HEARING RESUMES ON TUESDAY 24 JULY 2012 AT 9.30 AM

MR RENNIE ADDRESSES JUSTICE COOPER

MR RENNIE RE-CALLS
JOHN BARRIE MANDER (RE-SWORN)

Q. Professor Mander, yesterday we were traversing the Commission's questions and we got to the second set of questions on the north wall complex between lines 4 and 5. The preamble to the questions in this wall complex, there were four walls which can provide a lateral force resistance in the north/south direction and one wall on line 5 to provide lateral force resistance in the east/west direction. First question: given the lateral force resistance in the east/west direction what level of ductility would be appropriate in designing the wall and the inertial forces generated between the wall and the floors?

A. Could we have the slide up, the relevant slide please?

Q. Yes we could indeed, that's BUI.MAD493.1.

A. I think that's the one. Okay, so I will talk through how I used – the cursor, oh it's a different look today. East/west direction is this way, up and down on the page, north/south this way, so the main issue is when the loads are coming this way, this whole area is put under shear and shear is not particularly ductile as a ductile mechanism, it's quite brittle but you can get some inelastic response and a fairly modest ductility capability would be inherent in that even with relatively poor details and in the order of about 1.5 to two. The shape of the walls is such also that a large ductility factor, although it could be achieved as per the detailing at the toe of the wall, as if you know it had kind of endless demand on it, if it was, if they were just blade walls or (inaudible 09:35:30) then you would get substantial ductility out of the walls but as a system, as a unit, you know I think you really need to look at this more as a system in which case you would probably get a ductility factor of about three which is most likely less than what you would be designing for. Now I might say though that since the 1980s our understanding over the years has matured somewhat and that back in those days there was a, almost a tacit belief that things were inherently ductile providing you put the good detailing in, but now we realise that you must look at things that are systems and not in isolation, so I think I've covered the first part but just help you if I haven't.

Q. The first part of the question?

A. Yes.

Q. Well on the face of it yes. Did you want the second part of the question again?

A. Yes please.

Q. The second part relates to the inertial forces generated between the wall and the floors. I'll read the question if you like. Given the lateral force resistance in the east/west direction what level of ductility would be appropriate in designing the wall and the inertial forces generated between the wall and the floors?

A. Okay, so I think I answered that one, did I not answer the first question, is that the question number 2?

Q. No we haven't got to question number 2.

A. Oh sorry.

Q. Question number 2 if you'd like it now was –

A. No, I think I answered the first one adequately.

Q. Was what was the load path of the shear transfer between the floors and the wall complex?

A. Okay, so I think there's potentially two load path mechanisms and it depends from which direction the loads come. So if you have a south to north pulse, sorry an east to west pulse, then the inertial forces are going to be in the easterly direction and it's going to be pushing down the page and there will be a shear transfer in this vicinity just from a kind of shear beam type action where this is like the web of a beam and then this beam here, the beam is like a flange so you if you can imagine a big I beam lying on its side the wall, in many people's minds functions in that fashion, and that will work satisfactorily picking up some shear on this, particularly the first wall and then it will get it into this corner region, not too badly, and then beyond here though it starts to become very precarious because where you have a strong region in here, the shear stresses around here are going to be quite high so they'll be prone to early failure and that's why I said it's likely that will only get a very modest ductility capability out of this before the reinforcing steel will show signs of distress, very significantly, and then by the time you get down to here there's really very little hope of transferring the shear into this latter part of the wall here. Now if it's going in the other direction however, if we have inertia forces from the east to the west going up the page, then really the same applies but I think there is a secondary mechanism that will kick in, like what can happen is in the vicinity of these walls or out at the end of the starter bars where you have an inherent weakness, and you have to rely on the slab steel to provide a lot of the shear resistance you will get a tearing and I know there was the other diagram that Commissioner Fenwick used to put up with the red and black, big heavy red and black lines where the floor may tear. That is really a weak plane way out here beyond those starter bars but even if that fails it's not necessarily the end of things because you can, providing the floor plate stays intact, you get an alternative type of mechanism that's based more on the strut action so you get some struts coming into this region in here on a diagonal, roughly somewhere between 30 and 45 degrees from the axis of, which is moving and that is a secondary back-up system. Now that's not something that designers even knew about really at the time. It wasn't carefully thought through, however we now realise in hindsight that that is, it can be used as a primary mechanism if designed for accordingly and it also can be used as a secondary mechanism.

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Q. The third question: would the wall complex warp under the action of the shear transfer? Can you account for the observed vertical cracking in the wall complex?

A. Yes there will be warping, warping torsions in this and so this will, it'll go, you'll get a torsion on this way and that will manifest itself as kind of a corkscrewing effect of the wall, and that corkscrewing effect will indeed put vertical cracks between the, you might say the webs and the flange of the wall in the tower.

Q. The fourth question: what other structural actions are associated with shear transfer from floor into the structural wall complex?

A. Well I think I mentioned that. Like as shown in here you are going to have a shear force and a moment. Now moments can be unpacked and reassigned as force couples and so you'll get a potentially a force couple pulling out at the western-most flange of the tower and you'll get a compression going in at the eastern, the larger eastern wall, it's almost the middle one there, and also to a lesser extent these. Well perhaps quite a bit to the east in fact, so you'll develop the moment quite nicely in that fashion. The shear, as I mentioned, I mentioned that mechanism before in terms of in-plane shear. This is basically the, you consider the yellow floor plate in the diagram to be essentially like an I beam lying on its side. This is a thin web, and it will have quite a decent capability of transmitting shear but the high shear stress intensities tend to be at where these dashed lines are and this is often where you have discontinuity problems and it will show signs of distress. Now if it's well reinforced there the reinforcing will bridge those potential crack zones and alleviate any problems and then it'll just shunt the problem out to where the reinforcing steel terminates, maybe further out here roughly where the M is.

Q. The fifth question: is the detailing of the junction between the floors and the wall complex adequate to resist the shear force and associated action?

A. I don't believe so. I think it will, it, well I should qualify that. Shear is an interesting phenomenon because often in the very first cycle of shear loading you can generate quite a high peak force and that's because that is going from a pre crack to a post crack state. Once cracking takes place, however, you get shear sliding or slipping along a shear plane and the strength deteriorates quite rapidly and it's, after that on subsequent cycles there's little hope. So in some respects this would have a decent fighting chance with taking a big first impulse, but beyond that and on any reverse pulses it's going to really struggle.

Q. Now the sixth question, Professor, related to calculations in respect of the magnitude of shear force. My understanding is that you have not done those calculations?

A. No I haven't done those. I, of course I'm familiar with, loosely familiar with both of those codes. But I haven't done the calculations.

COMMISSIONER FENWICK:

Q. From your answer I take it you've done no calculations on this. This is sort of speculation on what you think might happen. You haven't actually put any figures and numbers to the stresses and forces under these conditions. You're just saying this is what you think might happen, is that correct?

A. Yes, that's correct.

Q. Do you not think there's someone assessing this, it might've been an idea to actually put some numbers on it?

A. I, I felt that I wanted to focus my efforts on the beam column joint because I really feel that's where the distress is. I've put a lot of effort into that Sir.

EXAMINATION CONTINUES: MR RENNIE

Q. And indeed 0.3 of this exhibit is the beam column joint, it's at 0.3. Now you've presented a separate set of evidence in relation to this joint?

A. Yes.

Q. I think your joint is from a slightly different perspective, that's all?

A. Yes it's really a mirror image of what's presented here.

Q. In relation to the issues raised by the Commission in the paper and diagrams that I've just taken to you is there anything you wanted to add to a discussion you presented on this joint?

A. Well, from memory, from Commissioner Fenwick's questions to other people, based on my calculations again this is a very, very interesting problem. Of course one of the key concerns and the key issues is whether or not the bottom hook bars are going to satisfactorily be grabbed, shall we say, and held in position. Now I don't believe that we can rely on classical bond. We don't have to I believe in this sense. There's sufficiently high stresses on the bars and given that you have a reasonably decent radius bend there, there's enough anchorage that can be trapped in that bend zone to do quite well. Now that's if and providing you have a fairly high axial force on the column because in that case the stress block coming across the column is actually wider than shown. This has got a relatively modest axial load, although not in this picture here. In the upper storeys I would concede that the bottom beams are going to have a much greater chance of being pulled out, whereas in the lower storeys which are perhaps more problematic the cracks that are shown on the diagram here are likely to occur and there is a chance of some pull out. But before that happens, on the very first cycle of loading the fixed end moments in the longer spans are going to be somewhere in the neighbourhood of WL squared over 12. So they're kind of the fixed static moments that you would expect to get, and then under side sway you're going to have a negative moment of roughly equal magnitude added to one side and then subtracted from the other. Now when you add that to the gravity moments the positive moment in the beam has a relatively modest moment in it and according to my calculations I actually haven't done it for this particular case. I looked at the case where there was some over strength based and also based on probable strengths but I, and I've done this twice now. Like the first time I did it was for using a lot of the information that came out of phase one of the NTHA, and then the second time I just did it a few days ago based on the most recent information that came out of the more recent and second phase of the NTHA. So I would, seeing those ones are more fresh in my mind I will talk to those. So I'm calculating for the negative moment which as this is drawn will be on the top left-hand side of the diagram as going to, no hang on, it's round the other way, top right-hand side of the diagram. It's going to produce a fairly large moment I'm calculating at the face of a column in the beam it's about

340 kilonewton metres and then on the other side it's only about 44, so it's quite modest. So that means the bottom bar stresses are not all that high. So the demands in terms of the anchorage into that bent bar region is relatively modest. Now this needs a little bit of qualification because under cyclic loading there is a phenomenon known as shakedown, which means that you get redistribution of forces and the moments kind of go backwards and forwards. If you're drawing it diagram-wise it starts up high, comes down and then settles down to some lesser amount and you end up with more or less similar top and bottom moments in the beams. Now when that happens of course there are going to be larger demands on the bottom bars but that will only happen under a number of cycles of loading. The other, so the other thing that I'm finding in my calculations is that when you have a substantially high axial load on there one of the very interesting things that happens is that the joint gets into great difficulty once again under several cycles of loading so I'd like to walk through that. If you take a cut or a free body through the middle of the beam column joint where I've got the cursor right now all the forces that are coming from above can be resolved into basically two things. There's no moment in the middle essentially so we're taking a cut where there's a notional inflexion point in the joint and you get a vertical axial force in here. Now if there was no axial load coming down from above there would be no force to speak of in that joint simply because the shears from the gravity come in as struts down in the corner so there's a strut in here. Because of this modest moment and we're actually not at the maximum moment because there's also a strut coming down here. It seems odd but that's how it is, and so what we end up having is essentially nothing going through the middle. That's the kind of a beam theory would say that, and in fact that might be what you would expect to get on the first cycle but what happens on subsequent cycles is that you're going to have tension here and this has got a switch on this bar, well these two bars, this tension has to switch somewhere from one end to the other into compression and the only way that it can do that is through some form

of bond and anchorage. Now if you start off with a perfect bond that will probably work pretty well and particularly if the demands are relatively modest coming from these beams but the reality is that if you get some beam bonding you're going to get the tension propagating from here and it's going to remain in fairly high tension until it gets into a compression zone at least and then it will start reducing. So later on what this means is that after a few cycles of loading if you take a cut through here you're going to have to resist this gravity load and then all of the bars right across this section, all of those bars will be in tension. So what that means is that if you have to add that force or the total force generated by longitudinal reinforcement of the column which is about 880 kilonewtons if you use just yield stress, the probable yield stress, on those bars. Once you add that to the axial force in the column, then the vertical shear, you might say VJV is extraordinarily high and the concrete stresses are also very high. In fact, and I've done some calculations on those and they are in the order of about 12 MPa just for, this is one of the higher columns I looked at this for. Now what this means is that you're going to get deterioration and one of the things that we've had a lot of debates in the NTHA panel on this and there's two schools of thought and one school of thought is well you can use a (inaudible 9:54:04) circle approach whereby you take the actions at the end of the members and then transform those into the equivalent axial and shear forces and then from that calculate what the shear stresses are inside the joint. That sort of allowable stress approach is fine but it's conditional that the joint essentially remains mostly uncracked. As soon as it becomes cracked that theory is quite inadmissible. So then you have to resort to another theory or some other ideas and so some of the best ideas I think come from truss-type models and the modern truss-type models work on the fact that you have high compression bands and tension bands and within those compression meridian they are weakened by the fact that there are going to be some companion transverse strains which will in turn weaken it. So in this case here we have this very strong band shown via the green lines but out here both the beam steel and the column steel is in high tension and certainly in the columns it's going to be a yield strain. We're going to be looking at a strain of about .002 or more. Similarly down in here. So right across the diagonal through here we're going to have a substantial tension stress and if you use the Collins and Mitchell Theory or the Vecchio and Collins Theory based on modified compression field theory and if you apply that then the stress on the face of it, 12.3 MPa, doesn't sound very high but the fact is that the transverse tensions markedly weaken the joint and then they will reduce the compression strength of the concrete down to even below that. So the question though is how long does it take to get there. Like it's not going to happen on the first cycle but it will happen after a few cycles. So I know there's been this kind of intense debate that we've been having on the NTHA, and in many respects we're both right, I think the others hold to the view that we can use the equations that are in the NZSEE Guidelines which really came from some work that Professor Priestley did some years ago and it's quite old stuff though and I must emphasise that. I think the thinking since that stuff was presented has moved on and in that regard I have been doing a lot of work in that myself and I would refer the Commissioners to Papers 100 and 101 in my résumé which is stuff that is hot on the press right now. Unfortunately I can't give that to you but you can have it for free in about, probably in about a month's time when it comes out in AFCE. I just saw the gully proofs the other day. So we have been doing some work and it's very very similar to this sort of problem for very large –

JUSTICE COOPER ADDRESSES MR RENNIE

Just a minute. Mr Rennie, how long is this going to go on for? Have you any idea?

MR RENNIE:

In relation to the presentation Sir?

JUSTICE COOPER:

Well the witness is responding to a question you asked some time ago which, if I recall, it was something like, "Is there anything else you want to say?" Now it's not really very helpful. If the witness is now going to go on to summarise articles that he has written I think I for one would like to see an end of it.

MR RENNIE:

Well, Sir, it was my intention simply to provide the opportunity to respond to the questions without limiting the witness but on the basis that the Commission is not further assisted, we can simply stop at this point Sir.

COMMISSIONER FENWICK:

Q. Thank you. I mean what you've said there the point of that diagram is it's a different one from the one you analysed and the figures I've got which are actually on the next slide are a bit different from your ones but they come to similar conclusions. The main point behind this was to look at the relative strength of the joint in the column and the point I was trying to make with that slide, which is incorrectly reproduced in a lot of your other diagrams you've got, was that you can look at the force of those, those forces, and see that the joint zone's actually weaker than the column.

A. I agree with that.

Q. You agree with that but I don't think the argument you're using is quite the same but I accept what you've said and that's a valuable contribution. Thank you very much. I don't think we need to hear more about it.

CROSS-EXAMINATION: MR ALLAN – NIL

CROSS-EXAMINATION: MR REID

Q. Professor I'm counsel for the Christchurch City Council and I have some questions for you about the red stickering process that you describe, how it differs from the red stickering process that was undertaken by

Civil Defence following particularly the September earthquake. That's the focus of my questions that the City Council and Civil Defence were involved in the rapid assessment process that you'll be familiar with rather than the owner-initiated process that Mr Coatsworth was involved with. So at the end of your presentation of your second brief of evidence professor you were questioned by the Court about your conclusions.

A. Mhm.

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Q. Do you recall that and I think particularly in relation to item 1 you made a change effectively to item 1 of your second brief. Do you recall that it read as you'd written it, "Older buildings could not be expected to survive the demands exposed prior to and during the 22nd February Christchurch earthquake." That's as you'd written it, and I think you made an alteration to it to say, "all buildings were really in that category." Is that correct?

A. Well when I wrote this it was, I don't know, a couple of months back now I guess and at that time I was unaware that so many buildings were in a state of distress in Christchurch, so –

Q. Yes.

A. – and a lot of this has come out to do with the cyclic loading effects, the cyclic demands and so forth.

Q. Yes.

A. And so I still stand by what I said originally, like, that the context here to be fair was the CTV building and specific I was referring to that era of building.

Q. Yes, so I'm not arguing with you about what you said previously and what you've changed it to, but is your evidence now that your cautious approach would apply effectively to all buildings?

A. Yes.

Q. Yes, and I think did you make a qualification to that to say you were thinking of engineered structures?

A. Yes, although there are many structures that you might say are not engineered such as masonry structures but they're recognised as structures where the public congregate and meet and so forth and I think the profession owes it to the public to make pronouncements to be helpful in regard to those as well.
Q. Yes. So the category of buildings that you would place into the category of guilty until proven innocent would be all engineered structures plus all masonry buildings, is that correct?
A. I think so, yeah.
Q. And engineered, by engineered structures do you mean structures where an engineer has been involved in the construction of the building?
A. Well, design.
Q. Designing the structure of the building.
A. Mmm.
Q. So would that, on that basis apply to many residential homes?
A. It may do. There are residential homes where there are as you well know in the Christchurch area where the City Council would require engineered pieces to be done. Sometimes it's as simple as a lintel on a garage.
Q. Yes.
A. Other times it's a retaining wall that needs to be put up and one needs to look at the safety of those sort of objects as well. You can't always take for granted that everything’s going to be okay.
Q. Yes.
A. Little did we know that there were many rocks ready to come down and I'm sure they would have been loosened up in the first earthquake, so those are not engineered objects but they're, again engineers can be usefully employed in making observations and pronouncements about those sort of things.
Q. Yes. Well on the basis that most of the CBD or all of the CBD really is engineered, it would be closed.
A. Yes.

Q. Is that correct?
A. Yes.
Q. And am I right to think most of Christchurch's suburban centres would also be closed?
A. Yes.
Q. Would that be –
A. Yes.
Q. And did I understand you correctly that those residential buildings that have had some engineering involvement in construction would also need to be closed?
A. Possibly, yeah.
Q. And over what sort of radius would we be talking about in this sort of context?
A. Well again that's the part that's done by a desktop analysis. It's possible nowadays immediately following an earthquake that information streams in from Geonet and so one can get an attenuation relationship based on the shaking intensity with radial distances say from the epicentre quite quickly, and then using that information you can go by a building category. Now I still feel that there is some qualification needed here because you will be well aware that there are the four Rs of response, readiness – reduction, readiness, response and recovery and what we're really talking about here is not the response and recovery so much which is after the fact, but more to do with the readiness. So if you have a plan that when a building say if there's going to be an earthquake at this location, and if we know the shaking is of this intensity, then our building could be in trouble and that sort of information I think could be held perhaps by the Council. I think indirectly that is there but I don't believe it's in a very useful form in that there's a lot of the percent NBS information is available but I don't believe that's particularly useful for this call because there are structures that will – it all comes down to does it pass or fail this 34 or 33 percent bar and I don't think that that's a very accurate way of dealing with the problem.

Q. But do I understand you to say though that at least in the first instance, the approach would be that you adopt a guilty to proven innocence approach?
A. Yes.
Q. And you shut everything down for a start?
A. Well I think this is more or less what's happened in Christchurch subsequent to the initial earthquake and again as I have said, yesterday I – my feeling is that had we had the foresight that this could have been a major second event, we probably would have done that but we didn't.
Q. Well what happened in the CBD after the September earthquake is that it was closed initially and then there was a process, a rapid assessment process.
A. Yes.
Q. Where buildings were stickered based on a damage assessment process and what you're suggesting is something quite different to that, isn't it?
A. Well not really, I'm saying that's not sufficient alone because people looked at the CTV building and they looked at it without plans and then said, "Oh, she's fine, can't see anything."
Q. Yes.
A. But behind the innocent looking skin and façade there must have been damage of sufficient nature to cause alarm because the alarm bells were raised by the occupants on many occasions, but that message was not adequately conveyed upstairs shall we say.
Q. So you're really talking about the introduction of an additional step?
A. Yes.
Q. To that that was undertaken after September.
A. Yes.
Q. Whereby a desktop analysis is undertaken before damage based assessment. Is that correct? Is that fair?
A. I think that would be prudent to do that, like, they should go hand in hand and immediately you would send some inspectors out to look at

the ones that are most problematic because again some of these looks can be deceptive.

Q. Yes.

A. It's interesting to me how you know after many months libraries can be closed and everybody thought yesterday they were okay.

JUSTICE COOPER:

Q. I'm sorry, just so I'm following you, a desktop study would have shown that the CTV building was problematic. Is that what you're saying, after the 4^{th} of September earthquake?

A. Is that my question, sorry.

Q. Well yes.

A. I believe so. If an engineer had the plans they would have noticed that it was largely devoid of a lot of the modern detailing that one would be pretty well sure to have in the beam column joints and the columns, the detailing that leads to robustness and it would cause one to give pause about at least downgrading this from red to yellow. Maybe you could go as far as yellow if it was deemed to be reasonable but again one should be loathe to go to green.

Q. And that's a conclusion that you say could have been readily reached after the 4^{th} of September earthquake?

A. For the class of building yes.

Q. Well what's the class of building?

A. Well other ones like it. I understand that we're going to be hearing from John Henry who has designed other buildings like this before and possibly after the CTV. It seemed to be a common form of construction that was well known to exist in the day and it seemed to be –

Q. So the class is the building - is addressed by Mr Henry.

A. Yes and well it would have, I understand the City Council now have or maybe the region, I don't know if it's Christchurch City alone, but there are indicator buildings and there may be an indicator building, now if you choose the right building as an indicator, as it's very difficult to know which one to choose but there will be an indicator type of the CTV type of building. It could have been a different one but an examination of the drawings and a closer look would have gone a long way to help identify that there are problems.

