 2.16 PM

CROSS-EXAMINATION CONTINUES: MR REID

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

a building which are in case of failure a risk to life shall be designed to possess ductility?
A. That's one, the key clause yes.
Q. Is that the principal reason that you adopt the view you do?
A. I think there are clauses in both standards that relate to ductility, so yeah.
Q. You accept though that there are clauses in 3101 which go the other way?
A. Well can we, I think we need to get specific about that. I don't accept that, as a general rule yeah.
Q. So am I right to think then that your view about ductility applies to all columns in all buildings?
A. I think at least limited ductility, so there's a level of ductility which may be covered by the limited ductility provisions in some cases, so.
Q. Yes, but it would be contrary on your interpretation to these codes to design a building that had gravity columns only?
A. Yes it would yes.
Q. In any situation?
A. Yes.
Q. So the design approach that Mr Henry describes in his evidence, gravity columns protected by shear walls, that would be an illegitimate approach on your interpretation. Is that correct?
A. Well it may be, in the case let's say the CTV building, the shear walls were designed as fully ductile and I would accept that the columns could possibly have been designed as limited ductile. So to that extent they were protected by the walls to allow that, so, but I would not accept that they could be designed without ductility, so ...
Q. My question was, so that gravity columns, so columns that have no ductility or are designed without ductility would not be permissible in any circumstance?
A. No.
Q. So that design, a design approach which permitted that would be illegitimate?

A. I believe so yeah.

Q. Have you read the evidence of Mr Henry?

A. Yes I have.

Q. He describes three buildings that he designed at 58 and 64 Kilmore Street and which – and Spicer House on 329 Durham Street which were designed with that underlying premise. You've read that evidence?

A. I've read it but I don't have any knowledge of those buildings.

Q. No, but if they – if he's correct that they were designed, he designed them, but on the basis that he's accurately describing the underlying premise with which they were designed, those buildings would be designed in an illegitimate fashion wouldn't they?

A. Yeah, in my view yes.

CROSS-EXAMINATION: MR RENNIE

Q. So just following on from where my friend left off Mr Smith, in that case why would the code contain the provisions that the columns in the CTV actually were detailed to, if there could never be a need for such provisions?

A. Yeah, the – look I, so you're referring to, I've said in my evidence that I think it was the clause that allowed that level of detailing, I didn't think was clear as to the limitations of that clause in the standard and so I think the standard was not sufficiently clear in my view on the limitations of that clause but my interpretation was that it would not be able to be used to rely on to justify the columns in the CTV building?

Q. Well would the clause have any other utility?

A. Well I've explained according to Bull there were construction methods around where you had beams and columns constructed of separate elements that when they deformed did not act as moment resisting if you like and he argues that they could be used in that situation. That was one of the applications that it could have been used in.

Q. I take it we don't have direct information as to what the intentions were of the authors of the code in 1984 in making that provision, except to the extent that it could be inferred from the commentary?

A. I guess so, yeah, I don't have the information no.
Q. So it's more a matter of your arguing that the clause should not be read that way?

JUSTICE COOPER:
Which clause sorry Mr Rennie?

MR RENNIE:
Well I'm not quite sure which clause the witness was referring to Sir so perhaps he can tell me.

JUSTICE COOPER:
We should clarify this because you didn't specify it in your question either.

COMMISSIONER FENWICK:
3.5.14.3.

JUSTICE COOPER:
(a).

COMMISSIONER FENWICK:
(a), is that what you –

MR RENNIE:
That's my understanding Sir, yes.

MR SMITH:
So there is perceived to be a conflict between this wording so the 3.5 – can we bring this clause up on the screen.

JUSTICE COOPER:
Yes, so it's STA.0016.28.

CROSS-EXAMINATION CONTINUES: MR RENNIE

A. Yes, and I'm also going to refer to paragraph 26 in my evidence, the third statement.

Q. Mhm.

A. So in item – the paragraph 26 I've identified four different clauses in the standard where you can design confinement for a column. Okay, if I'm reading this clause 3.5.14.3 the first line group 2 element shall be detailed to allow ductile behaviour. Now if I look out at those four options that I've given in paragraph 26, (b), 6471(b) which is 26.2 would not comply with that first line in that clause, so I've ruled that out as an option. I would then read on and say additional seismic requirements, now the other – the thing to note is that 647 - 71(a) is still admissible under that clause and that is not in the additional seismic requirements. So 6471(a) is still an option of those three when you're reading through that clause. So I consider that clause basically all of 3.5.14.3 is basically talking about whether it needs to be fully ductile or limited ductile, in my view.

COMMISSIONER FENWICK:

Q. Can I ask you how you interpret the commentary on that clause?

A. So the commentary, okay, 3.5.14.3, yeah. Is there a particular part of the commentary or, as I say you know I've got three what I consider three valid clauses that I still could use to design confinement. One of them is in the part of the code that is under the non seismic provisions and the other two are under seismic provisions, one for limited ductile and one for a fully ductile.

1426

Q. Do you think we need to bring that up? That will be ENG.STA.0016A.32. If we can highlight C3.5.4, 14.3(a) please. Right-hand side of the page about a quarter of the way down clause C3.5 14.3(a) if we could just enlarge that.

A. So my interpretation of that is let's say we had inelastic analysis that gave us less than V-delta deformation we're saying that we don't need to comply with the additional seismic requirements. So of the four options that I've listed in paragraph 26, 26.3 and 26.4, we would not need to comply with but it still leaves the choice between the other two and I've ruled out one of them because one of them does not allow any ductile behaviour or does not ensure any ductile behaviour. So that would leave me with 6471A which is the .9 strength reduction factor which actually when you work out the confinement under that clause it is the most onerous of any of those four.

Q. So was this clause left in by error in your judgement?

A. No I don't think it was. According to my interpretation of Bull's paper is that it had certain applications but I don't think they were clearly defined in the standards so that it was open to misinterpretation. Now I don't think you could say that 6471B detailing under that such as we had in the CTV building would not comply with the first line of the clause 3.5.14.3.

Q. I apologise for interrupting.

CROSS-EXAMINATION CONTINUES: MR RENNIE

Q. No, thank you Sir, and while we're on that if we can go back on the same page to C3.5.14.1.

A. Yes.

Q. "The definition of a secondary element is more particular than in NZS4203 and includes such primary gravity load resisting elements as frames which are in parallel with stiff shear walls and do not therefore participate greatly in resistance to lateral loads".

Do you see that?

A. Yes.

Q. That on the face of it in the Commentary is saying that 3101 is more particular than which I understand to mean more specific in detail than 4203. Do you accept that?

A. In this particular case, yes.

Q. So that in applying 3.5.14 you would be guided, would you not, by 3101 not by 4203?

A. Well it's been more particular in the definition of secondary elements and so I would accept that, I would rely on the definition in 3101 but you still have to use the base standards to design your building.

Q. In the Foreword to 4203 at page 10, I don't think we need to bring it up, the following statement appears. This relates to the 1984 amendments. You're welcome to turn to it. It's the notes of the 1984 Edition and it says there:

"Among the amendments for significant contributions is an up-grading of the section dealing with earthquake provisions. It also irons out any parts of the loadings code that happened to conflict with the various materials codes, in particular the newly issued concrete code NZS3101 – the design of concrete structures."

Do you see that?

A. Yes, well I'm listening to you. I'm not reading it.

Q. No, but that appears at page 10 of 4203 and appears, depending on your interpretation of the words "irons out", to indicate an intention that 4203 would come into line with 3101. Do you accept that?

A. Yes.

JUSTICE COOPER ADDRESSES MR RENNIE

Q. That's as said in the Foreword to the 1984 Standards though isn't is Mr Rennie.

A. That is correct, Sir.

Q. Wouldn't you take that as implying that the wording in the 1984 Standards is intended to reflect the relationship that should henceforth exist between this Standard and the material standards?

A. I accept Sir that it's only a Commentary as to purpose. I'm not suggesting that it has an actual operative status but what I am putting to the witness is that it clearly shows that the two were considered and I'm just about to put to him the parallel statement from 3101 which appears on page 12. It simply says, "It should be noted that some provisions in this code are based on proposed amendments to NZS4203 which at the time of publication are being finalised." That's the 1982 3101 is being finalised in contemplation of the adoption of the 1984 4103.