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Q. Do you know what the indicator buildings were?

A. No I don't.

COMMISSIONER FENWICK:

Q. It was a shear wall building so this, your type would include all shear wall buildings would it?

A. I don't believe that you can look at a building and I think this may have been part of the problem. If you look at a building and say shear wall building fine because the displacements are going to be small, that's exactly been the problem here is that it's in part a shear wall building but in part it's a moment frame structure that doesn't –

Q. No if it didn't have moment frames in it it relied on shear walls you would say it would be okay is that right, if it was a modern building just had shear walls and –

A. No, no, not at all.

Q. So we include then shear walls in your category. Now is there any category of building which is not covered by a description you've given? The answer yes or no will do.

A. I would say no.

Q. So all buildings are covered? All buildings that have been red stickered?

A. Well no, well yes but can I qualify this please because I did say under the readiness thing if plans can be gotten together ahead of time under the guise of readiness the owners and also the regulators should know for a category of building or even specific buildings if these buildings are getting into a danger zone and then knowing that it should be possible to make pronouncements about their ongoing safety or otherwise.

JUSTICE COOPER:

Q. That wasn't the case in September was it? You're talking about an ideal system I take it.

A. Absolutely yes I am.

Q. But in the meantime you've said that the CTV building should have been red stickered and if I follow the answers you've given to Commissioner Fenwick correctly that would apply to all the buildings in the CBD and also to the buildings in the commercial centres in the suburbs. Is that right?

A. Yes.

Q. And that would have been pending the carrying out of extensive investigations which would have taken what period of time do you think?

A. Well a lot of the buildings could be dismissed from the list very rapidly because it would be if one can show that the detailing is adequate and the performance is likely to be good and the excitation is deemed modest coupled with at least a reasonable inspection at least what the and I think a level two inspection is preferable to a level one inspection but some buildings may be permissible to carry out a level one inspection but others level two.

Q. Is this the rapid assessment you're talking about because that's what was carried out.

A. Yes I realise that but level two is more than rapid. It's a little slower. So the problematic buildings what I'm really saying is they should have a rapid, a level two assessment along with at least a look at the plans.

COMMISSIONER FENWICK:

Q. And this earthquake sequence there were four major earthquakes that went on for 16 months so you would have red stickered Christchurch for 16 months? After each one you'd want a detailed assessment of the building to see what damage you had?

A. No because I think after that pattern if the other ones are smaller then you have some idea at least that what the ongoing damage is. You can look at the growth of that damage and I understand that's what's been happening with the indicator building is to look at the progression of the damage, and so you're looking at two things really the quantum of damage and also the increment of it from earthquake to earthquake.

Q. You were saying in the CTV you could not see the damage. You would have to find that by analysis.

A. That analysis can be quite simple. Like when people say that it doesn't mean you have to do a NTHA that's going to take months to do. That can be done in probably a couple of hours.

Q. Sorry it can be done in?

A. Well it could be done in a couple of hours because you know –

Q. Professor Mander, how long would it take you to look at the CTV drawings before you could fully understand the way it worked and all of the details in it. I don't, you know could you do that in a couple of hours and do the analysis?

A. Well it doesn't take very long to look to see that the details are not all that flash and then do the calculations. They can be done. There's simple ways of doing this actually and looking at T joints what sort of drifts they should be able to sustain and so forth and then try and relate that back to the excitation, the intensity that's been exposed to.

JUSTICE COOPER:

Q. You seem to be saying that the CTV design was one that was obviously inadequate. Am I understanding you correctly?

A. No I'm not saying it was obviously inadequate. I believe it was adequate for what was believed to be the design demands for Christchurch in the day and –

Q. But that after September it would be obviously problematic. You could reach that conclusion after a couple of hours' work?

A. Yes. They would have more or less met its Waterloo at that time.

CROSS-EXAMINATION CONTINUES: MR REID

Q. The proposition though that engineers would have been able to determine that a design level earthquake had occurred in the days following the September earthquake is problematic isn't it, because you're saying that now two years after the event. Mr Priestley or Professor Priestley and at least I think Dr Holmes as well disagree. There's no consensus about whether the September earthquake was design level. Do you agree with that?

A. They may not think there is but I thought that we've shown with my presentation yesterday on two counts that it more or less meets and I said it's essentially the same. It doesn't have to be the same. It essentially meets design level and it's very close, so that's my view.

Q. It's problematic though isn't it in the context of an emergency to make those kind of determinations, wouldn't you agree with that?

A. No. Dr Bradley who's sitting back there, he did the analysis double quick time and we had a lot of information available. Now he volunteered that, it wasn't required, but one could see that the level of ground shaking is very substantial and based on that alone it's very easy to do a lot of mental arithmetic in one's head. An engineer carries these sort of numbers around. So for example if you know what the peak ground acceleration is you know what, you have a spectrum in your mind's eye, you know roughly what's going to happen for a one second period structure and it's from that one can mentally work out what the demands are on based on the spectral displacement you can then work out what the demands out to the floor then you can look at the drawings and say hey can these drawings get this or not and you would look at the CTV ones and say wow goodness these don't have particularly robust details. We better take a second look and that's basically what I'm saying. So be cautious and not boldly go in and say she'll be right mate, let's just move on.

Q. All right well are you, just moving on to a slightly different topic, are you familiar with the New Zealand standards, New Zealand Society for Earthquake Engineer Guidelines?

A. I should be I guess. I was the co-author of it.

Q. Right. And are they based as Mr Kehoe says on the ATC20 document?

A. I don't believe so. I think we got a lot of guidance from ATC306 and 307. There is a little bit of ATC20 in it. We got information from a lot of sources on that one.

Q. All right well Mr Kehoe says in relation to the New Zealand guidelines and this is at 3.16 of his evidence. I don't need to bring it up. I'll just read it to you. He says, "Implicit in the methodology of post-earthquake safety evaluations is that immediate hazards and unsafe conditions are visible, physical conditions as opposed to numerically calculated conditions." It's his view about how the standard, how the guidelines work and the basic assumption of them. Would you agree with that?

A. No.

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Q. No, why?

A. Because that's outdated information that he's working on. He's basing that on something that he and I wrote actually in FEMA 306 that was brought up. I was also a co-author of that, and the world's moved on since then.

Q. Yes, but what I'm talking about is the New Zealand guidelines as they currently stand –

A. Yeah.

Q. – they're based on that assumption aren't they, whether the world's moved on since then is a different issue?

A. Okay, I'll concede that.

Q. And that's the assumption, and how they work in practice is that the rapid assessments level 1 and level 2 are directed at identifying damage aren't they?

A. Yes.

Q. So on your system that you're now suggesting where there's a desktop type analysis of all buildings and they're categorised as red or not red on that desk top sort of basis –

A. Yeah.

Q. – that the rapid assessment process would be completely redundant wouldn't it?

A. Absolutely not, absolutely not. No there's a false premise in this line of questioning is that you can, you have to see things, and my point is and I thought it was clear yesterday that there are certain types of damage that are hidden and the CTV would've had a couple of those. There's longitudinal reinforcing bars would've been strained substantially so regardless of what cracks would tell you, even if you could or couldn't see them, there's damage done to the steel first of all. Secondly, there's going to be hidden damage in those connections and that could not be seen by virtue of how the structure was constructed. You have these beams that frame together and with a circular hole where the joint is the concrete would have been placed in there, it would've shrunk and the cracks would be contained within that. You wouldn't know what's going on inside that. So again a thorough look at the plans, even a cursory look at the plans would indicate that.
Q. But the rapid assessment process doesn't envisage looking at the plans or doing any calculations (inaudible 10:22:50)
A. No well that's a pity I think in my view.
Q. Yes.
A. Like I think to do that without the plans, particularly in this day and age where we have all these wonderful electronic media, it's not too much beyond the pale to have this either loaded or have it wi-fied where you can go round the streets and bring the plans up. Now a lot of jurisdictions do that in the United States, I know that. Corpus Christi is a case in point. They've been doing this for over 10 years.
Q. Yes but again you're describing an ideal process aren't you, rather than the process that actually occurred?
A. It may be ideal but in New Zealand's minds, but in other jurisdictions it might be considered to be essential.
Q. Yes, but in terms of the process that's set out in the New Zealand standards as was followed in Christchurch, in the New Zealand guidelines as was followed in Christchurch, what you're proposing something different to that isn't it?
A. No I'm saying this is a supplement and addition to it.

Q. And addition?

A. What I'm saying is it's essential that you do that but it's not sufficient. It's, let me repeat that. It's essential that you do a visual inspection, obviously, but it's not sufficient in itself alone.

Q. Yes but your criticism though is a criticism of the process described in the guidelines that don't contemplate the calculations and review of the plans and so on, that would be necessary in your view.

A. It's a criticism of the guidelines?

Q. Yes.

A. Well I suppose it is, yes.

Q. Yes.

A. I don't mind criticising my own work.

Q. No. And the reason that I'm asking these questions is because for the inspectors on the ground in Christchurch at the time in the days after the earthquake they were following a process that was set up by civil defence under the guidelines and diligently doing their best in accordance with them. Would you agree with that?

A. I totally agree with that but I –

Q. Yes.

A. – I would surmise that many of those would be yearning for more information.

Q. Yes, but it's unduly harsh, isn't it, to criticise those people?

A. I'm not criticising the people.

Q. Well, I'll just take you to your evidence at page 5. This is really what I want to ask you about. So at page 5 you're saying – this is, sorry this is the first statement of evidence. This is where you're talking about the red stickering process and you're talking about then what the inspectors should've done, and you're saying that, you say at, well the fact that there had been a design level earthquake, to summarise, "…should have served as a signal that substantial inelastic response would have occurred, whether seen or unseen, and it was therefore concerning that inspectors did not immediately red sticker the CTV building."

A. Okay I should clarify that. I wasn't referring to the civil defence ones because a lot of those people aren't engineers if I'm not mistaken?
Q. No that's correct.
A. Yes I don't mean those at all. I mean the second pass through which is perhaps the one that Mr Coatsworth did. That sort of level, the engineer inspection, that comes later.
Q. So just to clarify then, your criticism of the process set out in the New Zealand earthquake guidelines, and as followed in Christchurch, it's, your criticism is it's of the standards rather than the inspections carried out on the ground?
A. I'm not critical of anything that was done on the ground, and even Mr Coatsworth, I believe that he did the best to his abilities at the time with the information that he had, but it was not sufficient to show that there was hidden damage there.

COMMISSIONER FENWICK:
Q. Were you here in September?
A. No I wasn't.
Q. You didn't come out?
A. No.
Q. You didn't offer any advice after the September earthquake or make any comments about the stability or the damage which occurred in Christchurch after September earthquake?
A. No, no.
Q. Except in the one paper report? Or one report?
A. Yes, that's right, there was a reporter from the New York Times that called me, yeah.

CROSS-EXAMINATION: MR MILLS
Q. Now Professor Mander I think this issue got clarified finally yesterday but I just want to confirm that my understanding is right. And it relates to this rather unusual description of evidence by an expert witness as a

submission. Now I take it, what you told us yesterday is that it was your decision to call it a submission?

A. Yes.

Q. And in that you, I gather, got some advice from your lawyer daughter that that was an appropriate term?

A. Well she, she didn't know and we didn't actually know that at that time that I would be coming here. Whether or not it wasn't firmed up, and so she said to me that submission is the right thing to call it if you're going to go you can walk away from it 'cos it's submitted anyway?

Q. Yes. Now I think I'm also right, but just confirm this for me, that at the time you wrote that submission that your intention was to defend it in the academic sense?

A. Well I don't think those were my words. I believe Mr Rennie said that but this is not too different to what we do as academics.

Q. Yes, and defending a paper in the academic sense, just, I just want to take you through what that means, tell me if you think this is right. That if you're defending a paper in the academic sense you're defending it really before your academic colleagues, would that be right?

A. Yes.

Q. And it would be typical to take up a position and then defend it?

A. Yes.

Q. And in defending it one would typically take up the techniques of argumentation and debate in order to defend the position?

A. Not necessarily.

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Q. How would you describe it?

A. It depends on what you're trying to do, like I don't believe that we can have intellectual debate admittedly but I don't believe that this is what we're doing here is an intellectual debate. It's - the field of engineering is both an art and a science and it sort of acts at the intersection of those two parts of the discipline and often times we switch caps, even within one piece of work, so.

Q. Which you agree with me that there are a number of aspects of your submission, now evidence, where the approach that you have taken really does reflect an approach of techniques of argumentation and debate designed to fend off and defend your position?
A. Possibly.
Q. Yes, well I think it's more than that isn't it, and I'll give you an immediate example which is in both your submission and also put right at the forefront of the power points you subsequently developed. I can take you to it if you like, but I think you'll probably remember this, and that is the initial quite dismissive criticism of the Hyland Smith report based on what they had said in the executive summary. Remember doing that?
A. Yes I do.
Q. And within a few lines of doing that and having treated it so dismissively you then acknowledged that if one reads into the body of the report that in fact the executive summary is just that, a brief summary of the substantive points that are made later?
A. Ah, yes.
Q. And the purpose of setting up what I would call a straw man and then knocking it down, that is typical of the methods of argumentation and debate that one would take up in defending a position. Do you agree?
A. Possibly.
Q. Now your submission itself is undated. I see that the statement of evidence which attaches that submission is dated the 10th of June 2012. Now when were you first advised that your submission, now evidence, would have to comply with the Code of Conduct for expert witnesses?
A. Probably fairly early on. I think that would have been in April or thereabouts.
Q. Are you saying that at the time you were preparing this submission that you were conscious of the fact that it was required to comply with the requirements of that Code of Conduct?
A. Yes.
Q. So you didn't at any stage feel when you looked at that expert witness code and considered things such as the straw man I just referred you to,

that that didn't sit very comfortably with the obligations under the expert witness code?

A. Well on reflection I don't think I was all that sure where the DBH report sat. I don't believe and I kind of had it in my head at the time that it wasn't commissioned by the Royal Commission, it was a separate body that was put out by a different sector of the Government.

Q. And did that justify taking an approach to it which was not consistent with your obligations as an expert witness. Is that what you're telling me?

A. What I felt was that it didn't have a balanced view and so I was really trying to, and I'm trying to be constructive in that I wanted to offer another point of view on several points.

Q. So you were going after it, I take it?

A. Not in that sense, like there's a lot of things as I've said in there that you can – are quite useful but there are a lot of things that I don't agree with, and I – well I guess it's just me. I don't like mincing words.

Q. Have you ever given expert evidence before to a Court?

A. No sir.

Q. I'm just going to remind you of the obligations that you're now under in giving expert evidence because I'm going to ask you a number of questions about things that you've said, and it is important for the Royal Commission that it knows that when you answer these questions that you are very consciously aware of the fact that you're not here as an advocate and that you are constantly conscious of the role that you're playing here so I'll just ask that the expert witness code be brought up. It's BUI.MAD249.0529.1.

WITNESS REFERRED TO CODE OF CONDUCT

Q. So there it is there. Now the ones I just want to remind you of before we move on is first of all that you have an overriding duty to assist and here it's the Royal Commission impartially. Secondly, it's the second limb of that first point that it's on relevant matters that are within your area of expertise. So that's what you're here for, to give evidence within your area of expertise and at least when opinions are involved only within your area of expertise. Understand that?

A. Yes.

Q. Now if we could just go back to the document itself. When you give an opinion you'll see there in 3(d) you're required to state the facts and assumptions on which your opinions are based, see that? Three (d).

A. Oh sorry, D yeah.

Q. State the facts and assumptions on which the opinions that you're giving are based.

A. Yes.

Q. And then under E the reasons for the opinions that you're giving. So that's your role here, and so now I'm just going to ask you some questions where I think you now, if not before, will have that very much in the forefront of your mind I hope in answering them.

Now first of all the question of existing relationships that might affect your impartiality. You have quite fairly and properly disclosed that you have an existing relationship with various members of Alan Reay Consultants Limited, haven't you?

A. There's only one that is in there now, Chris Urmson, yes.

Q. Now, but you've had a previous relationship through your son's time with Alan Reay Consultants Limited?

A. That was a three month summer holiday job.

Q. Yes. All right, well you've properly disclosed those things. Now what about your relationship with Dr Reay, how long have you known him?

A. I've only known him by name and up until maybe about 10 years ago I remember meeting him at a committee meeting we had here in Christchurch when I was at Canterbury University, and then I saw him at a couple of other local meetings that was about all and I guess we knew each other by name and that was about it.

Q. All right, okay.

A. So we were acquaintances more than –

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Q. Yes. Now can I take you then to some of the statements that you've made in your evidence which potentially seem very sweeping and I just want to examine them with you. We could go first to your evidence and it's WIT.MANDER.0001.44. Now you'll see that you say there under 1.1, "The CTV building was designed and construction in compliance with the applicable design and building codes ....." and so on. That's the part I want to just focus on for the moment. Your opinion, which is what that is, that the CTV building was designed and constructed in compliance with the applicable design and building codes. Now I'm not sure exactly what you're saying there so I just want to try and clarify this first. Are you saying that in your opinion the CTV building was in all respects code compliant at the time it was permitted?

A. Well if my understanding is correct then if the City Council gave a permit to it, then it complied. So everybody, every engineer knows that in spite of what may be permitted there is inevitably going to be errors and omissions in every job and some of them are small and some of them are large and some of them come to light later on.

Q. So you're not expressing here any independent view of whether it was designed in a code compliant manner?

A. No.

Q. You're just saying Council gave it a permit, ergo, for my purposes it's code compliant. Is that what you're saying?

A. Yes.

Q. And then you say that it was constructed in compliance with the Building Codes. What's the basis for that opinion?

A. Well that really is connected back to the previous phrase about given the permit. It was like it was designed in accordance with the code. The calculations were done, now whether or not they were correct it's not for me to say I feel. Others are going to comment on that. I don't hold a strong view either way on that and I realise that it comes down to the minutiae of how one reads parts of the code. My own personal view is I wouldn't have designed it quite like it is. I wouldn't teach students to do it like that either but that seemed to be a fairly common practice so

evidently the Council must have thought that it was okay at the time and that's my view.
Q. All right well we'll come back to some of those issues. So we've agreed that you're not expressing any independent view on compliance at all?
A. No.
Q. Now you say the question of being constructed in compliance with the Building Codes is really part and parcel of that same point but it's not is it. The issue of how a building is designed and whether that is compliant and how it's built are quite distinct issues aren't they?
A. They can be. Obviously back in those days, as I recall, there would be building inspectors in Christchurch that would come around and make spot checks to see that things were being put in in accordance to the drawings.
Q. So once again, despite the fairly emphatic language you've used here, you're not expressing any view of your own about whether it was built in accordance with the permit plans?
A. Ah, yes that's correct.
Q. That's correct that you're not?
A. No I'm not.
Q. Now are you aware that in the Hyland Smith Report that they identified a list of construction defects?
A. Yes.
Q. And you're aware that Mr Frost in his evidence also identified construction issues of concern?
A. Yes.
Q. And in fact the note I made of something you said in your evidence yesterday when the question of the pre-cast connections with the in situ connections, this issue of the smooth ends on some of the beams was that you said it was possibly not all that fantastic?
A. Absolutely.
Q. So we know then don't we that in fact the building was not built in accordance with the permit plans?

A. Ah, for the most part they conformed and again I would surmise that this was inspected by the City Council and they would have considered this to be satisfactory.

Q. When you say for the most part it complied, what's the basis for that statement?

A. Well there is one thing that I am particularly concerned about and I think it would be a major construction difficulty is the presence or absence of the spiral reinforcement in the beam column joint. Now, as specified, I believe it should have had a spiral, admittedly the pitch is very large so you would only get about one and a half turns of the spiral, well maybe two turns at best, within the joint. That's really not sufficient to do much but nevertheless it was there but from the forensic evidence it's hard to see any clues that there was some steel. You would see even if the concrete turned to rubble in the joints it would probably leave the steel behind because it has left the longitudinal steel behind, you would see remnants of that. You would probably see remnants of the spiral and it may well be, I don't know about this, but it may well be that the way the beams were interlocked and then the order in which the concrete was poured there was no opportunity to actually put the steel in that joint and it was buried before it happened.

Q. Yes, so you're telling me that you think it likely that that's another area, a significant area, where the as built didn't comply with the permit process?

A. That's possible, yes.

Q. Probable?

A. Ah, well I suppose probable is more than 50% possibility. Is that what you mean by that?

Q. Yes.

A. I think that's possible that is probable.

Q. You think it's probable?

A. I would say it's more than 50% likelihood.

Q. That's based on your observation of the relevant facts?

A. Yes.

Q. Now your refer in your evidence in a number of places, and I won't take you to all of them but I'll give you the references so that they can be noted and I'll take you to some of it if you need me to, to the absence of the drag-bars on levels 2 and 3 and you say in your evidence that the absence of the drag-bars on levels 2 and 3 played a key role in your collapse scenarios. Is that accurate?

A. Play *a* role. I don't think the major role. I think the structure would have fallen down perhaps without those but the fact that they're not there is not helpful.

Q. All right well let me check your evidence on this. The first of the references, I won't take you to all of them but I'll just note them for the Commission really, the first of the references is at page 53 of WIT.MANDER et cetera, 53 and then page 81 you refer to, you're talking about your collapse hypothesis with a north-south direction and saying that the north-south direction has gathered much discussion by others which all refers back to perceived inadequacy of the drag-bars, the lack or failed drag-bars would be affected by a northward pulse and so on. Do you agree with me it's getting some emphasis in there?