CROSS-EXAMINATION CONTINUES: MR RENNIE

Q. Do you accept that?
A. Yes.
Q. So that between those two references it's apparent that the two codes were developed over much the same time period and with an intention that they be consistent between them?
A. Yes.
Q. And the passage in the Commentary to 3.5.14, again which I've referred you to, on its face appears also to contemplate that 3101 is the reference point for the designer in relation to concrete because it is, as I put it, more particular. Do you accept that?
A. No. I'm just saying the particular thing referred to the definition of a secondary element only in my view.
Q. So in that sense do you accept that there is a difference between the standards even though one may not have been intended?
A. Yes there is a difference in that definition.
Q. And would you accept that you are arguing for an interpretation which appeals to you in your opinion as being the preferable interpretation?
A. Ah, look it's the only consistent interpretation that I was able to come up with. I mean I don't see any other consistent interpretation.

JUSTICE COOPER:

Q. When you say "consistent" –
A. Well if I'm looking at both standards across all those clauses.
Q. Your interpretation attempts to reconcile the two?
A. Yes, yes.

CROSS-EXAMINATION CONTINUES: MR RENNIE

Q. And in doing that you reject the alternative which some other engineers have adopted in their evidence in this matter of following the concrete code on this point?

A. I think those other interpretations are inconsistent even without considering 4203.

Q. Well there will be an opportunity to review that in due course. Thank you Mr Smith.

CROSS-EXAMINATION: MR ELLIOTT

Q. Mr Smith, Mr Reid and Mr Rennie asked you some questions about the fact that the NZS 3101 does have provisions for non-ductile columns in it. I just want to refer you to a section in the Bylaw, ENG.CCC.044A.87 and if we could highlight 11.2.5.2 subsection A. That provision talks about the building as a whole and its elements et cetera being designed to possess ductility but it goes on to say, "That shall not apply to small buildings having a total floor area not exceeding 140 metres square and having a total height not exceeding 9 metres." Now firstly the CTV building did not fall into that latter category did it?

A. No it didn't.

Q. And would you say that these non-ductile columns which are in the code would find a place within that type of building described in the latter part of that clause?

A. They could do, yeah.

1436

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

A. Yes.

Q. And I'll show that to you ENG.STA.0018.47.

WITNESS REFERRED TO SLIDE

Q. If the table on right-hand side could be enlarged please? From 1 down to 7. So that's a table, Mr Smith, in which a structural type factor can be nominated with a particular value, is that right?

A. Yes.

Q. And is it correct to say that as one moves down the table, one moves from ductile through to limited ductile, through to elastically responding structures?

A. Yes.

Q. Just note at this point that under item 2, if you look to the right-hand side under the S column there's provision there for 0.8 Z in the case of ductile coupled shear walls. And then under item 3 again on the right-hand side there's provision for selection of 1.0 Z as the structural type factor. I'm just going to show you one or two sections from Mr Harding's calculations. Firstly, BUI.MAD249.0272.1.

WITNESS REFERRED TO SLIDE

Q. And you may be able to see this, but if we highlight the top section of the page. Can you see there that he appears to have selected a value of "S" of 1.0 Z?

A. Yes I do.

Q. And that's when he's carrying out, he's calculating the basic seismic co-efficient?

A. Yes.

Q. Can you say if that appears to be a calculation made in respect of the north core?

A. Can we zoom out to the whole page again please? Well I would interpret the first calculation would be considering the, does it get into, sorry I can't identify whether it's the north core or.

Q. That's all right, we can ask him when he comes in. I want to refer you to a second value in his calculations BUI.MAD249.0272.28.

A. Just before we move off this one, you said that S equals one times Z?

Q. Yes.

A. And he's evaluated Z and included that S equals one?

Q. Yes.

A. Underneath that okay?

Q. Yes. He's worked out, he's applied Z as equalling one, that's right.

WITNESS REFERRED TO SLIDE

Q. And this next page, if we look down the bottom, highlight the bottom section?

A. Yes, so this appears to be relating to the south wall.

Q. And he nominates an S value of 0.8 Z?

A. Right, and S of 0.8 underneath that.

Q. Yes. Now I'll come back to that distinction in a moment. Given that they appear to be the two S values he's used, I'm going to take you back to a section in NZS 3101 1984, ENG.STA.0016.24?

WITNESS REFERRED TO SLIDE

1441

Q. And if 3.5.1.1 could be enlarged please, 3.5.1.1. Mr Smith, 3.5.1 is headed, "Methods of design," top left-hand – and it refers to a design method which shall be used and it is the case isn't it that the applicable sub-clause would be A?

A. Yes that is correct.

Q. And that refers to ductile structures being subject to capacity design?

A. Yes.

Q. So that this building would be subject, should have been subject to capacity design, is that right?

A. Yes, yes, that is right.

Q. Now I will just invite you to assume as I say Mr Harding can confirm this, but let's assume for present discussion purposes that the value of 1.0Z related to the north core and 0.8 related to the south wall?

A. Mhm.

Q. Now the effect of the selection firstly of S equals 1 is that ductile yielding is actually designed to occur at about one-fifth of full code loading, is that right?

A. That is correct, yes.

Q. And it is clear the north core did not yield in that way on the 4th of September or indeed even on the 22nd of February, is that right?

A. Ah...
Q. Or perhaps the latter?
A. Certainly the latter. We found a level – quite a low level of damage in that north core which wasn't consistent with severe hinging at the base of that wall, yeah.
Q. It did not yield in the likely plastic hinge region?
A. It may, there may have been a yielding to a minor extent in some regions of that wall but –
Q. On the 22nd of February?
A. Yes.
Q. But not on the 4th of –
A. And the 4th of September.
Q. Now, I just want you to consider given this difference between the structural type factors of the north core and the south wall –
A. Well can I, before you go on, I would say that it would be inconsistent to assume different structural type factors for different parts of the structure so you would, by choosing the S factor you are choosing the overall level of seismic load that you are going to design that building for, it would seem to be inconsistent to choose a different factor for one end of the building as compared to the other end, so...
Q. So there is a problem there potentially?
A. I think so yeah.
Q. Well given that there appears to be that difference in the selection of structural type factor for different parts of the building and as I say Mr Harding can comment on this, if we take that as the case, if we were to assume that the diaphragm connections that remained intact, it would follow from that disparity that the south wall would be likely to yield before the north core, wouldn't it?
A. Yes, yes.
Q. So that the south core would be behaving plastically while the north core could be continuing to be behaving elastically?
A. Yes that is right.

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

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

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

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

A. Oh, it represents a line of zero moment I understand but, ah, so we're talking about anything above the line is tension on top of the beam and anything below the line is tension on the bottom face of the beam.
Q. So are the columns represented above and below that line?
A. Sorry the columns? Oh, sorry, okay, the middle, they're quite small scale but there are some moments in the columns.
Q. We can enlarge those if you like?
A. Is that what you're saying?
Q. Well let's just enlarge the diagrams, both of them please. Can I ask you about the moments in a moment, but just asking you about what the diagram seems to represent and is it right that they seem to represent column height both above and below floor level in those diagrams?
A. Yes they do, yeah.
Q. And according to those diagrams for gravity loading as you say, the columns are picking up a moment at the beam column intersection. Is that right?
A. Yes.
Q. Now look at the particular detailing of the beam column joint that emerged and Commissioner Fenwick has represented this in a diagram, BUI.MAD249.0493.3.

WITNESS REFERRED TO SLIDE

Q. Would you agree there we have column vertical bars passing all the way through the joint?
A. Yes.
Q. We have beam top bars passing all the way through the joint?
A. Yes.
Q. And we have beam bottom bars lapped with a hook into the joint?
A. That's correct.
Q. So do you agree that that is detailed as a moment resisting joint?
A. It, the both, the method of construction did join the beams to the columns in a monolithic way so that when they deform they will pick up moments, both the beams and the columns yeah. I, is that answering your question or not really?