A. Yes, the presence or otherwise you mean?

Q. Yes.

A. Yes, yes.

Q. And then at page 86 in relation to your southward mechanism you refer to the fact under "The Trigger" you say, "Irrespective of the merits of whether the drag-bars had sufficient capacity to restrain these inertia forces, the fact remains that there were no drag-bars in the lower storeys. Such lack of restraint permits the lower level floors to move relatively freely southwards…" and so on, so again I take it you're attaching some real significance to the absence of the drag-bars on levels two and three?

A. Yes.

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Q. And then again you've got a reference to this in the drawing that you've got under figure 3.4 on page 87 where you say under your step one that a southward collapse mechanism, "Due to the absence of drag bars in the lower storeys there is a large strain demand placed on the slab steel. After one or two cycles the bars fracture due to low cycle fatigue." so again do you agree with me that the absence of the drag bars is seen as significant in a number of your different scenarios?

A. Only that one.

Q. Only that one?

A. Because the east west is really the dislodging of the connection on the west wall and the north one it doesn't require the crumpling up scenario so it's only the southward pulse one.

Q. All right. But in relation to that it's significant?

A. It, yes but basically the slab steel can break somewhere, anywhere in fact.

Q. But the absence of the drag-bars on your analysis for that scenario is significant?

A. Well on reflection it could still go in spite of the drag-bars. It could still fracture out in the slab steel more near one of the joints too.

Q. Do you agree with me that at least in your written evidence it's significant?

A. I do agree with that yes.

Q. Now I want to ask you more generally about the diaphragm connection and get your view on this and the note I made during the course of your evidence yesterday was that in relation to the diaphragm connection you said "…everyone agrees it's a problem. There are errors and omissions all around." Do you remember saying that?

A. Vaguely yes.

Q. Now what's your position on the north core floor diaphragm connection. Put the drag-bars off to one side because they weren't there at the time of permitting. At the date of permitting do you have a view on whether that was code compliant?

A. Well I do have a view that as I said before that if the council said it's got a permit then it's compliant as far as they're concerned and the registered engineer would have designed it at the time and he would have considered it to be compliant. The fact that it may have had undisclosed errors and omissions that came to light later on goes beyond that point in time in my view.

Q. And so your view on whether it was code compliant, do you have one?
A. I would prefer to just say that if the council considered it to be okay, that's okay.

JUSTICE COOPER:

Q. The question is whether you have a view. That's the question.
A. I'll just say that I'm not going to give a view on that.

CROSS-EXAMINATION CONTINUES: MR MILLS

Q. Do you not have a view?
A. I'll say no I won't.

JUSTICE COOPER:

Q. Just a minute. You can't say I'll say I won't have a view. Do you have one or not?
A. I don't have a view on that.

CROSS-EXAMINATION CONTINUES: MR MILLS

Q. So you've not considered despite all the time –
A. I have considered it and it's very troublesome to me.
Q. Troublesome in what sense?
A. Well I must say and going back to your earlier points about I might sound emphatic about how I've written it but that's really to say how I saw things transpire at the time and, but to make a judgement as to whether it was right or wrong at the time, I don't have the expertise to say that.
Q. Are you not familiar with the New Zealand code requirements?
A. Of course I am familiar with them but I'm not as a practitioner and in those days I was newly graduated and I wasn't actually practising in that

particular area then although I was very familiar from my studies at both the undergraduate and graduate level of the codes.

Q. You've expressed some fairly emphatic views about a number of issues about the design of the building. From an academic perspective have you not looked at the code and looked at the drawings and formed a view on whether or not that diaphragm connection was compliant?

A. Well the trouble I have with this is that different people feel that it is and –

Q. And what's your view?

A. And my view is it wasn't sufficient. I would have done more.

Q. Yes all right so your view then, do I take it, is that the diaphragm connection did not comply with code at the time of permitting?

A. Well I think this is getting back to this best practice versus compliance bogey actually.

Q. Well let's take it in two parts. How about the first part? Is it your view that it was or was not code compliant at the time of permitting?

A. It possibly wasn't and it certainly wasn't best practice.

Q. And what is it that makes you equivocal and to use that word possibly, and bearing in mind what your role here is as an expert witness?

A. Because I'm relying on what I, partly on the views expressed and even in those days there were, I do know, I am aware that there were various views as to how one interprets that, interprets the code, and some would interpret it in a very legalistic sense and others would take a very liberal view of how to interpret the code, and I believe that this has been a very liberal interpretation of the code and in the minds of the designers at the time this would have been fair game and it doesn't mean to say I have to agree with it – I don't.

Q. So you don't agree. How do you know what was in the mind of the designers at the time?

A. Well we can look. It's evident by what they produced.

Q. So let me just get my head around this. Are you saying that we can reason back from the resulting design and say that we can conclude

that the designer deliberately and consciously produced the design knowing that it was potentially not compliant with code?
A. I don't believe a designer would do that. They can make unconscious errors of omission. That, the way you pose that that would almost like saying they consciously did it. I don't believe that they did that, no.
Q. Do you think there might have just been a mistake here in which the way the design was done?
A. I looked through the calculations and I say they're moderately hard to follow. The handwriting's a bit scruffy and it's difficult to follow but I think it was executed very similarly to how we were taught to do design at the University of Canterbury for example. It's quite similar, so there's a standard pattern of how you carry and execute all the calculations. That was quite evident there and carried through all the checks, and those were done, so again it comes down to this trigger of how you read clauses and put different nuances on them and different people do things differently.
Q. So on this these two camps that I think you've described of those who take a liberal interpretation of the code and those who take a more conservative interpretation of the code, which camp are you in?
A. I'm a professor and so I have to teach design to students, that's one of the things I do, and I believe in exercising caution and if in doubt I typically say and I tell students because I know this is tug of war between the developers and often when they are really trying to get down to rock bottom price I always tell them they'll argue with you about the littlest things about whether these hoops should have been there and I always tell my students about whether these hoops should have been there and I always tell my students what is a hoop box, two bucks a piece or whatever, $1 in place, $2, you multiply that through the building, it's the difference in the cost is in the noise of contracting.
1100
Q. Mmm.
A. And, but of course people are often hungry to get jobs and make sure that they keep repeat business and this was a tough time to do business

back in the 80s and so I think everybody was operating with very, very sharp pencils as it were and stripping out everything that they believed wasn't necessary and my own view is not to do that. I would not teach that to students because I feel that more steel if it's rightly placed is better than less.

Q. Yes, all right. Now just to finish that point having dealt with the code compliance issue, you're I take it quite firm that the diaphragm connection to the north core does not represent best practice?

A. I don't believe it does, no.

Q. Now were you aware of the fact that Mr Geoff Banks who we'll be hearing evidence from later in this enquiry, who was at the time an employee of Alan Reay Consultants Limited, were you aware of the fact that he was the one who had a primary role in the drag-bar design?

A. The redesign?

Q. Yes.

A. Yes.

Q. And were you aware that he has advised counsel assisting in response to an information request that in his view the diaphragm connection as permitted was not compliant against 1991 standards. Were you aware of that?

A. It's the 91 standard, yes.

Q. And were you aware that he also has agreed that there were no relevant changes in the standards affecting the diaphragm connection between 1986 and 1991?

A. Yes.

Q. All right, so you're aware that he at least says non-compliant –

A. Right.

Q. – at the time of permitting, you're aware of that?

A. Mhm.

Q. Now you were here when Professor Priestley gave his evidence weren't you?

A. Yes, yeah.

Q. You probably recall that I think in response to a question from me and I can give the reference to it to the Commission in a moment, that the lack of design connections, speaking about this issue of connection to the north core, “Was very remarkable.” Now you associate yourself with that view of it, the lack of design connections, “Was very remarkable.”

A. I never thought of it as that at the time. I guess my mind has been more fixating on the beam column joint region. I feel that that's more problematic.

Q. Yes.

A. But I do agree that it's – I would say it's remarkable, not very remarkable.

Q. Yes. Now the reference to that, just for the Commission is it's the transcript for day 57, at page 54, Professor Priestley’s comment to that effect. The transcript reference I've got noted here is 20120711. Now the next thing I want to take you to professor is there's a reference in your evidence at page 48 and this is again the numbering reference that the – that we've put on these documents, WIT.MANDER.0001.48 and this is under 1.3, if you look there in that first paragraph under 1.3, you’ll see that you say, “This deviance from customary ductile detailing remains a contentious issue in the Hyland Smith report.” Now this is about the confinement on the concrete columns. I just want to ask you about the way you've described that as a deviance from customary ductile detailing, and then I made a note yesterday again that you said in the course of evidence, “Coatsworth may have assumed a well detailed ductile building but looks can be deceiving.” Remember making that comment?

A. Yes I do.

Q. Now what are you – when you say there's a deviance from customary ductile detailing, are you referencing that back to the requirements of the code?

A. Yes I am, like as I've said before, like I think a lot of this hinges down on the trigger. Is it in or is it out, and my view is that it's in, you should use ductile detailing and so it's deviating from that, okay.

Q. Now is it your view that the way in which the columns were detailed did not comply with the requirements of the code?

A. No because you see they could show that it complied and this again comes back to this bogey of compliance versus best practice.

Q. Yes.

A. I would not advise a student to do this even though they could graduate the next day and go out and read the code and say, well we're going to do it, have a shear wall building, we're going to have these gravity load columns, and we're going to do a deflection check and show that the check is less than what it's permitted to be by that code.

Q. Yes.

A. And then if it conforms to that we can avoid having all this steel, let's do it that way. That could be a conscious design objective.

Q. Mhm, and the reason you wouldn't do that?

A. The reason I wouldn't do that is that it's not best practice, like as you get the added insurance for relatively modest cost of putting in the transverse reinforcement that will make the columns very ductile. Now as it turns out in my view, it may have – that even alone may have been insufficient in saving the building because I believe the columns were as I said in my concluding remarks yesterday, my calculations show that the columns would have had a much better chance to be – if they were a little bit larger in diameter to perform better. Now often engineers are loathe to cross their architects at this point but I don't think that argument will hold because the exterior columns have less intense axial loads on them in the interior columns.

Q. Yes.

A. And so at least if the interior columns were bigger diameter, like I said 500 millimetres, in fact 600 millimetres would be better. Like it's not making a whole lot of difference in the grander scheme of things in terms of cost, it'll be a little more expensive but not markedly so and you'll get marvellously better performance out of the structure had that change been made at the time.

JUSTICE COOPER:

Q. Professor Mander, just help me, when you say in this statement, this deviance from customary ductile detailing, do you want us to take it that in your opinion ductile detailing, the kind you're discussing, would have been customary at the time that this building was designed?

A. Okay, so I believe it was customary in the sense that as I mention elsewhere, this was about the time quite a sea change was taking place from the large number of moment frame buildings that were being constructed around the city, and then elsewhere in New Zealand where, and many of those structures back in those days were designed and built and constructed by the Ministry of Works. Those structures generally were very robustly designed, but they were moment frame buildings, they may have had an elevator shaft in them, but they wouldn't, it wouldn't go as far as calling that class of structure a shear wall structure. So if you switch the paradigm and then all of a sudden say ah, this is a wall building, and so the wall is going to be the primary lateral load method of resistance, and we'll just check that the columns can go along for the ride, then that is a major sea change, but what I'm saying is that from the point of time that this happened, this was the change so if you go back in time that was the custom of the moment frame structures. Building contractors and designers would have been familiar with doing that but this was like the brave new world of changing the method of construct – or design and construction to this new generation of pre-cast structure.

COMMISSIONER CARTER:

Q. What you're saying is that it would be customary in the conventional poured cast in situ reinforced concrete buildings, it would be customary?

A. Well most –

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Q. Would you extend that to precast concrete?

A. Most buildings were precast, I mean cast in place prior to this time. The columns of course were cast in place but I think everything was designed as modular components so that it could be all placed together fairly easily to essentially cut down the construction time, and I think a lot of the design, if you remember the relationship here was where the, this was a developer led project where the developer wanted a certain style of structure that was contractor friendly to them for ease of construction, and so how the designers approached this was sort of break from the mould and design things a little differently, and I think this is where a lot of those changes took place, because of that, a change from how things were previously done to how they became to be done back then, and as of now.

CROSS-EXAMINATION CONTINUES: MR MILLS

Q. This issue about whether these columns could be designed solely to carry gravity load was dependent wasn't it on the deflections of the building being limited within a certain range?
A. Yes.
Q. Have you read the evidence of Mr John Henry?
A. Yes.
Q. You're aware then that there's a serious question that will be looked at more closely in, I think in the next round of this hearing, as to whether the deflections were accurately calculated?
A. Yes.
Q. Yes. And if they weren't accurately calculated and the drift was greater than what was calculated then that would flow through in a fundamental way to the subsequent design decisions that were made wouldn't it?
A. Yes that's correct.
Q. Now the next statement in your evidence that I just want to get you to comment on, the same type of question is at page 49 and again it's our numbering system. I'll just get that brought up.

WITNESS REFERRED TO SLIDE

Q. You will see you say there in the second or last full paragraph, "There is an analogous problem to low transverse reinforcing steel in the columns: no transverse reinforcing steel in the beam column joint regions." Now of course you've talked about that quite a lot, but again I just want to ask you whether you are saying there that that is a code compliance issue.?

A. Well again it comes down to this all or nothing trigger and however, having said that, I do believe that even under gravity load design there probably should be some reinforcing bars in the joints. I'm very familiar with this problem because back in the early '90s we did a lot of research on this when I was in the State University of New York at Buffalo and we were investigating common gravity load design and for the interior columns it's very unusual in plain gravity load design to have any beam, any steel in the joints. You do on the exterior joints though, typically just two hoops. That was more or less a standard detail that's considered to be sufficient to conform to the ACI code from which the New Zealand codes were derived. And the point I'm making here is that even under some drift you're going to get a moment in those joints. And so an astute designer would've realised this and then put in some shear steel in the joints just as insurance.

Q. So definitely not best practice?

A. No.

Q. And code compliant or not?

A. Well again if it meets the drift requirement then people feel that it doesn't need anything and like I find that difficult, I really do. Like I don't, I think that's not good practice for sure but I would rather go to a high level and use, and go with a good practice as Professor Priestley espoused, but on the other hand I feel that the designers had the right to do that in that they seemed to have, I don't know, maybe exploited a loophole in the code.

Q. What's the hole that you think that was exploited?

A. Well the fact is the deflection thing, it's really the deflection thing. Like I really believe that that's been tightened up subsequently.

Q. It's possible here isn't it that far from exploiting a hole in the code that the deflections were simply wrongly calculated?

A. That's also possible.

Q. Yes. Next at page 50, you'll see there again in the last paragraph just above 1.4, finally on this point of ductility, "It can be shown that if the NZS 3101 code prescribed amount of transverse reinforcement was provided in the columns, this would not necessarily have prevented the collapse," et cetera, et cetera. Now I read that any rate as expressing an opinion from you that code prescribed amount of transverse reinforcement was not provided in the columns. Is that what you're saying?

A. That just simply means closely spaced reinforcement as given by the relevant clause, the seismic clauses in the code. It doesn't mean the non-seismic clauses.

Q. No.

A. So in other words if the deflection was exceeded then this would be triggered but based on compliance, or based on good practice which I would still advocate, if you put the code amount of confining steel in it's still going to struggle, and it's largely because the column is small in diameter.

Q. Yes just let me bring you back to the specific question, and if this isn't what you intend to say then say so. But it says, "If the NZS 3101 code prescribed amount of transverse reinforcement was provided in the columns," now to me you're saying that it wasn't. It didn't have the code prescribed amount of transverse reinforcement. You say, "If it was," et cetera, is that what you're saying, that it didn't have the code prescribed amount of transverse reinforcement?

A. No it didn't have, it didn't have, clearly it didn't have.

Q. All right thank you, that's all I needed to know thanks. Now finally then if we can just go a bit further down that page, now I'll find this now. I am looking for a reference to inadequate lock in details on the east-west beams which I thought was on the same page, now I don't see it. Well perhaps I don't need it. We'll see if Professor Mander needs to actually find it but…. Well I thought that there was actually a specific reference to inadequate lock in details on the east-west beam, so I'm sorry about this but I will find it over the break if need be, but let me just, because it may not be necessary because Professor Mander will probably remember this reference. You do make a reference to inadequate lock in of the, in the details of the east-west beams into their seats and I made a note yesterday when you were giving evidence that you had added to that by saying that, "The seating sill on that western wall was only about 20 millimetres, it was not well anchored, it was quite poor," remember those comments?

A. Yes, yes.

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Q. Well again, same sort of question, are you saying that the inadequate lock in details of the east-west beams into their seats on that western wall was a code compliance issue?

A. That's a more difficult one because this gets down to the art of detailing and we as engineers, the implementation of how one lays out the reinforcing steel have quite a lot of latitude but specifically what I was meaning is the lock-in detail and you'll recall just a few minutes ago this morning we were talking about Professor Fenwick's interior beam column joint with the green lines okay and it had the hooked up bars on the bottom. Basically you've got to get the hook well past the centre line as much as possible and ideally right to the other side of the joint. Now when you make the column it starts out as a square column, all the other ones are square, they're going to have a modest fighting chance to survive but when you narrow things up by 100mm then you don't have an opportunity to reduce your cover accordingly and so it's the anchorage or the locking in of those bars is unavoidably smaller and so that would be what I would tell my students at least this is not a sufficient detail to ensure that one is going to have good anchorage of that bottom reinforcement or the top reinforcement for that matter and so consequently it's prone to fail earlier.

Q. So you're not expressing a view on whether it's code compliant?

A. No.

Q. But you are saying again this is not best practice?

A. I would say certainly by today's standards but even back then and if I refer to Professor Priestley he invoked Professor Paulay's textbook. One of the things that that book became very famous for was one of the final chapters in it which is called "The Art of Detailing" and that's exactly what I spoke of. It's a fabulous exposé on how one should put detailing together. It's not a science. It's an art and this is the art of the discipline and how you apply it.

Q. I see that the reference I wanted on that inadequate lock-in details. It's at page 81 for the record. Just to pin this down, I think you're agreeing that poor lock-in detail on the western wall was not best practice in your view by the standards of the day?

A. Most probably by the standards of the day and certainly by today's standards.

Q. Now I think where we've arrived at on this as a result of those questions is not code compliant in some respects but certainly I think in all the matters we've looked at not best practice?

A. Well my best practice, no. What others may think could be different to me.

Q. You're only being asked to give your own opinion and you're qualified to give it as an expert and that's where we've gotten to isn't it?

A. Mmm.

Q. Now on the other hand you make a number of statements about the CTV building which, at least on my reading of it, are quite praiseworthy and I just want to take you through these and see if you can just help me to understand how you're reconciling your positions on this and the first of them is at page 48 and you'll see there down in that last full paragraph, "The CTV building was in fact quite revolutionary as the details of the design are clearly contractor friendly." Now are you referring to this building as being revolutionary in a complementary and positive way?

A. Well the construction, yes. How it was constructed would be quite markedly easier to implement than many of the cast-in-place buildings that went before that.

Q. Yes, but the result of it is a building which I think we've just agreed after going through a number of different points is a building that doesn't meet best practice and in some respects at least was not, in your view, code compliant. So does that make for a revolutionary building?

A. Now this is where you asked a similar question of Professor Priestley and he said that clearly it wasn't. He didn't think it was innovative or, but I think if you listened carefully to what he said it was basically the design side of it I think he was referring to and I would agree with that. The design was very orthodox, albeit taking a lot of liberties, but the construction method is very contractor friendly, something that one can build rapidly using tray decks – that was pretty uncommon at the time – and using a lot of pre-cast elements such as they were meant that the erection of the main bulk of the structure could proceed quite rapidly.

Q. So despite the numerous criticisms that you have made in response to the questions I was putting to you, are you giving a tick of approval to this building?

A. No I don't mean it like that. I'm basically saying you need to look at delivery of a structure in a holistic sense. That means that it's a turnkey operation. Basically it's the design and the construction, the fit out and in that sense given the economic conditions of the day which were very high interest rates which means the time cost and money was horrendous by today's standards and, you know, like prior to that time they were pretty horrendous so, given that, I think that the conditions were dictating more or less that things be done in a new more efficient way and the constructors or the contractors had explored ways to do that which were quite new at the time so it was a revolution in thinking that we can get away from all these cast in place structures which take a long time to erect to something that's more modular and more efficient to construct.

Q. But I think you're agreeing that the result of this revolutionary contractor friendly approach has been to produce a building with a significant number of serious problems?

A. That's one of the clear, unintended outcomes.

Q. Yes. Now you've also described the building, and this is at page 89, I won't bother to go to it right at the moment, I don't think I need to, you refer to it being "designed and constructed in an innovative manner". Now I just want to ask you, putting those two passages together, "quite revolutionary", "designed and constructed in an innovative manner", do you agree with me that if somebody is embarking on a structural design that is new and revolutionary it's designed and constructed in an innovative way that it requires an experienced designer to know how far the envelope can be pushed before we get a disaster?

A. Yes I tend to agree and many of the world's breakthroughs have ended in disasters, but they've nevertheless been breakthroughs. Tacoma Narrows Bridge would be a good case in point.

Q. But you wouldn't put into the hands of somebody who had done no multi-level building design at all a revolutionary innovative multi-storey building would you?

A. Well, um, I would, no I can't answer that because this is really a business decision as to how you run a firm and I haven't been in that situation.

Q. Do you not have a personal opinion on this? You're an experienced academic. You've spent a lot of time both in the field and in academia thinking about these issues?

A. Right, well from that point of view then and putting that caveat on it, I agree.

Q. You agree what?

A. It would desirably have more oversight and I think most of the engineers in the room would agree that New Zealand and certainly in other countries such as the United States have moved on a lot since that time and that there is now, back in those days there was a lot of reliance on the checks and balances being vested with the council and now it's more vested with the profession and there's a lot of peer reviews done by other engineers and so a lot of those comments, like others would have the opportunity to comment on the suitability of that type of design and I don't believe that that was the practice in the day.