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

WITNESS REFERRED TO SLIDE

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

1456

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

Q. And do you want me to show you or do you – would you accept that 9.4.6 provides equations as to how much of the horizontal joint shear can be carried by the concrete mechanism.
A. Okay, yeah.
Q. Now so my question is we know that there was R6 spirals every 250 millimetres, would those have been sufficient to satisfy those code requirements?
A. I've said in my statement that I think as a minimum you needed the minimum amount required for shear reinforcement. There is a certain minimum amount of shear reinforcement which applied in the columns and also through the joints and anything that comes out of these formulas here would be in addition to that so, but even that minimum reinforcement was not provided so.
Q. So the question if I was to say, was one leg of an R6 spiral every 250 millimetres sufficient to satisfy the requirements I've just pointed out to you, would that be a yes or no?
A. Well there were some other requirements in addition to this one, so, but it was not sufficient to satisfy the requirements of the standard for the joint as far as I'm concerned, yeah.
Q. And Mr Harding I think had said that he intended there would be no spiralling through the joints themselves so it would follow that that could not satisfy those particular requirements if that was the case?
A. That's correct, I mean one of those 9.4.5 if we can expand that again.
Q. Bottom right-hand corner.
A. So I mean the design principle is that the joint shear shall be assume to be resisted by a concrete mechanism plus a truss mechanism comprising horizontal and vertical stirrups or bars, so if there were no horizontal ones the mechanism doesn't work.
Q. Thank you, now one or two questions on cranked splices and I refer you to BUI.MAD249.0284.15, I think Professor Priestley may have referred to cranked splice region of the –
A. In the columns?

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

A. Yes.

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

A. The spiral bars?

Q. Yes.

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

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

A. Gravity and earthquake, yes.

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

A. There could be, yes.

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

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

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

A. Yes it is.

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

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

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

A. Yes okay.

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

A. No I didn't.

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

A. Yes I believe it did.

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

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

Q. Yes.

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

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

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

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

1506

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

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

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

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

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

JUSTICE COOPER ADDRESSES MR RENNIE

Some new matters arose during that.

MR RENNIE

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

CROSS-EXAMINATION: MR REID – NIL

RE-EXAMINATION: MR MILLS

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

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

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

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

Q. Yes but I have to say for myself I wouldn't have read this any differently and this is ultimately I suppose a matter for submission and legal argument but I tell you how I would read it. You tell me if you disagree with this that the later standard is being said to be the master document until such time as the earlier standards are revised and if you've got a conflict or difficulty of reconciliation between earlier standards and this one then, subject to the language where appropriate, this one is the one that controls, others are read subject to it. In other words, endeavour where appropriate to fit around what's in the 1984 standard. Is that how you would read that?

A. That's how I'd read it, yes.

Q. Now whether that is the correct reading will, as I say, be a matter of submission ultimately. Now the only other thing I wanted to ask you in an effort to try to minimise the differences between people where possible is to ask you, you've heard the evidence that Dr Jacobs gave this morning?

A. Yes.

Q. Are there any differences between your view that you've been giving us in your evidence and the evidence you've heard Dr Jacobs give which you regard on these questions of code interpretation as being significant, any differences of significance?

A. I wouldn't like to say that I agreed with all of Dr Jacobs, it's possibly some things were not explained in the same terms I would use but I didn't see any major discrepancies there so I think we need to get more specific if you think there's a difference between us.

Q. I'm not suggesting there is. I'm just trying, where possible, to sweep away any differences that may not really matter in the end in terms of the way these provisions apply which is why I put it in terms of is there anything that you're aware of in your evidence and Dr Jacobs' evidence which you regard as being a difference of significance?

A. Well I'm not sure. I think the particular thing is this clause 3.5.14 in 3101. I've said that I think it would not allow the detailing. I just wasn't clear on what Dr Jacobs' interpretation of that particular clause was, whether he'd had such a, shall we say, definitive view as I have.

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

A. No, no.

COMMISSIONER FENWICK:

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

A. Yes.

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

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

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

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

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

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

Q. How are you defining ductility?

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

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

A. The top one, yes.

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

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

JUSTICE COOPER:

Mr Carter?

QUESTIONS FROM COMMISSIONER FENWICK CONTINUES:

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

Q. It's going to be working twice as hard as say a moment resisting frame where you're going to have at least two going, or structurally you had two walls that were similar were going?
A. Yes.
Q. Do you agree?
A. Yes.
Q. So what should the S factor be? The S is meant to represent the structural form in its ability to dissipate energy?
A. Oh, I see what you're saying, so in fact a higher than one could've been used, or should've been used here.
Q. What I'm saying is should S be .8 or perhaps S should be 1.6 or perhaps it should be somewhere between the two. Do you think there's some doubt about that?
A. Possibly, I mean when we looked at it in the DBH report we evaluated S as one, so.
Q. Forget the DBH report, what's your opinion? What would you do if you were faced with this building?
A. Okay, there is one, I would like to bring up one clause in this loading standard if I could to answer that.
Q. I've got it marked here I think. Yes it's ENG.STA.0018.47.
A. You're going to read my mind, which clause I'm…
Q. I hope that's the right one, that's the commentary on the left-hand side and it's the ray, yes.
A. No it wasn't what I was looking for.
Q. Okay well if you look –
1521

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

Q. It would double the energy dissipation wouldn't it, compensate for having to work twice as hard?

A. Yes it would, yeah.

Q. I mean that's not something I suspect everyone would pick up, but I just wanted your reaction to that and thank you for that. I'm not quite sure if you're agreeing with me or not but that's fine.

A. Okay. I mean if I could just bring up this clause on page 33 and 4203, someone's got a reference for that.

JUSTICE COOPER:

Q. It's 0018.38.

A. So we're just talking about the top if we could just isolate it to 3.1 and C3.1, so the top two.

Q. Top third of the page?

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

QUESTIONS ARISING – ALL COUNSEL – NIL

WITNESS EXCUSED

JUSTICE COOPER:

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

MR RENNIE:

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

JUSTICE COOPER:

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

MR RENNIE:

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

JUSTICE COOPER:

Do you want to tell me after the adjournment?

MR RENNIE:

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

JUSTICE COOPER:

I thought you said it was in the commentary.

MR RENNIE:

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

JUSTICE COOPER:

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

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

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

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

HEARING ADJOURNS: 3.27 PM

HEARING RESUMES: 3.46 PM

JUSTICE COOPER:
Mr Reid.

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

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

MR REID:

Yes as I understand it.

JUSTICE COOPER:

Yes.

MR REID:

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

JUSTICE COOPER:

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

MR REID CALLS:

ARTHUR O'LEARY (SWORN)

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

WITNESS READS BRIEF OF EVIDENCE FROM PARAGRAPH 2

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

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

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

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

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

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

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

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

Q. Just stop you there Dr O'Leary.

MR REID:

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

JUSTICE COOPER:

Yes.

MR REID:

That could be taken as read.

JUSTICE COOPER:

Yes any objection to that course, yes thank you.

PARAGRAPHS 18, 19 TAKEN AS READ

EXAMINATION CONTINUES: MR REID

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

WITNESS CONTINUES READING BRIEF OF EVIDENCE FROM PARAGRAPH 20

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

The loading and concrete standards in the bylaw.

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

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

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

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

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

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

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

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

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

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

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

The CTV building horizontal load resistance was provided by shear walls. The shear wall configuration of the CTV building would have placed it in the category of a moderately eccentric structure according to clause 3.4.7.1 of 4203 in a commentary to that clause. I accept that there is room for differing interpretations of the definition of moderate eccentricity in the commentary to the clause but I believe that the intent of the definition was to limit the amount of torsional resistance provided by elements required to resist predominantly translational modes.

Reasonably regular structures more than four storeys high, even with a high degree of eccentricity, were allowed by clause 3.4.7.1(b) of NZS4203 to be analysed by quite unsophisticated methods of analysis, (the static method or two dimensional modal analysis), although the standard recommends (but does not require) a more sophisticated three dimensional modal analysis. If the two dimensional methods of analysis were used, calculating torsionally induced inter-storey drifts would be precluded. I discuss this further later in my evidence.

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

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

Q. Can I just stop you there Dr O'Leary, you read that last sentence as resisting instead of restraining in the third line of 35.
A. Yeah, it's restrain.
Q. Thank you.
A. This was in my view a widely held understanding amongst structural engineers at the time, and until recently.
A. The corollary of the ability to share the torsional resistance was that if a wall (in the case of the CTV building the wall on grid 1), was highly loaded by translational effects of the earthquake (to the extent of significant yielding), then the torsional load that it may have been resisting would be shared to the other torsional resisting sets of walls.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

JUSTICE COOPER:

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

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

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

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

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

WITNESS CONTINUES READING BRIEF OF EVIDENCE AT PARAGRAPH 46

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

Building inter-storey drift limits.

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

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

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

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

Q.