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Q. Just let me before we take a break just cut through some of that by saying that there will be evidence given in the form of an internal Christchurch City Council memorandum by Mr Brian Bluck who was the head of the building department at the time that will make it clear that the Council regarded the structural designer as carrying the ultimate responsibility for whether or not the building complied not the council. Does that surprise you?

A. No.

Q. So your statements about it being on the council because they permitted it and okay because the council permitted it doesn't sit very comfortably alongside that does it?

A. Well I don't think they want to take the responsibility for everybody else's mistakes so that's understandable.

Q. So ultimately it lies with the designer?

A. It does.

HEARING ADJOURNS: 11.32 AM

HEARING RESUMES: 11.48 AM

CROSS-EXAMINATION CONTINUES: MR MILLS

Q. Professor Mander, I just want to tidy up one or two loose ends or possible loose ends from yesterday and make sure we're on the same page on things and principally the statement that you made in your written evidence about the vertical accelerations in September being exceptionally high, and as you know you make that statement several times in different ways. Now as I understand it, where we got to yesterday is that you've now agreed that you would not say that they were exceptionally high in September. Is that right?

A. Okay –

Q. Now perhaps to forestall what might be some uncertainty that might creep into that, can I – I should break this into two parts. Are you now saying that at least at the one to one and a half second period for the building, that the vertical forces were not exceptionally high in September?

A. I don't believe I ever said that they were like that. I always meant they were for the, yeah, part of the spectrum out to about three hertz.

Q. So whether or not you did and I could if I wanted to I could take you to some passages that do say that, but the important point now is you're saying your evidence is that at that one to one and a half second period, the vertical forces in September were not exceptionally high, agreed?

A. Agreed.

Q. Thank you. I take it that what you are saying though just again to get some clarity around this, is that at less than about 0.5, sorry 0.4 seconds, then the vertical forces are a bit more than the one-third of gravity. Is that the other half of your evidence on this?

A. Two-thirds I think.

Q. Sorry two-thirds.

A. Two – well there's two parts to this answer and because you're – the way the question is posed is ambiguous in that it's not clear to me if you mean in absolute terms or in relative terms, so let me answer it both ways.

Q. Okay.

A. In absolute terms the – in hindsight now that we know that we've had the Christchurch earthquake, those are not exceptional but in relative terms they at the time, they were exceptional given that they were greater than the peak ground acceleration, they were greater than the horizontals and that wasn't expected by design at the time, or these were designed for, and –

Q. But that's in this early period range of about .4 of a second.

A. Yes and the reason is that that's the frequencies that excite the vertical modes, in particular the floor slabs and overload the columns.

Q. Yes. Now do you agree with me that no one has established exactly what the effect of the vertical accelerations were on the CTV building in either September or February?
A. Well not exactly. I don't know what exact means actually in this context of earthquake engineering.
Q. Okay, so we're in agreement that no one has established it exactly.
A. Right.
Q. Would you agree with me also that whatever those vertical acceleration forces were on the building, that they're likely to have exacerbated any existing structural weaknesses in that building?
A. Absolutely.
Q. And I think we're also agreed that there were a significant number of structural weaknesses in that building?
A. Yes.
Q. Now I'm just going to put to you some points that, as you'll quickly appreciate it, don't originate with me and this is about the February vertical forces and these are points that Professor Priestley made in giving evidence and I just want to cross-check with you to see if you and Professor Priestley are in agreement on this. Do you agree that how a building reacts to vertical forces is affected by each of the following points and I'm going to run by you, maybe others as well but you'll want to add. The first one is any previous cracking in the floor slabs?
A. Yeah, that will affect it.
Q. Secondly how different floors in the building might respond?
A. Yes.
Q. Third, by the live loads in the building?
A. Yes.
Q. And finally by the axial flexibility of the columns?
A. Yes.
Q. Thank you, so agreement between you and Professor Priestley on that. Now just again a fairly small point just to try and sweep this away. At page 68 of your evidence, you make the statement, I'll just let you find it and I'll find it myself.

A. Yeah, I've got it.

Q. So it's the second to last paragraph on page 68, I don't know whether we'll bring it up or not but in any event, you say there, "It is inevitable that the two displacement and force maxima will coincide momentarily producing extremely high loading and stress demands on the materials."

A. Yes.

Q. Now as I understand what you're referring to here is that at some point in the spectrum we'll have the vertical forces and the horizontal forces coming together. Is that the point?

A. Yes.

Q. Now Professor Priestley and this is – can we bring up the transcript, is that possible. All right, then it's day 58 at page 54, and it's at line 18 and following. What have I got wrong here. Let me try an alternative from the Compusoft report, which is ENG.COM.0001.78. Now you'll probably understand this better than I do I suspect, but my understanding is that the Compusoft report and it may be on this and the next page, but I think is the principal page, shows that, and it's in figure 56, shows that the peak vertical acceleration is at 3.7 seconds and the peak east/west acceleration is at 6.5 seconds and I see that I've just been helpfully handed the page from the transcript which I was looking for from Professor Priestley, it's at page 55 just for the reference, I may come back to it if I need to. Now Professor Priestley makes – what Professor Priestley said about that statement of yours, Professor Mander, is this, "Relating to vertical accelerations, Dr Mander on page 27 in the second to last paragraph claims that it's inevitable that maximum vertical load and maximum drift will occur simultaneously. My reading of the results from the time history analysis is that this is not the case as it assumes a steady state response. In fact examination of the Compusoft results on figure 56 shows that the peak vertical load occurred at about 3.7 seconds but the peak east/west response occurred at about 6.5 seconds. These are not simultaneous." Now I'm just inviting you to comment on that.

A. Okay, so the idea that was behind that comment was that you do get large drifts and a peak will coincide with that. Now if you look at about 6.2 seconds you have a high peak of both the red and purple line coming together, so that's not quite at the peak.

1158

Q. Yes.

A. On the opposite side you actually get a trough and that's when the axial load is a minima and I haven’t mentioned much about that but minima axial loads are and as part of the oscillation can be quite problematic as well in terms of the performance, when it's jumping around and then now, and then it goes on, and then further out, so I wasn't, I wasn't necessarily meaning the peak peaks, but as the peaks go they can come together like this – the arithmetic behind this is relatively simple. The periods of motion for the east/west direction, well north/south or east/west direction are going to be somewhere between one and a half and two seconds, and then if you have 5 Hertz so that's like 10 peaks that you can get in that time, the chances are that they won't absolutely coincide but they’ll be within a few percent of being close enough.

Q. All right, well I think that's helpful, that just clarifies your evidence on that. I want to ask you now some issues or some questions about the evidence you've given that's in several places and we've touched on it already actually, some of it, about the development market in the 1980s if I can just describe it generically in that way for the moment. You've given a number of opinions about the nature of the development environment in the 1980s and the first thing I just want to check with you bearing in mind, as I reminded you earlier and as you agreed that any opinions have to be within your area of expertise. I’ve looked at your CV and on the face of it any rate I can’t see anything in there that would give you expertise to express an opinion on the development market in Christchurch in the 1980s so I just invite you to explain what your source of factual information is that enables you to give the opinions you’ve given about the 1980s and whether you are qualified within your expertise to give those opinions. And I’ll take you to them so you know what I'm talking about. The first of them is at page 48. What I'll do I think might be the easiest thing is just take you to different passages and then let you comment on them generally. So at page 48 we've got under 1.3, "A liberal interpretation of the 1980s building design code allowed the designers to choose other strategies to provide earthquake resistance." And so that's the first of them and then a little further down if we could go back to the main page, then a little further down you say, "However during the 1980 era of building construction there began a time when developers and contractors put immense pressure on structural designers to look at buildings at low cost coupled with rapid construction details. The former mould was broken, moving from cast-in-place moment frame systems that were the hallmark of a mini building boom in the 1970s..." and so on. That led into the point I asked you previously about quite revolutionary and then further down on that same page we've got in front of us you express the view that it appears to be for these reasons that the structural designer evidently sought a simpler form of construction that avoided the use of copious quantities of transverse reinforcing steel to provide a ductility capability" and then on page 50 again dealing with the 1980s you say, "But in the 1980s at the time of design, such columns and joints would have been considered an expensive and unnecessary luxury that would minimize the developer's profit margin." Now as I said those are all opinions being expressed about the development market in the 1980s when the CTV building was designed and built. Where does your knowledge of that come from?

A. If you read page two of my resume.

Q. Yes I have.

A. Top line. I was the deputy group manager for strategic planning property business for the New Zealand Railways and part of my business was to be au fait with those conditions.

Q. That was in 1987 wasn't it?

A. Yes.

Q. And so did you delve back throughout the 1980s when you took up that position in 1987?

A. I was familiar with what was going on at the time and you know from having lived through the era.

Q. Well just looking –

A. It is a – I admit it's an opinion and it was an opinion not as an academic clearly but as a, by virtue of the position I held at the time.

Q. Well let's look at what you were doing during the 1980s and this is page 7 of your CV on our numbering. So from 1979 to 1983 you were a PhD candidate as a New Zealand Railways Research Fellow at that time you were in Christchurch. Then from 1983 as I understand it through to 1986 you were principally based in Wanganui. You say 1983 to 1986 systems engineer, railway electrification project office Wanganui?

A. That's right.

Q. And then in 1987 you're in Wellington as the deputy group manager and strategic planning manager for the Property Business Group of New Zealand Railway Corporation?

A. Yes.

Q. And then later in 1987 you go to United States, correct?

A. That's at the end of the year.

Q. So you have do you say firsthand knowledge of what was going on in Christchurch in the 1980s to be able to give the relevant, to give the evidence that I've just taken you to?

A. Well to be fair it's probably not so specific to Christchurch. I was more familiar with Wellington at the time and that was booming a little sooner than Christchurch but basically what was happening in Christchurch had been echoed by the earlier few years in Wellington so that's by inference I guess.

Q. Have you talked to anyone in Christchurch about the environment down here that you're describing?

A. No.

Q. And are you saying that the various code and best practice problems that I took you through and that you gave evidence on earlier this morning, are you saying that those issues, those negative issues that we breed on are justified somehow by this climate that you're describing?

A. No I'm not. Not justifying. I'm just calling it as I saw it.

Q. All right. I just want to ask you a few questions then about your evidence which relates to one of your collapse hypotheses in relation to the east-west movement and the western wall. Now I think where we got to yesterday was, you probably beat me to it by accepting that your description of the western wall as having the purpose of providing a substantive degree of seismic resistance for the building was actually not correct. I think that's where we got to isn't it?

A. Well it's not entirely no resistance because there were vertical rebars going through the cores that were grouted in and then there was a gap onto the beam so you can't exactly say that zero resistance. There is some resistance.

Q. No that wasn't what I put to you. Just let me put it again. I think you have now accepted that it was not a purpose of the western wall to provide a substantial degree of seismic resistance. You agree with that don't you?

A. That's true. Any infill frame would normally fall into that category.

Q. All right. I think you then went on yesterday at the same time as you were, let me put it. carefully adjusting your written evidence that nonetheless the nature of the western wall, the note I made would just upset the whole wall what had happened to it, is that?

A. I believe any significant shaking and particularly even if one can see light through there it's not entirely clear where that was but it most probably was at the movement gaps but it basically shows that it wasn't as it was and the fact that there was a wall next door to it that there seems to be some anecdotal evidence that the equilibrium of that may have been upset a little bit by the fact of the demolition next door. Now it's not entirely clear but let me put round the other way. Whatever was done didn't make it healthier.

1210

Q. Now in terms of your various hypotheses for the gravity collapses, this issue about the east-west dislodgement of the beams and the significant issues you identify around the western wall that, as I understand your evidence, is your preferred hypothesis, your most favoured hypothesis?

A. No. There are basically six possibilities in there actually.

Q. Yes.

A. There are essentially two in the east-west direction. Don't forget there's the four level rendition which is perhaps the one that stands out most glaringly in the diagrams but not to say that the single storey collapse is a poorer cousin of that. It could have also happened, and then also in the north-south direction you have the north shunt and the south shunt, each having two possibilities. I would like to say that in many respects they are first among equals and it really depends on the nature of the loading that would have come in at the time as to what really happened.

Q. All right so when I read at page 55 of your evidence that in relation to the lateral displacements due to the influence of masonry walls on the west face that (and I'm quoting) "This may well have been the principal cause of damage of the CTV building in the Christchurch earthquake that I should emphasise the word *may*."?

A. Yes, absolutely. That's one of the several possibilities.

Q. Now that hypothesis that we're talking about or just been looking at, am I right that it does rely on a number of facts about the western wall that you've listed in that aspect of your evidence, the condition of the western wall?

A. No not really, not really.

Q. Not affected by that at all?

A. I think even if the wall was in pristine condition what I proposed later on is it's still a possibility. The point that I just made was that having damage or pre-damage to the wall wasn't going to help the situation. It was going to make it worse.

Q. Yes, so can I take it then that the factual errors which I could take you to about the condition of the wall we don't need to worry about those because they really don't affect the hypothesis you've put forward?
A. I don't believe so.
Q. All right and I won't trouble you by taking you through the factual errors. All right, I want to ask you now about this issue of cumulative damage and low-cycle fatigue. Just again see if I correctly understood your evidence on this and the extent to which there might be areas of disagreement with other experts. Now first of all I notice you mentioned this morning that you described yourself as a co-author of FEMA 306. Is that an accurate description?
A. Yes.
Q. And, as I understand it from your CV, you were a major contributor to FEMA 307 and 308?
A. Well they're all, as I recall, they were all together.
Q. Yes, yes, my understanding of 307 and 308 was that they were working papers feeding into 306?
A. Yes.
Q. Would that be an accurate description?
A. Yes, yes.
Q. But ultimately you describe yourself as co-author of FEMA 306?
A. Yes.
Q. Now again I probably don't need to take you to this although I will if you want me to because it's come up before in the hearing. I think my friend Mr Zarifeh brought this up. The summary of the FEMA conclusions, and just for reference this is ENG.FEMA.0003.18, but I don't think we need to bring it up.
A. May I ask that you do bring it up just for my reference.
Q. ENG.FEMA.0003.18. Now you'll know this inside out I expect. Now the passage that attention was drawn to previously is the one under that second bullet point down there on the left column – "Damage may not significantly affect displacement demand in future larger earthquakes."

So you remember this came up and I think Mr Kehoe was asked about this when he gave evidence. You remember that being done?

A. Yeah.

Q. Now at the time at any rate that FEMA 306 was finalised and published that, as I understand it, is a fair summary of the conclusions that had reached on that issue of cumulative damage. Is that right?

A. Possibly at that time, yes.

Q. And do I take it that at that time at least you weren't dissenting from that view?

A. Ah, no I didn't hold a strong view either way.

Q. Did any of the work that you were doing on FEMA 307 and 308 have any direct bearing on this?

A. Not really, no. I think somebody else actually wrote that. I didn't.

Q. But it's a FEMA report of which you described yourself as a co-author?

A. Yes.

Q. But you're not suggesting that you disassociated yourself?

Q. No, no, no, of course not.

JUSTICE COOPER:

Q. When was that written Professor Mander?

A. I don't recall the exact date. It was the late '90s I do remember that.

CROSS-EXAMINATION CONTINUES: MR MILLS

Q. 1998?

A. Yes. We did the work probably a couple of years before that and the whole publication process takes a long time with FEMA so...

Q. Yes, yes. Now again I made a note yesterday when you were giving evidence that you acknowledged there's quite a bit of uncertainty with determining issues around fatigue line. Is that ...?

A. Yes that's true.

Q. Now do I take it that your position on this is that in order for the CTV building to have undergone relevant fatigue that it must have sustained physical damage. Am I with you thus far?

A. Yes.

Q. But do I take it that what you then say is that however that damage might be hidden?

A. Yes.

Q. And I take it you're not saying it's hidden because it couldn't be seen. It's just in places that weren't able to be readily examined?

A. No, that's not even true. It can be, if we're talking about reinforcing steel, reinforcing steel can experience significant fatigue life consumption and it won't seem anything wrong and you can do measurements on it and you won't know. The experimental results that I showed yesterday for example one of them may have had, say, 10 cycles of loading. If you looked at the ninth cycle, the bar, by visual inspection it would have looked perfectly okay and then the tenth cycle there will be a fatigue crack appear and the performance thereafter will be quite different.

Q. Yes all right well I'll come in with some other issues about that in a moment but do you now hold a different view to the one that is expressed in that FEMA summary that I put up?

A. Well I need, what you have not been told I believe is the context of why that FEMA document was done. That FEMA document was done as a response to some residual issues that FEMA had with claims dating back nearly a decade to the Loma Prieta earthquake in the late '80s and back in those days there was another document that FEMA used, I don't recall which one now, it may have been an 80s C document that were used to judge the degree of damage on structural elements and it was largely done by measuring crack widths and going everywhere in a building and mapping cracks and then based on that there would be some determination that would be made as to, well, if you see the building is 60% cracked then you'll get a 60% payout and it was done simply as that and it was done, this whole study was because this was in dispute and so FEMA realised that they needed a more rigorous approach that was more quantitative and not just looking only for visual inspection and cracks and so forth and so that's why this document was put together and so you look at the cracks. Naturally you look at the physical evidence as you can but in companion with that analysis is also done to try and infer from that what's also going on and to the extent that the structure may have been taken through in an earthquake and based on that then it would be deemed as to the extent or inferences would be made from that and then they would do their work beyond that so our job was really to write the document to put in place those sort of procedures for assessing post earthquake damage both observed and unobserved.

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Q. But you're not suggesting are you that that summary of the conclusions that were reached about whether damage might significantly affect displacement demanded future larger earthquakes is not an accurate summary of where FEMA 306 came to?

A. Well, and again I think that's reasonably fair although typically what happens under earthquake excitation is over time a structure gets softened up and it is possible that certain designs and how they operate, they, their displacements will increase modestly.

Q. Yes.

A. Now as it comes back after an earthquake it may come back to show either more cracking or the same cracking.

Q. Do you now yourself hold a different view to the one that's expressed there?

A. I don't think it's different in the sense that the purpose as I mentioned it was to deal with both seen and unseen damage.

Q. Yes.

A. As sort of teased out by calculation and so I think this document was getting onto that. I don't believe that this was the last word then or now and that this whole field is fairly fluid actually in terms of how we look at damage after earthquakes.

Q. Yes. Now you're aware aren't you that Mr Coatsworth said in his evidence that he'd looked at all of the exterior beam column joints?

A. Exterior ones, yes.

Q. Yes exterior.
A. Yes.
Q. And he'd looked at all of them on levels 1 and 2 where there's no ceilings and where it was open?
A. Mhm.
Q. The latest time history analysis is indicating I take it that the key area of vulnerability may well have been at column D2 on level 1 or 2. Is that –
A. It may well have been but it's more likely to have been C2.
Q. Right. So the columns that Mr Coatsworth looked at on levels 1 and 2, that would include potentially the implicated column?
A. Yes, possibly I'm not sure which one he looked at, I don't recall. There's a photograph isn't there that's kind of all black.
Q. Yes and I'm going to take you to that in a moment. Now I understand your point that we might have at least particularly as far as the steel reinforcing bars are concerned, we might have some straining of that steel, if I've got the right terminology.
A. Mmm.
Q. That you say might not be visible.
A. Mhm.
Q. But it would – would it be surprising if the columns were looked at, let's assume all of them to be looked at, would it be surprising to find no visible signs of distress at all but nonetheless for there to be the kind of significant weakening that you're describing?
A. Well again I go back to how this was constructed. I believe that the concrete inside the joint zone would have shrunken somewhat and that may have masked any internal cracking that would have taken place, and the other type of damage that would take place within the beam column joints would be the fact that again I was alluding to this before on the discussion with the beam column joint with the green arrows with Commissioner Fenwick.
Q. Yes.
A. In there you rely a lot on the longitudinal reinforcing steel having to switch from tension to compression, and in order to do that the steel has

to be bonded to the concrete and repeated cyclic loading is going to deteriorate that concrete bond to the point that it doesn't function in its classical sense which is it's kind of acting like a glue round the longitudinal bars so you have to rely on other methods of providing resistance such as friction and once that happens then the joint in my view decidedly becomes weaker than the surrounding elements.

Q. All right, look I'm just going to take you to some photographs, which may or may not assist this. They are ones that were taken on about the 11th of February by Cunningham Lindsay who were the Loss Adjustors for Madras Equities. I just want you to have a look at these and in – just consider whether they affect your views at all. The first of them is BUI.MAD249.0476.23. Now as I said these were taken very shortly before the 22nd of February earthquake. There's the first of them and you'll see that at least on the exterior wall it's looking right up underneath the column isn't it, right underneath the beam where the column intersects with it.

JUSTICE COOPER:

Is this the east wall?

MR MILLS:

Yes it will be the east wall, yes.

CROSS-EXAMINATION CONTINUES: MR MILLS

Q. Now I'll give you a moment to look at that, but you've got a more trained eye than I do, but I see no signs of any distress at all in that jointing area. Do you agree with that?

A. Well I can't tell, you can't tell from photographs. These are micro cracks and the skilled eye needs to look up very close, like literally kind of this far away, you know, six inches.

Q. Well Cunningham Lindsay were going over this in order to make decisions as Loss Adjustors and nothing in what they concluded identified any significant damage of any kind. Any rate, let's just look at

some more, you may say the same thing, that these don't help you, but the next one is page 24, the same reference, next page and again you'll see it's looking right up underneath the beam column joint.
A. But looking underneath is not the place to look.
Q. All right, where should you look?
A. In the side.
Q. In the side.
A. Mhm, and those spandrels are kind of disguising what's going on. It's very hard to see you know what's there actually.
Q. Well if we're not looking at the spandrel are we?
A. Well those are the spandrels on the outside, those big chunky concrete pieces.