Drift Capacity of Columns and Column Confinement

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

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

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

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

JUSTICE COOPER:

Q. Was there a column at B5?

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

EXAMINATION CONTINUES: MR REID

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

A. Grid references.

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

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

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

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

JUSTICE COOPER:

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

A. That's correct, yes.

WITNESS CONTINUES READING BRIEF OF EVIDENCE

1621

A. "That is, it was appropriate to detail these columns as "members not designed for seismic loading". This discussion is based on the inter-storey drift –
Q. I'll just stop you there, sorry you read that as "discussion", it should be, "This conclusion."
A. Oh, sorry yes. This conclusion is based on the inter-storey drifts given in Alan Reay calculations at pages S15 and S16. It is not known how these drifts have been calculated but it is likely that they were from an output from the ETABS analysis that I understand was carried out at the University of Canterbury. The output from that analysis has not however been able to be found at this time.
The actual drifts do however seem reasonable to me. The drifts in the north-south direction are approximately five times those in the east-west direction. This may have been influenced by the drifts on grid F being magnified by torsional effects. The drifts may be the maximum drifts from any of the relevant load cases. Torsional effects would have a much lower influence on the east-west drifts.
The circular columns on grid F did not in my view meet the criteria of clause 3.5.14.3(a) of NZS 3101, as they did not remain elastic under the factored drifts (K/SM). They should, in my view, have been designed to clause 3.5.14.3(b) of NZS 3101, (additional seismic requirements of the standard).
The confinement reinforcement in the columns is governed by clause 6.4.7.1(b) of NZS 3101 for non seismic columns. The confinement spiral for the non-seismic columns, shown on the drawing S14, complied with that clause for the non-seismic columns. The confinement spiral for the columns on grid F should in my view have been designed in accordance with clause 6.4.7.1(a) and thus did not comply with respect to either the spacing or the size.
In conclusion I consider that the decision to design the columns as "members not designed for seismic loading", may have been justified according to the standards of the time, except for the columns on grid F

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

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

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

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

Spandrel panel separation.

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

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

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

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

Beam column joints.

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

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

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

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

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

COMMISSIONER FENWICK:

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

A. I think there might be a misprint there.

EXAMINATION CONTINUES: MR REID

Q. Carry on.

A. I consider that clause 9.4.1 should be part of general clause 9.3 and is misplaced as part of general clause 9.4. Even structural members such as columns in very stiff buildings are subject to seismic load reversals and on that basis the inference is that all such columns should be subject to clause 9.5.

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

JUSTICE COOPER:

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

EXAMINATION CONTINUES: MR REID

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

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

Plan assymmetry and vertical irregularity.

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

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

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

Wall on line A.

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

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

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

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

Diaphragm connection.

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

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

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

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

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

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

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

JUSTICE COOPER:
Q. So it is the last sentence.
A. Yes.

EXAMINATION CONTINUES: MR REID
Q. It's the last five words of the second to last sentence plus the last sentence.
A. Well just to confuse the issue there was an incorrect standard number put in there as well.

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

MR REID:

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

JUSTICE COOPER:

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

MR REID:

Yes he did.

JUSTICE COOPER:

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

MR REID:

Yes.

EXAMINATION CONTINUES: MR REID

WITNESS CONTINUES READING BRIEF OF EVIDENCE FROM PARAGRAPH 82

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

Robustness.

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

Documentation.

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

Construction Issues.

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

The concrete strength distribution.

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

Q. You don't need to read that –

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

Lower than expected concrete strengths.

I refer to paragraph 865 above.

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

I refer again to paragraph 86 above.

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

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

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

I refer to paragraph 90 above. S

hell beam reinforcing steel poorly developed in line C wall.

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

Connection at line 4-DE column missing a bar.

I refer to paragraph 92 above.

North core being significantly out of plumb.

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

Vertical seismic separation joints in the masonry infill.

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

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

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

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

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

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

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

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

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

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

A. I meant "could" sorry.

Q. That should be "could"?

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

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

Further observations of the Hyland Smith and expert panel reports.

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

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

Two dimensional elastic response spectrum analysis was allowed for buildings such as the CTV building. I refer again in paragraph 33 of my evidence. The two dimensional modal analysis noted in clause 3.4.7.1(b) of NZS4203 is what a two dimensional ERSA would produce. How the two dimensional ERSA analysis was carried out was that the structure was modelled in three dimensions and the earthquake, then the earthquake input was applied along the two principal axes of the model as two separate analyses.

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

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

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

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

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

A. Yes.

HEARING ADJOURNS: 4.55 PM

INDEX