JUSTICE COOPER:
Q. I don't follow, in the side of what?
A. Well the side of the joint, like there's going to be – the side part of the beam I should say as it frames into the joint. So everything is a reasonably uniform thickness, this is like kind of a big outrigger there, the joint itself as built like that, well as shown like that of course looks immensely strong, so one would not ever think that there's going to be a problem with the joints because it gives the sense that these are very large strong chunky joints.

CROSS-EXAMINATION CONTINUES: MR MILLS
Q. Yes.
A. And the photograph that you've given there is deceptive in that the columns look strangely short so one would look at – take a cursory glance at that and say, oh, short squat columns, and if you did have – had you seen the plans you would have realised there's not a lot of transverse reinforcement so you'd then jump to the conclusion, ah this is a candidate for a shear critical failure and you would be looking for big diagonal cracks in the column as probably more likely to happen than something in the beams, in the joints, in the beam column joints.

COMMISSIONER FENWICK:

Q. Can I just seek little bit of clarification in this case, you're putting this degradation down I take it to that the bond slip of the vertical column bars–
A. Well that can –
Q. – (overtalking 12:29:51) contention to compression, and you talk about this being a process. Is that correct?
A. Well that can be of them.
Q. If that occurred where do the bars slip to?
A. Within the joint, like no, they're not slipping it's the bond is being destroyed.

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Q. The bond is being destroyed, but if the bond is destroyed it implies the bar is getting longer if it's in tension or shorter if it's under pressure doesn't it?
A. Yes.
Q. If it's getting longer, if you got the bars to be true, how can you have that without a crack?
A. Well as I –
Q. Can you explain it, I don't see how this is possible. The bar going forward, slips through here, anchors up here and now you've got the extension of the bar, the point where it's anchored down through here, it has to doesn't it? Surely? I mean how can you get it without a crack in it?
A. There will be, naturally there would be a crack that would be trying to form and there could be a very small growth in the length of the inside the joint simply because of the bars above are going to go into compression. So the overall displacement compatibility is going to require that the bars are the same length basically as they, just roughly inside the end of the column on above to the end of the column below. So there's a lot of latitude in there for having high compression strains,

and then also high tension strains. The net effect is still could be almost zero.

Q. Yes but earlier on today you said that when we had the bars slipping true you'd have high tension at the middle of the joint zone –

A. Yes. Yes.

Q. – on both sides, and you said this increase would be axial force on the member. If you had tension through there, there must be some crack to accommodate that tension extension?

A. Yes, and that will be on the edge of the, on the outer corners and then you get a diagonal crack going through the joint and my position is that this is somewhat concealed by the fact that you've got the precast elements which possibly have stronger concrete, even though the specified strength is the same, these are manufactured in factory-like conditions and there would be a tendency to safely assume that they would be a little bit stronger.

Q. So the crack was there but you couldn't see it?

A. No, not inside the, it's, it shrunk and these could be quite small cracks. I believe the presence of the pre-cast elements are, mask the possibility of seeing it easily, and the only option that, or the only possibility that you have of seeing anything is about a one inch wide slit down the middle, but there's a clearance gap between the two precast elements. There's a little bit of cast in place concrete there. You may be able to if you were looking see a little bit of an X right in the middle but it doesn't look like there was much. Now, having said that, I think in the lower columns that again the axial loads are so intense that if there are cracks that generate from one lurch to the left or the right, then when it wants to tend to straighten up as it would've been pulled back by the shear core, then the heavy axial loads are going to somewhat close down those cracks again. So I don't think they necessarily have to be highly visible.

Q. So we're now cyclic yielding in tension and compression in the interior beam column joints, that's your thesis?

A. Yeah, I think that's a distinct possibility. Now whether that happened to a great extent in the initial September earthquake I'm not entirely sure,

JUSTICE COOPER:

Q. Can I just clarify, if you'd been inspecting the building after the September earthquake, how would you have found that, or found evidence of that phenomenon? What could you do to discover it?

A. Well so this goes back to perhaps elaborating on it, or making a better answer to my question earlier today which I realised was probably too terse. And it really relates to understanding from simple calculations what level of excitation the building would've gone to, and if it gets out into the yield zone, because of the magnitude of the excitation, then one can easily tell by just doing a back of the envelope subassemblage, much as what I did in my own drawing, that the joint shear is going to be very, very high and knowing that there's no steel in there, that's the place to first look, that will be the clue to look, now then you look at it and then you say, "Mmm, can't see anything, is there something amiss here?" Well I think if you realise then that the concrete has shrunk and it may be difficult to see anyway. So my next, my view then is that well you would really need to confirm that there was no damage, and there should be some sort of non-destructive testing technique that one can implement to check for cracking in the joints.

Q. Well is there one?

A. Yes, ultrasonic tomography will do that.

Q. That's a commonly used technique is it?

A. It's becoming used quite a bit now in the United States.

Q. And you repeat that process with the class of at risk building that you referred to?

A. Desirably, yes, but then on the other hand –

Q. And after each earthquake event of significance?

A. No because I, this is where I do believe the FEMA 306 or 307 is more or less correct. Like if, unless the next earthquake is demonstrably stronger then I think it's reasonably, it's reasonably safe, it's not ultra safe but it's reasonably safe to assume that the stresses have not increased, or the deformations have not increased, rather the cyclic nature may have continued on which is going to continue to wear things down and continue to do damage which one can account for by calculation as I've tried to show, but the sev – the, so it's always this duality between the peak that it has experienced which is really related to the largest peak excitation or response from previous, all previous earthquakes, that's one aspect, and then the other aspect is the summation of all the cumulative cyclic effects coupled with that. And so once one makes that determination it is fairly easy to see that if the shaking is not getting, or is not increasing subsequent to the initial shock, then for the most part things are going to be reasonably okay.

Q. Well wasn't that the case here?

A. No, because there was a noticeable difference by people in the building that raised alarm bells after Boxing Day, and again that was because of the high vertical shaking almost directly beneath the building, and may not have swayed much laterally but –

Q. I thought your thesis was based on the level of shaking but you're now talking about the experience of people in the building?

A. Yes but that, okay, the level of shaking but that would also show scrutiny of those results, that the vertical components actually on the Boxing Day event as I recall are the same as if not maybe even slightly greater than the September event. And so because of their intensity, one should say, "Mmm, this is interesting, maybe we should take a second look."

Q. So after Boxing Day was the time for a second look. Is that your evidence?

A. Yes.

CROSS-EXAMINATION CONTINUES: MR MILLS

Q. Let's look at a few more photographs and see if they add anything?

JUSTICE COOPER:

Q. Well one more thing sorry, the technique that you say is coming into use in the United States –
A. Mhm.
Q. – and I've forgotten the terminology already?
A. Yeah, ultrasonic tomography.
Q. Is that dependant on specialist equipment?
A. It is.
Q. Is that equipment available in Christchurch?
A. I don't know.

CROSS-EXAMINATION CONTINUES: MR MILLS

Q. Let's look next then at page 25. You may well give me the same responses but I want you to be aware of what's in these photographs. Now again you'll see it's looking pretty close up underneath the corner connection between the beam and the column?

WITNESS REFERRED TO PHOTOGRAPH

Q. Any comment on that?
A. Well my comment is that it looks to me in that mid height there in the right-hand corner there's some sort of cover or placard and I would say rip that off and let's look behind that.
Q. Mhm.
A. I don't think you can do one of these street side things with a, no matter how powerful the camera is it's not sufficient.
Q. Next one that I've got tagged here is page 27 which may well evoke a similar response does it?

WITNESS REFERRED TO PHOTOGRAPH

A. It does. Too far away, fuzzy photograph, indecisive. Go there, get a ladder out and take a proper look.

1240

Q. So in brief not a lot to be said for it?
A. No.

Q. Okay. Let's see if there's any others here. Perhaps have a look at this one because this does show some spalling I think you'd call it. This is 0478.8. Any thoughts on that?

A. Well I can't tell what it is from this angle. The photograph quality is not very good and it looks to me as though that weathering damage has been patched up and so one would have to ask oneself why is this weathering damage even there. I suspect it may have be because, this is not an uncommon occurrence like manufacturers are not perfect. They sometimes, the reinforcing cage shifts in the mould and the cover concrete ends up being insufficient and once the concrete's cast some 20 or more years later it comes back to haunt you you get some rust behind there and it pops the cover off and you have to patch it accordingly.

Q. All right. Now just to confirm the areas that were open to inspection in the building I just want to go to some other documents, and the first of them is BUI.MAD249.0486 so that's a drawing that we've looked at several times and it shows the column placement. Now I think I'm right that the column marked as C7 which is on line 2D that that's directly above what's also been referred to as column D2. Now column D2 I think isn't it which you've, it's now been identified in the latest time history analysis is potentially –

A. C2 but I do believe D2 will be a close cousin.

Q. All right. So I want to show you that one first and then I'm going to show you one which shows the studio plan where the ceiling was open so you can see where that sits in relation to these various columns that seem to be now being identified so if we could go to WIT.SPENCER.0001.5 now you might have to mentally re-jig this because the orientation I think is opposite to the one we just looked at but you'll see in the middle there the area marked studio?

A. Yes.

Q. Now if we turn that sideways then it would be I think the same compass angle as the previous one.

A. Yes.

Q. And that I invite you to accept would encompass the area that had column C7 in it that we just looked at and the columns underneath that and it's a matter of fact that the ceiling in the studio on level one or the ground floor depending on how we're describing it was open.

A. Yes.

Q. And so when Mr Coatsworth looked at this he had full access to that beam column joint area above the studio. So I just bring that to your attention really.

A. Yes.

Q. And then finally and this is a photograph I think you've seen before. Would you just have a look at WIT.COATSWORTH.0001B.1 and there is a column beam connection in that basement area. And then finally if we could just look at 0001D.10 which I think you've seen before. I think you might have mentioned it a moment ago.

A. This is the one I meant.

Q. So there's the open ceiling in the studio area and it's readily visible. Now these things were all looked at by Mr Coatsworth. Quite a substantial amount of column area in this open space. You know what his evidence was. Do you say that looking at that connection and others similar to it that there still could have been hidden damage that wasn't manifested in any way?

A. It would be very hard to see. First of all for example in the lab when we do experiments we typically try to highlight any cracks by painting the concrete either white with a very flat latex paint or even whitewash so when the concrete does crack you see a little grey line. The difficulty with this painted a dark colour is that you're not sure what you're looking for because the cracks are almost the same colour as the paint so that's the first issue I would say that makes things difficult to view. Secondly knowing that this is in the ground storey this is where the axial load would be pretty intense and you showed me the figure before with all the oscillating axial loads. If you had any cracks in there those oscillating axial loads are going to shut those down in the ground storey so again there's a potential for a false sense of security that we can't see anything so therefore it ain't there but on the, by contrast there were upper storeys where the cracks did show up and that's because I believe there was insufficient axial load above those columns to shut down those cracks and also with the high level of axial loads and the relatively, the neutral axis within the section is going to be quite deep for this level of axial load and so the cracks are going to be even finer than they would be in the upper column if that actually happened.

Q. All right. I am just going to ask you to identify some points that will assist the Commission I think in understanding your evidence on this, and I just want to ask you what elements you say are involved in each of the following and I will give them to you one at a time. So which elements do you say suffered low cycle fatigue in the CTV building?

A. Well I pointed out in the photographs yesterday in the shear wall that's the obvious place to go looking at first or at least scrutinise that by calculation because that is exactly what the wall is designed to do.

Q. Yes.

A. And at the time and even know the whole phenomena is not well understood by most designers. I doubt that it's taught at the undergraduate level. I know I believe I did when I was here.

JUSTICE COOPER:

I think the question which parts of the building exhibited low cycle fatigue?

CROSS-EXAMINATION CONTINUES: MR MILLS

Q. We've got the shear wall are we talking both shear walls?

A. Both shear walls would be the obvious places.

JUSTICE COOPER:

Q. What you are saying it's the obvious place to look?

A. It's the obvious place to go to first because the designers chose those places as the primary seismic resistance component so obviously you would go there first.

CROSS-EXAMINATION CONTINUES: MR MILLS

Q. It's a slightly different answer than the question and you may not be able to answer the question but I was asking you to identify which elements in the building you believed suffered low cycle fatigue?
A. Those definitely, potentially would have.

JUSTICE COOPER:

Q. Well did they though? Is it your opinion that they did? That's what you are being asked.
A. Okay yes.

CROSS-EXAMINATION CONTINUES: MR MILLS

Q. All right. Any others, any other elements?
A. No any steel that is strained under reverse cyclic loading is suffering low cycle fatigue period by definition.
Q. So wherever there's steel in the building that's an element that in your view had suffered low cycle fatigue?
A. Absolutely.
Q. Right. My second question is what do you say are the main periods at which each of these elements are undergoing low cycle fatigue?
A. Well there's essentially two. There's the sway period which is either somewhere between 1.5 and two seconds effectively and any other longitudinal reinforcement that is subject to sway in the elements those elements on the reinforcing steel are going to be strained in both tension and compression and that is low cycle fatigue. Now it doesn't, don't misuse the terms low cycle fatigue and construe that that means failure. Like low cycle fatigue is a damage, is a slow damage that takes place but it accumulates to a point that eventually leads to fracture so there's two parts to it– there's the low-cycle fatigue cause and then there's the effect which is eventually a fracture and the loss of capacity.

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Q. Well that leads very comfortably to my final question. How many cycles do you say are actually required to cause fatigue failure in each element, each of the elements you've identified, how many cycles do you say are actually required to cause fatigue failure?

A. I haven't done the calculations. I could easily do those if you would like them to be done but basically it's a function of the plastic hinge length and the member dimensions but as a rough rule of thumb it so happens that the plastic strain amplitude, you can show this by calculation, that this plastic strain amplitude would be roughly similar to the plastic hinge rotation. These are both numbers in radians. One's in radians and one's a strain but the numbers are similar so, for example, if you have an inter-storey drift of 3% and the yield drift is 1% then we have a plastic drift of 2% and then for 2% you would just pop that number into the formula and it will tell you how many cycles there are so I don't know. I guess it's going to be four or five. It's not going to be very many potentially. Now there's a lot of scatter as you pointed out. It's not precise. There's a lot of scatter around those results so it can vary over quite a wide range.

Q. All right I've just got one further thing I think I can get through before lunch, then very little after lunch. I just want to take you to an article that you're referred to in your CV. It's Mander – Low-cycle Fatigue Behaviour of Reinforcing Steel. You're familiar with your own article presumably?

A. I hope so.

Q. I see it was published in 1994.

A. That's a while ago.

Q. I'll just give the reference to it. You may want to bring it up. It's WIT.MANDER.0001.17. I think that's the reference in the CV actually. Yes, you'll see it's referred to there. Now we've taken a look at that and I just want to run by you some conclusions, and tell me if you agree that they are proper conclusions to draw from that paper. I'm not trying to trick you. I'm just trying to see whether we've got this right. Now the first point I think you're making is that the number of cycles that the reinforcing steel will go through before it starts to suffer strain damage depends on the amplitude of the strain that it's exposed to. Is that accurate?

A. Yes.

Q. And I think you concluded that if the steel is loaded to a strain of 3% then it can undergo four full tension compression cycles before suffering any real damage. Is that right?

A. Fracture.

Q. Fracture?

A. Fracture, yes.

Q. Subject to that adjustment, that's a correct conclusion from what you say?

A. Yes.

Q. Did you also conclude that yielding occurs at a strain of about .8-1%?

A. Ah, not for the steel. Steel yields at .002 thereabouts.

Q. All right then somehow or another I've managed to get that wrong. Now at that level at which yielding first occurs, did you also conclude that at that level of strain the bar can undergo about 50 to 150 cycles before fatigue cracks initiate?

A. No, the one that you're citing was pre-strain hardening, so not yield but pre-strain hardening. Strain amplitude of .008, some 150 cycles.

Q. And is it also a correct conclusion from what you've done in that paper that the level of strain is proportionate to the level of damage that would be observed?

A. No.

Q. Not correct?

A. No.

Q. That's because –

A. – that's the square off basically. So if you double the amplitude you quarter the fatigue life.

Q. All right so...

A. That's on the plastic strain but if you look at it there's another equation in there that would basically say if you're just looking at the total amplitude, total strain amplitude, you double the amplitude you reduce the fatigue life down to one-eighth or 12.5%.

Q. All right. I imagine others than I might understand that. I'm simply a lawyer so I'll take that as a given and see if others make anything more of that. Now does it also follow from that article that the September shaking was large enough to cause yielding but not strain hardening that you would expect it to be able to go through about 50 to 150 cycles before fracturing?

A. Ah, possibly.

HEARING ADJOURNS: 12.56 PM

HEARING RESUMES: 2.15 PM

CROSS-EXAMINATION CONTINUES: MR MILLS

Q. Just one or two further questions Professor Mander just to wrap this up, first of all at page 43 of your evidence and again I'm referring to our page number. I'll just let you turn to that. You'll see that you say there the third and final purpose of the submission is to provide an alternative hypothesis, read from my own, through advanced computational analysis, sorry to the original collapse hypothesis proposed in the Hyland Smith report and this is the part I want to ask you about, "…it is shown that the columns independent of their degree of ductility capability, could have collapsed over the lower four storeys from a classic type of buckling known as", and I'm not quite sure how you pronounce it, but "Euler buckling." Now I just want to take you to what Professor Priestley said about this issue of whether the building might have survived with a greater level of ductility and just see if you agree with this. This is at – well this is the section of his evidence, you probably remember where he went through a number of different sized columns for varying levels of ductility and so on and talk about how they would have responded, you remember him giving that evidence.

A. Yes.

Q. And then at the end of it I think in a response, in response to a question from me and Sir this is at day 58, this particular passage is at page 4, Professor Priestley said, or this is just starting at the bottom of page 3, "It's important to emphasise that the reinforcement in the columns as designed there would also have to be in the joints, so if you had just reinforced the columns without reinforcing the joints, they would not survive, but my belief is that if the joint had also been reinforced with additional spiral reinforcement then the – I can't say that the structure would have survived but I can say that the displacement capacities would have exceeded that predicted in the time history analyses and therefore the time history analyses would not have predicted failure." Now I'd just like to know whether you agree with that comment?

A. I should read this again.

Q. Would you like it to be – would you like it?

A. If you did have it, yes.

Q. All right, this is the transcript day 58 at the beginning of page 3 and running over the top of page 4. So the key part of it is there at the top of that page.

A. Top of page 4?

Q. Yes.

A. Okay I can see that. So I understand that, I don't think that alters what I say –

Q. Well could you first just tell us whether you agree or disagree with that then you can go on and comment.

A. Okay, I don't agree with his conclusion there, and that's because, but I don't think we're that far different. There are some things in there that he says as a statement of fact. For example he says, "I can't say the structure would have survived but I can say the displacement capacities would have exceeded that predicted in the time history analyses and therefore the time history analysis would not have predicted failure.

Q. Mmm.

A. Well that's a statement of fact.

Q. Yes.

A. The time history analysis cannot predict this class of failure because it is a limited analysis. It is not doing true non-linear geometric analysis, it's doing a kluge on that which means it modifies the stiffness matrix to take into account secondary displacements but that's an indirect measure rather than a direct measure as a simplification, so –

JUSTICE COOPER:

Q. You've used a word that I don't understand. It sounded like a clue.

A. Kluge is a semi-technical term that computer buffs use where you basically kind of can't model something in precise ways so you do it in an indirect way.

Q. And how do you spell it?

A. K-L-U-G-E I believe.

Q. How do you – spell the whole word.

A. K-L-U-G-E.

Q. Sorry, you were in the middle of something I think.

CROSS-EXAMINATION CONTINUES: MR MILLS

A. So because of that the – it needs to be very clear to everybody that the non-linear time history analysis as using SAP2000 is not able to do this sort of prediction. It's got serious limitations but given the trade-offs that one must make with the level of complexity and the large number of components involved, then you – this is possibly still the best type of software to use if you persist in holding the belief that side sway is the major culprit. Now if you wanted to take it to that higher order we would probably need another order of magnitude of time which I understand is not available, and one would use a high level software such as Abacus and then you're fraught with calibrations and whole degrees of complexity to do with convergence and so on and so forth, so you go to those levels of analyses as a very last resort. If one was to look at how the analysis was done for the World Trade Centre, it took many, many years and eventually something like that would have been used.

Q. All right, but can I just ask, you said that you didn't agree with this as a matter of fact that was your first point wasn't it?

A. Right.

Q. But you said you didn't think that you were that far apart.

A. Well, yeah, and I can explain this a little bit more -

Q. Yes please.

A. – also using some of his diagrams so we may want to come to –

Q. Well I don't, I didn't intend it to take up that amount of time because I know we're up against the time with you leaving. I was just really trying to identify the extent to which you are apart on this. Is there a way of cutting through that a bit?

A. Yes, okay, let me be more direct then, I was trying to use some of his own evidence to destroy his own argument, but then if I can use my own that's probably sufficient.

Q. Mhm.

A. Basically if we go to the dark green lines on any one of my four diagrams.

Q. Want to tell us what those are?

A. Well the figure 3 point – your page 1.87 for example or let's try another one. Now I did point this out making my presentation earlier so if we can enlarge the lower figure and so I have the mouse pointing at the fourth floor, and up at this level here in this column as it's shown you're going to get a bending moment with tension on this side so it's going to come out here, if you were drawing the bending moment diagram, so I'm tracing what the moment diagram would look like with the mouse, okay so it's going to be out here, it's going to come out and then it's going to curl around and then come back here like this. Now the way I've traced that is that you have a high moment at this end which is called fixity.

Q. The end being the upper end of the green line?

A. The upper end of the green line, and then through this joint here.
Q. Which is your second joint down from the top of the green line?
A. Yes, that's a so-called inflexion point, that goes right through the joint itself and then out here there's another moment which is a high moment, same as this magnitude up here, but has at the slope of the moment diagram in here it's got no – there's no slope on it so in other words the line just in this region is vertical.

JUSTICE COOPER:
Q. So that's off the page.
A. Off the page yes, so what that symbolically means is that when it's flat like that there's no shear through this connection and then there's a mirror image of that about this plane and then it comes back down here. So basically what this means is you can have this well reinforced, and regardless of what's going on here, it doesn't need to have good or bad detailing in that connection. It can be anything you like, in fact if it's good it doesn't really, it's silent on the matter. The point is that it's the slenderness of this piece of member in here that really is what counts.

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CROSS-EXAMINATION CONTINUES: MR MILLS
Q. Yes.
A. And so that's more to do with the size of the column and how badly damaged it has been because if it's damaged the effective stiffness, the EI value, essentially decreases after increasing number of cycles of loading. So, for example, if the columns themselves between say this level and this level went out to a ductility factor of about four, for example, using the so-called Takeda model which generally what can be used for that class of structural element. Now that's an element in the software programme, Sir, of SAP 2000. When you use that class of element when it unloads the unloading stiffness would be governed by about the spare root of the ductility factor so if it goes out to a ductility of

4, the square root of 4 is 2, 1 over 2 is one half. So it becomes, in effect one half as stiff which means –

COMMISSIONER FENWICK:

Q. Excuse me Dr Mander, this is a column in the structural roots with shear walls. How do you get to a ductility of 4 in the column when they're restrained by shear walls.

A. I'm just illustrating a point that –

Q. Well I think you've got to illustrate the point with connection to the CTV building not an arbitrary building.

A. Okay, so admittedly the ductility amplitudes may not have been as high although the latest results are showing out to about 3% drift and we would assume is a yield drift of about a half percent. I don't think an inter-storey drift as registered in the columns of about 3% is beyond the pale so what that essentially means is that the column itself will become unstable. Just using the Euler buckling formula.

Q. Excuse me again Dr Mander. The typical strength of the column at the moment in the lower floors as I understand is about 200 and something kilonewton metres and the dead load axial load I believe does not, steady state, axial load does not rise above about 1500 kilonewtons. So that implies you can have round about 300mm before you get to that instability state. Vertical excitation, just before you get onto this, will not help you because it's impulsive what increases in one way will reduce it in the next point one of a second. So how are you getting Euler buckling if you have got this full strength well confined the whole way down. You've got to account for that order of deformation. Don't give me a theoretical answer.

A. I understand that. It's going to deform and then because of geometric shortening the floors are going to come down which in turn is going to put distress on the neighbouring columns and I agree that it is more difficult for it to happen with well confined elements because it's not going to fail so suddenly but it could eventually fail so I suppose –

Q. Professor Mander, let's go back. You're saying it's going to shorten, and I agree it will, by how much do you think it's going to shorten and will that not be able to be taken up by the transverse beams to stop the load transfer you're talking about. Let's keep this real please.

A. Okay well yes that's certainly possible. Admittedly it won't shorten a great deal but nevertheless I still believe that the displacements need not be all that large even if they are confined but they can be problematic and in the case of the confined columns if the cover concrete is coming off and particularly well in the outermost bulge in my diagram there that's when you're under high moment and high axial load, albeit momentarily, the stiffness is going to be somewhat reduced because if it's well detailed there will be plastic hinges at the end. Once the cover comes off then again the EI effect for the section is somewhat reduced compared with what the gross one is, coupled with the softening of cyclic loading effects so I don't believe that it can be ruled out as a impossibility.

Q. Professor Mander, just again, you're saying all this softening. You agree these columns run to quite high axial loads?

A. Yes.

Q. Where's the softening come from?

A. The softening comes from the repeated loading, cyclic loading of the structural elements, that they do soften under cyclic loading.

Q. How much?

A. Well there's the –

Q. – the ends you're pointing there.

A. There's the inherent P-Delta itself as well which, of course, is tantamount to buckling. Like, as I said, I think this is quite similar to just plain P-Delta between one storey and the next but it is a buckling failure which doesn't necessarily rely on bulk side-sway.

CROSS-EXAMINATION CONTINUES: MR MILLS

Q. Well we didn't get a level of consensus between you and Professor Priestley but I'll move on from that. Just then one final issue just to come back to these hypotheses that you've put up. I think where we got to yesterday, tell me again if this captures where your evidence got to yesterday, that you're now saying that you're not using these hypotheses that you're putting forward to dismiss hypotheses that have been put up by other witnesses?

A. No.

Q. What you're saying is that, however, these hypotheses that I've put up shouldn't be overlooked?

A. Yes, yes.

Q. And would I be right though that you have quite a high level of confidence in the hypotheses that you're putting forward?

A. Well if I'm to look at how they compare with the other ones, I really think it's number 2 in the DBH Report if I'm not mistaken from memory. It's quite similar to that. It's certainly not number one. I think it's a case that my looking at it with respect to Mr Holmes and Professor Priestley we're more or less seeing the same thing but coming to the conclusions for slightly different reasons and I think the reasons are relatively slight and I wouldn't like to make a big deal about the differences. It's just that I believe that you need to keep an open mind and the point that I was trying to make here is that unfortunately the software is kind of silent on these matters so it predisposes anybody doing the analysis to go in search of the side-sway mechanism whereas in fact it can come straight down.

Q. Yes, so in the end you're saying I think that your hypothesis, your favoured hypothesis, is grouping somewhere around also where Professor Priestley and Mr Holmes are?

A. Oh absolutely. We're not that far apart. It sounds like we may be miles apart but I can assure you we're not.

Q. And you haven't felt it necessary to do a shake-table test to arrive at these hypotheses have you?

A. It would certainly be helpful.

Q. You haven't felt it necessary to do it?

A. No I haven't done it but if one really wants to learn about what happens, it's a good way to go.

CROSS-EXAMINATION: MR ELLIOTT

Q. Professor Mander my questions relate to the issue of design objective and I'm just going to refer you to a couple of passages from your evidence.

Firstly, WIT.MANDER.0001.89. So referring there to paragraph 4.2. Your statement is that when the Darfield earthquake struck it imposed ground accelerations that were essentially similar to the design limits for which the structure of the building had been designed. As a consequence, the structure was damaged. Such damage would be expected by design and the structure did not collapse and met its design objective of ensuring life safety.

Secondly, WIT.MANDER.0001.45 and if the third paragraph could be enlarged please, the one below the indented section. So your comment there is, "Compared to code-based design motion the CTV building site withstood much higher than expected horizontal ground motions. Any structure to survive such a high level of shaking is a bonus. It was certainly not a requirement at the time the CTV building was designed and constructed."

I'm going to ask you some questions about those statements. Firstly you're aware you that Alan Reay Consulting Limited lodged a letter containing a schedule of comments with the Department of Building and Housing in December 2011?

A. Yes.

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Q. Did you see that before you prepared your submissions?

A. I have seen it and I don't recall if it was before or after. Either case I didn't find it particularly helpful or useful to me at the time. I tried to form my own view independently.

Q. I'm just going to show you one passage from that BUI.MD249.0195A.11. I'll just read it to you professor while it's coming up. So professor I'm just referring to the left-hand column 86BCR. I'm just going to ask for that whole row of comments if possible to be enlarged. So the left-hand side says and this is a quotation from the draft Hyland Smith report. "It is important to recognise that the expectation of design standards in construction is that even at the attainment of the maximum drift levels there should still be a low probability of collapse occurring." And then do you see at the right-hand side the comment from Alan Reay Consulting Limited, "That the code of the day (and incidentally the current codes) do not require checks beyond the design basis earthquake." And the observation that "if the expectation of design standards is that there is a low probability of collapse at 'maximum drift levels' then this lack of checking in the design codes is a serious deficiency." So for the purpose of this discussion I think what is meant there is that maximum drift levels occurring in excessive design level shaking so would you agree that the comment that Alan Reay Consultants Limited has made is similar to the sentiments that you had expressed in the two comments that I quoted earlier?

A. Yes it is and I hadn't realised that.

Q. Well it's not a criticism. It's just to identify what the position is. Do you agree that it's implicit in both what you and Alan Reay Consulting are saying there is that when considering the requirements of the code the designers of the CTV building were not required to turn their minds to how the building would behave in an earthquake producing shaking above design levels. It must be mustn't it?

A. I think that's what they're saying.

Q. And that's what you're saying too isn't it?

A. Well again it's the bogey between code compliance and best practice and where a lot of the earthquake engineering came from and that found its way to New Zealand I think maybe Professor Shepherd mentioned this was the blue book in California which had three objectives.

Q. Just pause there I'm not really wanting to hear about the blue book from California.
A. No but –
Q. My question is directed to code issues in New Zealand.
A. Okay.
Q. So can we proceed on that basis please and my question was just do you agree that your sentiments in effect similar to what they say about the code as opposed to best practice, yes or no?
A. Yes.
Q. And I'm just going to ask you some questions then about that particular issue. You've referred in your evidence to the east-west components of the CTV building and the description that you used was that of a dual system, that's right isn't it?
A. Yes.
Q. And in response I think to a point from Commissioner Fenwick you referred to the frame, moment frames?
A. That's right yes.
Q. Now the frames consist of beams and columns, don't they?
A. Yes.
Q. And that frame system is one of the dual components that you are referring to, is that right?
A. Yes.
Q. And the other of the, the other part of the dual system was the shear walls in north and the south.
A. Yes.
Q. That dual system was connected –
A. Yes.
Q. - wasn't it? I'm going to ask you to identify those parts of the CTV building which would have been called upon to resist earthquake loads in design level shaking. Consider here the east-west motion as that's the motion that we've just discussed. Just tell me yes or no would these parts have been called upon to resist earthquake loading - the frames?
A. Not by design no.

Q. Not by?
A. Design.
Q. By design level shaking?
A. No.
Q. What do you mean?
A. The designer would have ignored their resistance as part of design.
Q. That's not my question. My question is when the building was exposed to design level shaking.
A. Yes.
Q. Which parts of it would have been called upon to resist earthquake loads? Would one of those parts have been the frames?
A. Yes.
Q. And would have another part have been the north core?
A. Yes.
Q. And another part would have been the south wall?
A. Yes.
Q. And the beam column connections would have also have been called upon?
A. Yes.
Q. Are there any other parts which would have been called upon to resist seismic loads in design level shaking?
A. Yes.
Q. What are they?
A. Foundations.
Q. Anything else?
A. Well if you're putting it like that I suppose even the internal fittings, the west wall, anything that can provide accidental resistance is going to provide resistance. It may not have been designed for it but it will provide some resistance.
Q. By contrast there are parts of the building which would not resist earthquake loads is that right?
A. I suppose yes.
Q. For example the spandrel panel would that be right?

A. They evidently contributed. Like if it's showing some damage obviously it tried to provide some load. And then it broke in the process.
Q. Now would you consider please the parts of the building that were necessary for the survival of the building as a whole under lateral seismic loading. By survival you can take that to mean there would have been no full or partial collapse and again just tell me yes or no – the columns?
A. Yes.
Q. Beam column connections?
A. Yes.
Q. North core?
A. Yes.
Q. South wall?
A. Yes.
Q. Beams?
A. Yes.
Q. Connections between floors and the north and south walls?
A. Yes.
Q. So again you're really referring to those two parts of the dual system, that is the shear wall structure and the moment resisting frame?
A. Yes.
Q. Is that right?
A. Yes.
Q. Of the parts of the CTV which would have been called upon to resist earthquake loads, which would have been the ones whose failure presented a risk to life – the columns yes or no?
A. Yes.
Q. Beam column connections?
A. Yes.
Q. North core?
A. No.
Q. South wall?
A. No.

Q. I'm going to ask you now some questions about the bonus comment that I have referred you to earlier on.
A. Yes.
Q. Now do you agree that other building in the CBD failed in the same way as the CTV did on the 22nd of February?
A. I think PGC's getting close in part.
Q. Apart from the PGC?
A. Well apart from that not to my knowledge.
Q. So using your description was that just a bonus that no other building failed in the same way?
A. I think so in many respects.
Q. You are familiar enough to comment are you on NZS3101 1982 parts one and two?
A. Yes.
Q. I can refer you to some comments in that but I'll just see if you agree before having to do so. Do you agree that in strength design procedure margin of structural safety is provided by multiplying service loads by load factors?
A. Yes.
Q. By using strength reduction factors?
A. Yes.
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Q. Do you agree that where transverse reinforcement is present in the columns it can increase the strength of the columns?
A. No. It increases the ductility not the strength.
Q. So would you agree that in relation to those points, ductility and strength reduction factors and load factors, they are safety checks if you like within the code which give a building a better chance if followed of surviving earthquake shaking above design level?
A. Now that's a tricky one. It's to do with how codes are calibrated and that's quite a complex process. I don't know if you would want me to elaborate on that?

Q. No, there's no need to elaborate. I mean really it was just a question of whether you would think that those things would provide an explanation about why every other building in the CBD did not collapse?
A. Oh, I see, yeah okay, so it is, they, all of those reserve capacities for other reasons often end up contributing as to why buildings see less damage than experience, than you might get by prediction or by design. But the biggest thing I think is from materials overstrength. That's the largest factor.
Q. I'm just going to refer you to some sections in NZS3101, firstly I can show this to you if you like but do you –

JUSTICE COOPER:

It might help.

CROSS-EXAMINATION CONTINUES: MR ELLIOTT

Q. It might help if I show, ENG.STA.0016.24?

WITNESS REFERRED TO SLIDE

Q. This was just to confirm Professor that this section of the code refers to principles and requirements for the analysis and design of the structure subjected to seismic loading, you can see that?
A. Right, yes.
Q. And you agree that the CTV was a structure subjected to seismic loading?
A. Yes.
Q. I'm going to refer you to some of the commentary to that section which is ENG.STA.0016A.22?

WITNESS REFERRED TO SLIDE

Q. If the section at the right-hand column, the top half of the right-hand column could be enlarged please? So do you see there that commentary here is saying that, "To clarify the meaning of various loads referred to in the standards…" et cetera, "…the following definitions should be considered to apply," I'm just going to refer you to some of those definitions without necessarily asking you too many questions.

Firstly you see, "Code loads are defined as those specified by NZS4203." Would that be the load the building must resist in a earthquake producing design level ground accelerations?

A. Yes.

Q. Now refer you to the third point there, capacity loads. You see they are those that, "As a result of extreme seismic displacements would be required to develop the flexural over-strength of members, sub-assemblages or the entire structure as appropriate, in accordance with the principles of capacity design." I'm going to come back to capacity design, but you would agree wouldn't you that we see here a distinction between code loads in that first definition and loads imposed as a result of extreme seismic displacements, would you agree?

A. Yes.

Q. And you would also agree I think that based on that second definition, capacity loads relate to loads imposed due to extreme seismic displacements?

A. Yes.

Q. I refer you now to ENG.STA.0016.18?

WITNESS REFERRED TO SLIDE

Q. Referring to the definition down the bottom right-hand corner. This is the definition section of NZS4203 1984 and the definition down the bottom, bottom right can be enlarged please? I may have misquoted, I think this is from NZS3101 I'm sorry. The design load is defined as, "Combinations of factored loads used in design as set out in NZS4203 or other appropriate loadings code. In seismic design the design load may be either the factored load or the load resulting from the capacity design procedure, depending on the case being considered." So we can observe from that, do you agree, that design load can be either factored load when strength method is used, or the load resulting from the capacity design procedure, is that right?

A. Well –

Q. Based on that definition?

A. They go hand in hand.

Q. Now I refer you now to ENG.STA.0016.24?

WITNESS REFERRED TO SLIDE

Q. And if clause 3.5.1.3 could be enlarged? This says, "Wherever the requirements of the capacity design procedure apply the maximum member actions to be expected during large inelastic deformations of a structure shall be based on the overstrength of the potential plastic hinges." I'm not going to ask you to explain that, I'd just like you to note it. And I'm going to refer you now to the commentary on that clause which is ENG.STA.0016A.23, and if that commentary clause could be enlarged please? C 3.5.1.3 on the left-hand side? So just referring you to a sentence which is about halfway down there, "The actions derived in the capacity procedure will generally be greater than the corresponding actions calculated from the application of the designed seismic loads of NZS4203 or other appropriate code." Now I appreciate there's a bit of confusion probably entering here because they refer to "design seismic loads" and we've just had a discussion about that, but am I right in saying that actions mean the effects on a member due to earthquake loads, for example bending moments?

A. Yes.

Q. Is that right?

A. I think so.

Q. So seismic loads lead to actions such as bending moments, is that right?

A. Yes.

Q. And higher loads will result in greater bending moments, is that right?

A. No. Not necessarily, it could yield and then it'll be the same.

Q. I see, so if it doesn't yield does it result in higher bending moments?

A. No, it yield, when it yields and then you get inelastic performance and you get a plastic moment and you'll get increased strain, so you get energy dissipation.

Q. I see.

A. So it's the duality between the two that's important.

Q. Well I'm just really asking you to explain what you would say this sentence means, the actions et cetera. Does it mean that when capacity design applies the designer must consider member actions which may be greater than those which may result from the imposition of loads set out in NZS4203? I suppose the answer must be yes?
A. Yes that's correct.
Q. So does it follow from that that the designer is called upon to consider implications on members of ground accelerations greater than design level ground accelerations?
A. No absolutely not, it doesn't mean that.
Q. Can you explain why?
A. This is the interplay between loads and overstrength have to do with getting an accidental undesirable failure. Okay, so for example the principal design preset in New Zealand codes is that you, the designer conceives a desirable failure mechanism and the favoured one is a beam side sway mechanism, and so the beams become the weak link in the chain and you design the beams accordingly for their specified strength.

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Q. Yes.
A. And sometimes some members will have an under capacity in their design and then what you do is you remove that and then you put in deliberately higher strengths to get the probably upper bound level of the capacity, then you use that information of the beam overstrength to design the column. So you really only look at the earthquake loads once and the earthquake loads, well you put those on the structure, you determine your bending, design bending moments as per the earthquake loads. There are two of them as you point out, there's one that's above the normal load of the self-weight and you, like different codes around the world use different factors, but commonly it's about – it's some measure of the live load plus the dead load, and it's all about getting what probably might happen with concurrence with an earthquake so it's not likely that you're going to get the extreme gravity

load that you would get with an earthquake but you will get, there will be some probable load that's an upper bound, and then you also consider a lower bound case where you take not even the full gravity load. You take about 90 percent of it because that's on the premise that a builder can build the building as per spec but not all the fittings are in there so you're only about 90 percent of the load. So those are your two conditions and you only apply those to the earthquake loading which then are used to factor up the bending moments, well you get your bending moments, you design your reinforcing steel for those bending moments and we've had a lot of discussion about specified material properties but in the case of the steel you take the specified strength, in this case it would have been 380 megapascal yield strength, but then you amplify it knowing full well that that is like at the fifth percentile and that is a high probability, well 95 percent chance that it'll be stronger than that so you take a probable higher strength and then you use that information where you factor it up to get the overstrength of the beams and then use that to design the columns for moments. So that would desize the longitudinal column steel and then you factor that up yet again to make sure that it doesn't fail in shear.

Q. Right, so what you're describing there is a capacity design procedure.

A. It is.

Q. Now I just want to ask you some questions about that because you referred to that in your presentation.

A. Right.

Q. You've gone through it quite quickly, especially for a lay person so I'm just going to step you through it. Now to talk about capacity design though can we just identify what a plastic hinge is and I'll ask you to confirm what it is by reference to a definition in the code which is BUI.STA.0016.28. What about .22? I'll just read it to you, it's one sentence long professor. "Plastic hinge region, regions in a member as defined in this code where significant rotations due to inelastic strains can develop under flexural actions." So I take it you'd agree with that definition of plastic hinge?

A. Yes.

Q. So it's a point at which there can be rotations, they are inelastic and they develop under flexural action?

A. Yes.

Q. Agreed. And you've actually shown I think potential plastic hinge regions in the CTV by reference to a diagram in your evidence?

A. Yes.

Q. WIT.MANDER.0001.65. So by reference to that diagram you've identified them as being at the tops and bottoms of columns?

A. Yes.

Q. And the ends of beams?

A. Not all beams.

Q. Some beams?

A. Yes.

Q. And the base of the wall in this case?

A. Base of the wall definitely and also the base of the columns too.

Q. So in this particular building which you've portrayed here, given that there was I think a structural type factor S=1, is it right that the building was designed so that ductile yielding would take place at levels below design level shaking?

A. I don't believe that's the intention. I believe the intention was that the walls were designed to carry the load and then the columns checked that they won't go inelastic. Now if I'm right in thinking where you're heading with this is that there is no reserve capacity, that's the problem and I think that's part of the – well it's part of the main problem is that once you get beyond the design level of loads there's no reserves in terms of squeezing energy out specifically.

Q. Well that is something that I wanted to ask you about, but the other issue is just the compatibility of a capacity design procedure with the approach that seems to have been adopted with this building.

A. Right.

Q. That's, they're the two issues I'm looking to explore.

A. Right, okay.

Q. And just looking at that latter one dealing with capacity design procedure, am I right in saying that that was a capacity, was a provision that was introduced into NZS4203 1984?

A. Yes.

Q. And I just refer you to STA.00015.11 which is a section of the introduction to NZS4203. If the third paragraph could be highlighted please, the one beginning ‘An important new provision’. I’ll just ask you to read that.

A. Okay.

Q. So just in relation to that last sentence you’d agree that that notes the level of seismic coefficients have not been altered, that's right isn't it, and then there's a comment secondly that levels are considered satisfactory only where the relevant design standards provide an acceptable degree of ductility. That latter reference to ductility must relate to those parts of the building called upon to resist earthquake loads mustn’t it?

A. Yes.

Q. So it therefore must have included reference beams and columns mustn’t it?

A. One would hope so.

Q. Just while we're on that section, if the next paragraph could be highlighted please. I’ll just ask you to read that to yourself please.

A. Yes.

Q. So you’d agree that that points out to the designer that the precise properties of construction materials are unknown?

A. Mhm.

Q. Agreed, that properties of structural elements made from them are not clearly known?

A. Yes.

Q. Agreed?

A. Yes.

Q. That the interaction of these elements in a building frame is uncertain. Is that right?

A. Yes.
Q. And that design involves imprecision, agreed?
A. Ah, yes.
Q. So would you agree that what that paragraph is doing is urging a designer to take a cautious approach to building design?
A. Yes.
Q. And especially in light of the previous paragraph, isn't it urging the designer to be cautious about making decisions about which parts of a building should be designed for ductility or not?
A. Yes.

1505

Q. So the purpose of capacity design, do you agree, is that brittle failure modes are prevented and ductile behaviour is ensured in a major earthquake?
A. Yes.
Q. And that in capacity design what one is seeking to do is to design the structure so that inelastic deformation occurs in defined locations?
A. Yes.
Q. And those inelastic deformations are taking place in the plastic hinge regions?
A. Yes.
Q. So in capacity design the designer is asking in which of those potential plastic hinges do I want inelastic deformations to take place in an earthquake. Is that right?
A. Yes that's correct.
Q. And I think as you were describing before, perhaps in more detail than I understood, were you saying all structural elements outside of those plastic hinges are designed to have greater strength than the nominated plastic hinge area?
A. Yes, they are typically 40% stronger.
Q. But it’s also true that the nominated plastic hinge region must be robust enough to ensure the building will not fail –
A. Yes.

Q. – in a way that causes injury or death?

A. Yes.

Q. Do you agree that a designer should be determined to tell the structure what to do in an earthquake?

A. Yes that is a Tom Paulay statement that we –

Q. That's where it came from, and do you agree that that should ensure excellent inelastic response provided that as a complementary task all critical regions are judiciously detailed?

A. That sounds like it's straight out of his book, yes I agree.

Q. And it's the capacity design is doing exactly that isn't it, the designer telling the structure what to do in an earthquake?

A. Yes.

Q. In your statement you set out the strength hierarchy of the CTV building –

A. Yes.

Q. – in which you said the wall was strongest, followed by beam flexure, followed by column flexure, followed by joint shear and that hierarchy would have been the reverse of what a designer would be looking to do in applying capacity design?

A. Yes.

Q. So it's evident that what the designers of the CTV building were telling it to do in the face of earthquake forces above code level was to fail in the beam column joints first and then fail in the columns, which is exactly what happened?

A. Um, I don't know if I would be so strong as to say they were telling that. I've got no idea what was in their head when they came up with that reasoning.

Q. That's the effect of their design isn't it?

A. But it looks like that, yes.

Q. You talk about what the strength hierarchy would normally be and you refer to beam bending, followed by column bending, followed by joint shear, followed by foundation from weak to strong. Is that right?

A. Yes.

Q. That is consistent with capacity design isn't it?

A. It is.

Q. And as part of the process one is wanting to ensure that the plastic hinge regions where the inelastic deformations will take place would be in the beams?

A. Yes.

Q. And the reason for that is because if the beam hinges then there's a much better chance of people surviving than if the columns hinge first isn't it?

A. Not necessarily. This is a debatable point. It's just that when you have a beam side-sway mechanism you typically have a lot more column, I mean you have a large number of plastic hinges participating in the seismic response so I don't want people to think that the diagram that I drew earlier with the hinges at the column ends is actually valid and that would be an okay way to design and detail a building. The difficulty is, and sometimes this happens with a modern rendition of pre-cast concrete structures where you want to have a very long pre-cast concrete units and beams so they're brought in and essentially all columns are used as props but you detail them accordingly for ductility so it's a bit like rotating the whole paradigm around 90^{o} and so whereas you would normally get hinges at the beam ends you then have them on the column ends and that's actually valid. It's not often done but I have seen it done and the key and the trick is, of course, to make sure that you do have a sufficient amount of transverse reinforcement.

Q. That's right isn't it in that case?

A. Yes.

Q. If a designer was sitting down deciding where do I want my plastic hinges to be and I won't have transverse reinforcement, would you want them to be in the beams or the columns?

A. Well that's an interesting question because in a later code I believe, or may even be in about this time, one of the practices that was permitted that wasn't permitted in the earlier rendition that New Zealanders were using which was really the ACI 318.71 or 77, as I recall, and that particular code from which NZS 3101 grew out from was not overtly clear on capacity design. That is really a New Zealand phenomena that was developed at Beca Carter by John Hollings.

Q. In the 1960s?

A. Yes, so the Californian well American practice, more specifically California where most of this was being used, would permit columns to be detailed with large amounts of transverse reinforcement. There is an equation in there. As I recall it was equation A1 in the Appendix of that code which said that the columns had to be six-fifths of the beam of the girders, framing into the joint. Now that didn't guarantee that was not capacity design either by stealth or anything else. It was notionally stronger but not significantly stronger and so in the American practice both the beams and the columns had to be designed for a high degree of ductility. The New Zealand Code actually relaxed that on recommendations and research from Professor Paulay that permitted having lapsed glasses at the floor level and this was again for ease of construction but in order to do that –

Q. You're talking about a later code?

A. I'm not sure actually.

Q. If you're not sure then I think...

A. Well I believe it's relevant though because having like you said that the columns had to be stronger or detailed, I think it's a matter that you could make the columns actually stronger to avoid having some of that transverse steel.

Q. Well put it this way, if you were sitting down using capacity design to design a building like this and you had a choice between your columns or your beams having inelastic deformations and you were conscious of life safety, which of the two would you prefer the inelastic deformations to be at – it must be the beams mustn't it?

A. No. It's tempting to say that but I would always say and it's very easy for structural engineers to be neglectful of how the construction is taking place and I think that's important to bring into it is the method of construction, construction is important in that the architect or the constructor may have a desire to have these very long span units where it's very difficult to actually build beam hinges that are of a modest strength like the gravity load effects dominate the behaviour and in turn you end up getting extraordinarily unnecessarily strong columns from a gravity point of view just to make this whole strong column weak beam paradigm work and you get into major problems when the spans get up to and beyond 10 or 12 metres so it's okay to have the opposite view. It's okay, in other words, to detail the columns providing they are detailed for a high level of ductility but I would agree that in the first instance it is desirable to have weak beams strong columns because if you have a failure it is likely to be a localised failure rather than a general failure.

COMMISSIONER FENWICK:

Q. Can I just interrupt here. I think what you're saying, as I understand it, if you have a moment frame building of several storeys then you should go for beam yielding and the columns and the joint zones should be stronger?

A. Yes.

Q. But if you have a structural wall building then it's acceptable for plastic hinges to form in the columns because the wall will control the deformation levels and therefore whether they form in the beams or the columns is a bit immaterial. Is that -

A. Well that's also true, that's also true, yes I agree with that.

1515

CROSS-EXAMINATION CONTINUES: MR ELLIOTT

Q. Well considering a building like the CTV where we had this dual system – just try and confine your answer to that type of building.

A. Right.

Q. Would you agree that the designer approaching that building and using capacity design would need to consider the building as a whole and not just different parts of it in isolation?

A. Yes.

Q. And that because it was a dual system they would need to treat the columns and beams as part of the lateral load resisting structure.

A. Yes, yes.

Q. It would be artificial just to say well we'll use capacity design at the north core and/or the south wall and ignore, put to one side, the beams and columns wouldn't it?

A. I believe so.

WITNESS REFERRED TO DOCUMENT ENG.STA.0016.24 – CLAUSE 3.5.3.2

Q. I just want to ask you about this clause and its implications, if you'd just like to read it to yourself. So this is in the context of what's described as a severe earthquake and you agree the cause is saying that the structure shall be assumed to be forced into lateral deformations.

A. Yes.

Q. And that the lateral deformations are assumed to be sufficient to create plastic hinges.

A. Yes.

Q. And that those plastic hinges are considered to be reversible. Agreed?

A. Yes.

Q. And reversible means the hinge goes in both directions in the same plane, is that right?

A. Yes.

Q. So severe earthquake could mean two different things I suppose. I'll just ask you about both possibilities. If severe earthquake in this context meant an earthquake above design level then the code is telling the designer to make various assumptions about the performance of the building above design level, is that right?

A. If it means that.

Q. If a severe earthquake means a design level quake it's saying that one must assume there'll be plastic hinges which are reversible in the building, is that right?

A. Yes.

Q. And isn't it saying, therefore, given that we've identified plastic hinges, that the designer must have assumed that the columns would form plastic hinges which are reversible as well?
A. Yes.
Q. So it's telling the designer to assume that the columns will need to be ductile.
A. Indirectly it is.
Q. Indirectly, mmm. You'd be aware that the seismic provisions of the code in relation to columns identify potential plastic hinge regions in the ends of columns.
A. Yes.
Q. And that the clause in the Code, in fact 6.5.4.3 sets out transverse reinforcement provisions in potential plastic hinge regions and columns.
A. Yes.
Q. And you'd be aware that s 9.5 of NZS3101 relates to joints designed for seismic loading?
A. Say that one again please?
Q. I can show you if you like but s 9.5 of NZS3101 relates to joints designed for seismic loading.
A. Yes, yes, okay.
Q. And you'd be aware that one of the clauses there, in fact, 9.5.6.1 says that horizontal transverse confinement in beam column joints designed for seismic loading shall not be less than the transverse requirements specified for columns. Are you aware of that?
A. I'm aware of that.
Q. So if capacity design applied to the structure, just to summarise the position, you'd agree that the designers of the building would have been required to consider the behaviour the building as a whole, when exposed to earthquake loads, that's right?
A. Yes.
Q. You've agreed that designers of the building would have assumed or should have assumed that the columns would be called upon to resist lateral earthquake loads in at least the east-west direction.

A. Yes.

Q. And that the designers should have regarded the columns as being a risk to life safety in the event of failure.

A. Yes.

Q. And shouldn't the designers have identified the ends of columns, as you've done, as potential plastic hinge regions?

A. Yeah I think they should have.

Q. And, in fact, they should have assumed that reversible plastic hinges would form in those parts of the columns in a severe earthquake.

A. Yes.

Q. So it's true, isn't it, that they should have specified the transverse reinforcement set out in the Code for those plastic hinge regions?

A. I believe so.

Q. And they should also have designed for the same confinement in the beam column connections.

A. Yes.

Q. And there would be no reason to limit the ductility of columns due to any limitation on inter-storey displacement would there?

A. Sorry repeat that.

Q. There would be no reason for a person to go looking for some reason to limit the ductility of columns by virtue of any inter-storey displacement.

A. No you don't have to do a deflection check to my knowledge for that.

Q. You've referred to Euler buckling – assuming that the columns in the beam column connections had the type of seismic transverse reinforcement that we've just discussed would that type of reinforcement have made the building more or less susceptible to Euler buckling?

A. It would delay the complete catastrophic collapse due to Euler buckling but it wouldn't necessarily inhibit the onset if the columns were slender enough.

Q. So it would make it less susceptible to collapse in that it delays the collapse.

A. It delays it.

Q. You've said that, well perhaps you haven't said, Euler buckling refers to a formula by a person named Euler, is that right? He was around in the 1700s is that right?

A. He was.

Q. And you've said that Euler buckling could have occurred without large displacements.

A. Well it's a what we call an igon value problem. That means that when the stiffness of the member, combined with axial load, is insufficient to resist a large degree of axial force then the column will, well it will collapse. It can't, it ceases to take that load so instead of getting overloaded it's going to buckle.

Q. The point I was getting to is, on the basis that what you said is correct that it can occur without large lateral displacements. That would illustrate that lateral displacements are not something which should be determinative of whether to apply ductile detailing to a column or not.

A. Sorry you'll have to put that one by me again.

Q. Well doesn't the fact that Euler buckling can occur without large lateral displacements illustrate that lateral displacements should not be determinative of whether to apply ductile detailing to a column.

A. No that doesn't. The two are unconnected in my view. One has to do with the size of the column and then the other has to do with the expected deformation or required deformation capability of the column.

Q. You've referred to a loophole in the Code and I think we need to identify where that loophole is so that it can be confronted. Leading into that I think, based on what you've said, you've identified there are provisions in the code for non-ductile detailing the columns. That's right isn't it?

A. Right.

1525

Q. Now what do you say to the suggestion that in a six level commercial structure like this which is designed for ductile flexural yielding, that there is no place in that type of structure for non-ductile columns per se?

A. I would agree with that, again this I believe is the best practice thing. It goes, it may include but it goes beyond the code in that I think, you know, that's just the right thing to do, personally, personally.

Q. Yes, non-ductile detailing for columns may have a place in a building with one or two levels, would that be right? A smaller building?

A. Maybe, I know historically the intention was for more than two storeys because in two storeys whether you get a column side sway mechanism or beam side sway mechanism they essentially amount to much the same thing so it really doesn't matter which way round you put it, but whatever happens if, if you're weakening members to survive the earthquake and it has to be done through inelastic performance, so something has to go inelastic, therefore somewhere you will need closely spaced transverse reinforcement.

Q. So I'm just going to ask you to identify the loophole that you referred to earlier on and I think I know where you might have been referring to so I'll just see if I can take you to it?

A. Okay.

Q. You're referring to the secondary element provisions in NZS3101 are you?

A. Yes, yes, yes.

Q. Well just to take you there, firstly can I just show you the ENG.STA.0018.18?

WITNESS REFERRED TO SLIDE

Q. I'll just refer you to the definition of elements, if that can be highlighted, elements including primary and secondary elements. Would you just like to read that to yourself please?

A. Yes.

Q. I'm just going to ask you to keep that definition in mind as we embark upon this discussion of this particular section, particularly noting that the primary elements there includes beams, columns, shear walls. So the provision that I think you're referring to as this loophole is ENG.STA.0016.28.

HEARING ADJOURNS: 3.29 PM

HEARING RESUMES: 3.46 PM

CROSS-EXAMINATION CONTINUES: MR ELLIOTT

Q. Professor, given the time I just have three very brief issue which I can deal with. Firstly it was just to confirm that the loop hole, what you referred to as the loop hole is a reference to clause 3.5.14.3 of NZS3101 part one, is that right?

A. It appears to be so but I can't confirm without having seen the calculations right now as to whether or not the designer would have cited that as the reason for using it, but…

Q. In terms of your reference though to a loop hole that is what you –

A. Well it possibly is yes.

Q. Secondly just refer you to a section in your statement WIT.MANDER -

JUSTICE COOPER:

Q. Just let me clarify that last answer you are saying it possibly was a loop hole or you would say – are you saying it was able to be used properly in that way?

A. I don't believe so no but what I meant I'm not sure what the clause is where the deflection check is mentioned, if it is related to this or not.

Q. You're not sure?

A. Whether the clause that relates to this deflection check. I would probably start there and work backwards, change backwards.

Q. I'm still not following you. Just complete the sentence because you're speaking in a kind of shorthand.

A. The clause that requires the deflection check that the designer would have used to justify the pathway they took, I'm not sure what that is connected to in the document, in the code.

Q. Well can we display this provision in the code because it's seems to be the end point of a line of questioning Mr Elliott.

MR ELLIOTT:
Yes Your Honour but I'm conscious that it blends into code compliance.

JUSTICE COOPER:
Yes but the witness can legitimately ask to see the provision.

CROSS-EXAMINATION CONTINUES: MR ELLIOTT
Q. ENG.STA.0016.28. At the bottom right-hand corner 3.5.14.3.

JUSTICE COOPER:
Q. Is that the clause you're referring to Professor Mander?
A. Yes and that relates back to group two elements.
Q. They have to be in that category for it to apply.
A. Yes.
Q. So what do you conclude in relation to –

CROSS-EXAMINATION CONTINUES: MR ELLIOTT
A. Could you confirm or otherwise for me that what you're talking about is group two because I don't recall.
Q. The heading to the section in which that appears as secondary structural elements if we –
A. Group two does it say that?
Q. Well it says group two at the top of the clause.
A. Okay. This one does but the one that you read before.
Q. We could show you the whole page if that would help.
A. Well okay. Yes, yes, okay. They're using, they would be evidently using that escape route shall we say, invoking what they chose to do.

JUSTICE COOPER:

Q. But you don't agree with that approach?
A. No.

CROSS-EXAMINATION CONTINUES: MR ELLIOTT

Q. Secondly I was going to refer you to a section in your statement WIT.MANDER.0001.48. The paragraph beginning, "Although confined concrete columns..." you'll see there's a sentence in there, "It appears the deviance from ductile detailing in the concrete columns was contentious at the time the CTV building designer sought the building permit from the Council." You mentioned earlier on in evidence that you've read documents on the secure website of the Commission?
A. Yes.
Q. Now have you read the letter from Mr Tapper to Alan Reay Consulting Limited?
A. Yes.
Q. That letter does not raise deviance from ductile detailing in the columns as one of the issues does it?
A. I would need to see that again.
Q. Can I ask you this question: the statement that you've made that deviance was contentious, did you learn that from Dr Reay or from some other source?
A. No I was going on the evidence on the secure website. As I say I haven't really had any conversations with him over this so I can certainly vouch the fact that he hasn't primed the pump at all. What alerted my conscious to the contentiousness was the fact that the widow of one of the other gentlemen involved, I don't recall, has submitted some information and then there's the other fellow who I don't, again I'm terrible with names I'm sorry but you'll know who I mean, he worked for the Council and then he became the county engineer for Waimairi and then or was it Riccarton and then he was taking a lunchtime stroll looking at the building.
Q. So just to summarise you've inferred from documents that you've read that this was the case?

A. Yes.
Q. Finally it's a particular question on behalf of the families of those who died. You made a comment in your evidence that ARCL can feel vindicated because the structure survived the design level Darfield earthquake without collapse. You would agree that there's nothing in the performance of the building in the Christchurch earthquake for Dr Reay's firm to feel vindicated about?
A. Sorry for which earthquake? You mentioned Christchurch.
Q. The February earthquake?
A. February earthquake no, no.

RE-EXAMINATION: MR RENNIE – NIL

JUSTICE COOPER:

Professor Mander, there will be questions from the Commissioners. That will be something for you to look forward to when you return.

WITNESS STOOD DOWN

HEARING ADJOURNS: 3.55 PM

HEARING RESUMES: 3.58 PM

MR RENNIE:

The next witness is Dr Bradley and Dr Bradley has a short brief to read and then a PowerPoint to speak to. The matters addressed in the PowerPoint Sir, have as one might expect in a matter of this nature a considerable amount of numerical detail and we have actually uploaded as BRADLEY4.1 the material that the doctor will be speaking to in relation to the PowerPoint but he'll be addressing the PowerPoint Sir.

JUSTICE COOPER:

Can I just ask, just to make sure that I am keeping up with this, sorry, I will let you.

MR RENNIE CALLS
BRENDON ARCHIE BRADLEY (SWORN)

JUSTICE COOPER:

I am just going to have a discussion with Mr Rennie. Now there's a statement of evidence?

MR RENNIE:

That's 3.1 Sir.

JUSTICE COOPER:

Yes, and then there's a supplementary statement?

MR RENNIE:

That's the point I was just referring to Sir.

JUSTICE COOPER:

That's 4.1?

MR RENNIE:

That's document 4.1 and that is a written version of what Dr Bradley will be speaking to on the PowerPoint, and we provided that because he will be citing figures and formulae and so forth and it seemed that that was an effective way of making sure that you had available to you –

JUSTICE COOPER:

Well just let's just check this because the document which starts at 4.1 is 10 paragraphs of text and it has one of these response spectra graphs and various other things but there's only two pages of attachments.

MR RENNIE:

The spectra graphs Sir are slides 7, 8 and 9 of his evidence, of his PowerPoint.

JUSTICE COOPER:

Well there you are. We may not be on the same wavelength here. Can I just say this, I've got another supplementary statement which has the suffix 4.5?

MR RENNIE:

Yes Sir.

JUSTICE COOPER:

And that is more extensive and have more –

MR RENNIE:

That reflects slides 10 through 14 of the PowerPoint.

JUSTICE COOPER:

You don't make it easy. I know you're trying to but we've got sort of statements of evidence put together, collated from various sources. Now can I ask you another question so that we get a full grasp of what we're dealing with here. We've got documents which consist of graphs and depictions of various earthquake-related matters which have the suffix 524.1?

MR RENNIE:

Yes Sir that is the PowerPoint Sir which the first slide had previously appeared in is what it's intended Dr Bradley speak to.

JUSTICE COOPER:

And then another set which is numbered 524A.1 and following?

MR RENNIE:

My understanding Sir is that 524A is the current total set of the PowerPoint. So you may well have had an earlier version. We've got into this situation Sir by seeking to, I hesitate to use the word "abbreviate" in the circumstances but that was the objective Sir of how the doctor was going to present his evidence in the first place.

JUSTICE COOPER:

Well could I just ask the general assemblage of counsel whether we can dispose of the one, the set which is marked, which has the suffix 524.1 and replace it with 524A.1? Is there anybody there?

MR MILLS:

It's certainly the version that I've got.

JUSTICE COOPER:

You've got 524A.1 and you think that's the one we should be using?

MR RENNIE:

Correct Sir.

JUSTICE COOPER:

Right well now –

MR RENNIE:
Particularly because it's the one the doctor plans to use Sir.

JUSTICE COOPER:
Right, well the other ones might then, but well we can't dispose of the other one though can we because it's more extensive? It's got, it goes to 24 pages whereas the A.1 document goes to 18 pages?

MR RENNIE:
Yes, and the reason for that Sir was that there was at the point that that was filed some contention in respect of a couple of matters which are not now in contention and they have been dropped from that PowerPoint.

JUSTICE COOPER:
Okay.

MR RENNIE:
And that is why I'm saying Sir that –

JUSTICE COOPER:
Well I'm going to put a line, and ink line through 524.1 and I'm going to say it's been superceded.

MR RENNIE:
That's absolutely correct Sir.

JUSTICE COOPER:
So apart from that the position appears to be straightforward Mr Rennie?

MR RENNIE:
I've become sufficiently risk averse Sir that I don't think I'd even agree with that.

EXAMINATION: MR RENNIE

Q. Now doctor, you full name is Brendon Archie Bradley, you reside in Christchurch and you're a lecturer at the University of Canterbury and you also run your own seismic engineering consultancy business?
A. Correct.
Q. In accordance with the requirements of r 9.43 High Court Rules you confirm that you've read the Code Of Conduct For Expert Witnesses and that your evidence complies with the code's requirements?
A. Correct.
Q. Matters on which you express an opinion are within your field of expertise?
A. Correct.
Q. You have no interest or relationships with any parties to these proceedings?
A. Correct.
Q. Would you now read from paragraph 5 of your brief of evidence please?
A. I hold a Bachelor of Engineering, Honours, University of Canterbury, 2006 and a Doctorate of Philosophy in Civil Engineering, University of Canterbury, 2009.

I am a member of the New Zealand Society for Earthquake Engineering and other relevant professional bodies.

Since 2010 I have lectured at the University of Canterbury, Department of Civil and Natural Resources Engineering, in courses involving structural dynamics, earthquake mechanics and mathematics, geotechnical earthquake engineering and seismic hazard and risk analysis among others.

For 13 months during 2010 and 2011 I was a JSPS Postdoctoral Fellow in geotechnical earthquake engineering at Chuo University – Faculty of Science and Engineering. Between 2009 and 2010 I was employed by GNS Science as a seismic hazard modeller where I was engaged in research and consulting seismic hazard and risk analysis, tsunami reconnaissance, development of displacement-based fragility functions for earthquake and tsunami.

In 2010 I founded Bradley Seismic Limited, a consultancy firm through which I provide research and consultancy services and seismic hazard and risk analysis, structural and geotechnical seismic response analysis.

Throughout my education and professional career I have been awarded numerous scholarships and prizes, as detailed in my resumé. I have authored or co-authored 39 published journal articles, as well as numerous conference articles, books and book chapters and other publications, all of which are detailed in my resumé.

My expertise covers seismic hazard analysis, ground motion prediction and analysis and structure-specific seismic loss assessments.

My full resumé is attached and marked A.

I have been instructed by Buddle Findlay on behalf of Alan Reay Consultants Limited, ARCL, to provide independent expert advice on issues relevant to the collapse of the CTV building on 22nd of February 2011 following and earthquake of magnitude 6.3. In particular I have been asked to comment on the following issues: A –

Q. Just pause there doctor because A which is concrete is for a later part of the hearing. Today I think you're dealing with B.

A. So I should not read A?

Q. Just go to B.

A. That's fine. B, analysis of ground motion aspects of the Canterbury earthquakes.

The principal sources of information I have referred to and relied upon in preparing this evidence are referenced in my reports annexed.

1610

Q. Now we'll just pass over for the same reason the concrete material at paragraphs 15 to 18 and come on to 19.

A. Analyses of ground motions. My report 'Ground Motion Aspects of the 22nd of February 2011 Christchurch Earthquake Related to the Canterbury Television Building' is annexed and marked D.

Q. Now you intend to present that report in PowerPoint form?

A. That's correct.

Q. And can we go to slide 1 please, and starting from slide 1 I think we now need to go to slide 2 and would you just take it from there please doctor?

A. So this is reading from that report but I won't read every section of it, so related to this slide comes under section 2.1 of my report and to summarise that section, no instrumental records of the ground motions at the CTV site were obtained from the 4th of September 2010 Darfield and 22nd of February 2011 Christchurch earthquakes. However numerous ground motions were observed in the general vicinity of the CTV site. The four closest strong motion stations which are part of the permanent Geonet instrumentation network are shown on this slide as well as their approximate source to site distances which range from 720 metres, the closest station which is denoted as CCCC to 1850 metres to the farthest station denoted as CBGS.

JUSTICE COOPER:

Q. That CCCC site which is Catholic Cathedral College as I understand it, is that somewhat off Barbados Street is it?

A. That's as I understand it Sir, yes.

EXAMINATION CONTINUES: MR RENNIE

A. The next – the next comment related to the slide comes under section 2.2 of my report. While these distances that I've noted above ranging from 720 metres to 1850 metres may appear to be relatively close to the CTV site, ground motion characteristics can change appreciably over such distances, particularly at high frequencies or shortened vibration periods as will be elaborated upon.

Q. Can we go to slide 3?

A. This slide illustrates the ground motion accelerations recorded at various locations in Christchurch, as well as the surface projection of the inferred earthquake rupture plane shown as a rectangle. This is for the 22nd of February 2011 earthquake. It can be clearly seen from the slide even at a qualitative level that the characteristics and by that I mean the amplitude frequency content and duration of the ground motion is variable between recorded ground motions in close proximity to one another.

JUSTICE COOPER:

Q. Is that rectangle there the inferred location of the fault that ruptured on the 22^{nd} of February?

A. That's correct, that's the surface projection of the fault over which the majority of the slip occurred.

EXAMINATION CONTINUES: MR RENNIE

Q. Go to slide 4.

A. On the next slide, this slide represents the elastic pseudo acceleration response spectra of the ground motions recorded in the 22^{nd} of February 2011 earthquake at the four strong motion stations in close proximity to the CTV site. When viewing this slide it is immediately apparent that the response spectrum amplitudes are firstly well above the design amplitudes for a vibration period of one second which is approximately that at which the CTV building is deemed to behave in the lateral direction.

What is also important in examining this figure on the slide is that the variability and the response spectra of the ground motions observed at these four locations. It can be seen that the variability is a function of the vibration period considered with for example very little difference in the response spectra for periods greater than 2 seconds but differences on the order of a factor of two for periods in the range of 0.5 to 1.5 seconds. This large variability is significant given that the period of the CTV structure is estimated to be on the order of T=1 second and given that the exact ground motion at the CTV site is unknown.

The vibration period dependence of this variability noted in the previous paragraph is physically understood and occurs because of the fact that long period ground motion, that is ground motion which has long periods of vibration has significantly longer wave lengths than short period ground motion. Short period ground motion which as I've mentioned has very short wave lengths is significantly modified via reflection, refraction and amplification of the local geotechnical characteristics of the site. Furthermore because of the short wave lengths the incident high frequency ground motion below each site will also be different. In contrast long period ground motion is generally not significantly affected by near surface geotechnical characteristics and also the longer wave lengths mean that the incident ground motion below each site will be similar.

Q. Can we go to slide 5?

A. This relates to section 4 of my brief. A strong ground motion instrument was deployed at the CTV site in March 2012. While no ground motions were recorded at the CTV site from the 22nd of February 2011, Christchurch earthquake, the ground motions recorded at the site since March 2012 can be compared with those concurrently observed at nearby Geonet strong motion stations in order to understand any peculiarities in the ground motion at the CTV site.
Ideally such instrumentation would have been deployed shortly after the 22nd of February 2011 earthquake and thus could have recorded the strong ground motions from the 13th of June 2011 and 23rd of December 2011 earthquakes which were two notably large earthquakes since that time. In examining the earthquake events which have occurred since March 2012 I have only considered those events with magnitude greater than 4. I can elaborate on that if needed but the reasons are given in the brief. It is noted at the front that these ground motions from small magnitude recordings are small relative to ground motions experienced in the 4th of September 2010 Darfield and 22nd of February 2011 Christchurch earthquakes. With for example peak ground accelerations between 2.5 and 5 percent of gravity compared with 10 to 50 percent of gravity in the main two earthquake events.

Q. Can we go to slide 6.

A. This figure illustrates the response spectra of the 12th of April 2012 magnitude 4.62 earthquake, the 20th of May 2012 magnitude 4.8 earthquake and the 25th of May 2012 magnitude 5.2 earthquakes recorded at the four Geonet strong motion instrument sites as well as the CTV site. By examining these three figures it can be seen that the response spectrum at the CTV site and the nearby instruments are at a qualitative level very similar. It should be noted that the ground motion at the Christchurch Police Station which is depicted in the first two or the upper two of those three figures in the slide is notably lower as a result of the fact that this instrument is in the basement of that structure and therefore is affected by the kinematic interaction of the structure with the underlying soil. Ground motion records from the basement of this Christchurch Police Station as well as from the basement of the Westpac building are therefore not appropriate for use in conventional non-linear response history analysis.

As with the examples of the ground motions recorded in the 22nd of February 2011 earthquake at the four Geonet stations, these three figures on this slide illustrate the significant event to event variability in the ground motions recorded. Thus these results shown on the slides serve merely to illustrate that the site response at the CTV site cannot be rejected as being different than the site response at the other four stations. This is not the same as saying that the site responses are the same.

It was previously mentioned that the ground motion amplitudes in these three events up to 5 percent of gravity in terms of peak horizontal ground acceleration were small relative to the 4th of September 2010 and 22nd of February 2011 earthquakes. As a result the analysis of these events does not clearly examine the effects of significant non-linear soil response which would have been more pronounced during these larger ground motions.

An improved consideration of near surface non-linear soil response would require additional in situ testing of the geotechnical site conditions as well as more representative soil modelling in the non-linear time history analysis. In that regard it is noted that the complexity of soil modelling using simple linear springs is highly simplified in comparison for the detailed modelling of the structure in the Compusoft report which was part of the DBH report.

I would note finally with relation to this slide and the remainder of that first brief of mine that since providing my evidence in written form Tonkin and Taylor have provided information that they accept the general basis for the inclusion of the REHS ground motion in non-linear time history analysis and I can confirm that these four ground motions have been included in the non-linear time history analysis

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Q. Now we go to slide 7 Doctor? We're now discussing the 4 September earthquake the ground motion in the Christchurch CBD in what had been called design ground motion.

A. With relation to this slide and the topic in general it should be first noted that ground motion severity on a structure is a function of (1) amplitude (2) frequency content and (3) the duration. Page 46 of my original evidence given in WIT.BRADLEY.0003.1.8 which is shown here in the slide illustrates the ground motion response spectra of the four ground motions which were in close proximity to the CTV site and which my evidence conclusively demonstrates are representative for the ground motion at the CTV site. I would add there that this statement is made in the absence of further geographical and seismological data.

Firstly, it should be noted that such elastic response spectra represent ground motion severity primarily in terms of its amplitude and frequency content, and only partially in terms of duration. It can be seen when examining this ground motion response spectra shown on the current slide that at a vibration period of 1 second which is of relevance for the CTV building the design spectra acceleration shown in the black line has a value of 0.375g. The observed ground motion spectral accelerations of the four Geonet strong motion instruments are 0.27, 0.35, 0.35, and 0.40, which in percentage terms correspond to 72%, 92%, 92% and 107% of the design spectrum at that value of one second.

JUSTICE COOPER:

Q. Have you given those figures in the order in which the sites appear in the legend?
A. That's correct.

EXAMINATION CONTINUES: MR RENNIE

A. That is, the range of the four records is 72% to 107% of the design ground motion spectral amplitude with an average of 91%. Hence, the amplitude and frequency content related aspects of ground motion severity are approximately equal to the design response spectra.

JUSTICE COOPER:

Q. The difference from your point of view is between 91 and 100 is not material for that purpose?
A. I think as a general question there is obviously a 9% difference but I think in this case that has taken the average of four sites but that's not to say that the CTV site which the ground motion is unknown is the average of those four so as noted above the range of those four sites from 72% to 107%. More formal probabalistic calculations would show that there's a 40% chance that the ground motion at the CTV site was above 100% and a 60% chance that it's below 100.
Q. So it's not possible to be more precise?
A. Unfortunately not Sir.

EXAMINATION CONTINUES: MR RENNIE

A. Ground motion duration is at least partly considered in response spectra because a response spectrum illustrates the peak displacement of a single degree of freedom structure; and large displacements will not occur if the ground motion duration is so short that a state of resonance, which often leads to such large displacements, cannot be achieved.

However, a response spectrum only illustrates the single peak displacement, and therefore a separate, and explicit, measure of duration is insightful. There is no question that a larger ground motion duration (for the same ground motion amplitude and frequency content) is more severe on a structure.

Ground motion duration is principally a function of earthquake magnitude. Since earthquake magnitude is indicative of the time it takes for the earthquake rupture to actually occur. And at that point I should note the difference between the time of an earthquake rupture and the time that which the ground shakes.

Q. Go to slide 8.

A. In order to understand the appropriate duration of a so-called 'design ground motion' for Christchurch it is necessary to examine the contribution of various earthquake sources to the seismic hazard in Christchurch. This is conventionally referred to in scientific terms as a seismic hazard deaggregation. The results for a spectral acceleration of 1 second for site class D in central Christchurch are shown on this slide and the average magnitude based on this slide works out to be a value of Mw7.4. Hence the duration of ground motion from this mean magnitude of Mw7.4 is not significantly different than the ground motion duration for an average magnitude 7.1 ground motion.

Therefore it should be noted that the assumption that a typical ground motion will have an extremely long duration is not consistent with earthquake sources which dominate the seismic hazard for Christchurch upon which this design response spectrum is based.

Q. Go to slide 9 please.

A. This slide provides data which were recorded at strong motion instrument sites following the 4 September 2010 Darfield earthquake. On the X axis shows the distance from the earthquake source to the instrument location and on the Y axis a measure of the ground motion duration. Significant duration means the duration over which the ground motion amplitude is significant.

JUSTICE COOPER:

Q. Sorry I just didn't catch that last bit.

A. Over which the ground motion duration is significant. The definition of what is large and small, this is quantified in this figure to provide certain boundaries. So it's not necessary the duration that a person would feel but that which is significant.
What this figure shows and to say it in words the magnitude M_w7.1 ground motions from September 4 2010 Darfield earthquake were actually larger on average than the mean durations expected as a result of several physically understood phenomena. Those phenomena in summary the complex bi-lateral rupture and also basin-generated surface waves which produce longer ground motion durations on average than you would expect. Hence the ground motion duration in the CBD from the Darfield earthquake given that it's above average for a magnitude 7.1 is very similar to what would be expected from an average magnitude M_w7.4 earthquake which as I have explained represents the mean magnitude of earthquake sources dominating the seismic hazard in Christchurch.

JUSTICE COOPER:

Q. This reference at the bottom of the slide to Bradley 2012, is that your principle statement you're summarising here or is that the report you did for us?

A. No that's –

Q. Or neither?

A. Neither of the above. That reference you cite there is basically a scientific examination of the ground motion from the Darfield earthquake in general. The statement is above is relating that to this particular situation we're talking about.

EXAMINATION CONTINUES: MR RENNIE

Point 8 – Following the above hence, in my opinion the ground motions in the CBD during the 4th September 2010 earthquake were essentially equivalent to a design ground motion for structures with a vibration period of 1 second at the CTV site." And again that comment is based on the two aspects of the ground motion not being significantly different than the design level given the uncertainty and response spectral amplitude and that the duration from the event was larger than expected and that magnitude 7.1 is not notably different from magnitude 7.4.

Point 9 – A lack of observable damage in the 4th of September 2010 earthquake is, in my opinion, not sufficient evidence to state that the ground motions from the 4th of September 2010 earthquake were not equivalent to the design ground motion. This is because, for example, simplified design methods are conventionally employed and contain several locations of conservatism. For example fifth percentile characteristic strengths are used which results in a factor of 1.25 and 1.4 under prediction of the mean yield strength for grade 275 and 380 steel. (Reference there Adri

no and Park 1986). Another example is the neglect of additional damping which results from non-linear response of non-structural elements which may either be of a structural nature or purely architectural or functionality nature.

Point 10 – Furthermore, the lack of observable damage in post earthquake inspections does not imply that damage did not actually occur. For example, Professor Priestley, WIT.PRIESTLEY.0001(1) notes on paragraph 80 of his evidence that crack widths of only 2mm would be required to fracture the mesh in order to commence the disconnection of the floor diaphragms to the North Core and this may not have been easily identified. Analyses for Compusoft Engineering Ltd, both in the initial report and revision as part of the NLTHA panel indicate that such disconnection is likely to have occurred (specifically they found disconnection in the case in which the input ground motion was from the CCCC station, but no disconnection in the case of using CBGS ground motion).

Q. Thank you and if you now go to slide 10 and this relates to your comparison between the Christchurch earthquake and potential Alpine Fault earthquake scenarios.

A. I think in general but particularly following testimony in the recent weeks, it's become clear that structural engineers consider the Alpine Fault as the main earthquake they needed to design for in Christchurch and I have some brief comments on that which I'd like to provide.

WITNESS CONTINUES READING FROM SUPPLEMENTARY STATEMENT AT PARAGRAPH 1

A. "The statements below are principally based on a technical publication in the 2012 New Zealand Earthquake Engineering Conference." It may be worth noting that that publication was a conference publication and therefore isn't subject to scrutinous peer review process. However the results presented aren't exactly revolutionary so I hope that wouldn't be a problem.

"Again I repeat the sentiment that ground motion severity on a structure is a function of its amplitude, frequency content and duration. The seismic hazard in Christchurch is comprised of larger faults at regional to large distances (and that was on one of my previous Figures). For example, the Alpine Fault has a postulated rupture magnitude of approximately magnitude 8.1 and the inferred source to site distance of approximately 130kms from Christchurch. Other notable faults include the Porters Pass fault and the Hope fault, among others.

Due a lack of historically observed large magnitude events, the ground motions which eventuate from such earthquakes have relatively poorly understood characteristics compared with the knowledge for small to moderate magnitude earthquakes.

However, their general characteristics are well known and can be well illustrated by comparing the ground motions in the Canterbury earthquakes with those observed from the M_w9.0 Tohoku earthquake in Tokyo which at a source-to-site distance of 110km between the Tohoku earthquake source and Tokyo is similar to the 130km distance between Christchurch and the inferred location of a potential Alpine Fault rupture.

The Figure on the current slide illustrates that ground motions recorded in the Christchurch CBD in the 22nd of February 2011 earthquake, 4th of September 2010 earthquakes and those recorded in Tokyo Bay in the 11th of March 2011 Tohoku earthquake. From the slide, and I should note on the slide that both the X axis which is the time and Y axis which is the ground motions acceleration are shown in the same scale for all three ground motions that are shown there. It is evident that the three ground motions vary widely in their amplitude and duration. The CBGS ground motion from the 22nd of February 2011 earthquake which is shown in red has a very large amplitude (nearly 0.6g) and short duration (approximately 10s of intense shaking). This is the result of M_w6.3 rupture at a short distance (approximately 4kms). The CBGS ground motion from the 4th of September 2010 earthquake has a longer duration (approximately 30s of intense shaking) but reduced acceleration amplitude and this is the result of the M_w7.1 rupture at a short to moderate distance (approximately 14kms).

Finally, the Urayasu ground motion recorded in Tokyo Bay, shown in blue, during the 11th of March 2011 Tohoku earthquake exhibits an acceleration amplitude similar to the 4th of September 2010 CBGS ground motion (in green) but a significantly larger duration (approximately 150s of intense shaking). Clearly these three different ground motions will affect structures and soils in different ways, depending on the vibration characteristics of the structure/soil and the potential for strength and stiffness degradation due to cumulative effects.

Q. Go to slide 11 please.

A. The Figure on the current slide represents the ground motion response spectra. So on the X axis it's the period of vibration and on the Y axis it's the pseudo-spectra acceleration. The four red lines are those ground motions recorded in the CBD of Christchurch at the previously mentioned four Geonet stations. The blue lines are the ground motions recorded at three locations in Tokyo Bay. Those three locations have very similar soil characteristics to those in Christchurch and the dashed black line is the ground motion response spectra to the current New Zealand loading standard. I apologise that this is not the loading standard of 1984. However, I chose to use the same figure as a published document rather than create a different one.

This figure provides a comparison of the geometric mean response spectra observed in the Christchurch CBD during the 22nd of February 2011 and 4th of September 2010 earthquakes with observed ground motions in Tokyo during the 11th of March Tohoku earthquake. In this Figure the ground motions, maybe I can skip part of this statement up until the start of the next page, since I've already repeated that. The sentence beginning, "It can be seen..." still paragraph 7, line 6.

It can be seen from the figure that the response spectra for periods of lower than 4 seconds that the response spectra are larger from 22nd of February 2011 earthquake than those in Tokyo from the M_w9.0 Tohoku earthquake. Again this resemblance that the ground motions expected in Tokyo are similar to those in Christchurch. Also, while the response spectra of ground motions in the Christchurch CBD from the 4th of September, I should flick to slide 12 now.

This slide is the same as the previous figure expect now that the red lines which represent the ground motions from the 22nd of February earthquake have been removed and replaced with the green lines representing the 4th of September 2010 Darfield earthquake. While the response spectra of ground motions in Christchurch CBD from the 4th of September 2010 earthquake and in Tokyo from the 11th of March 2011 earthquake are similar, the effects of near-source forward directivity can be clearly seen in several of the response spectra from the 4th of September 2010 earthquake at 2 to 3 seconds.

JUSTICE COOPER:

Q. You've changed the scale slightly haven't you with the acceleration axis?

A. Yes I have Sir on the Y axis I apologise has changed.

WITNESS CONTINUES:

The comment on near-source forward directivity evident in the 4th of September 2010 earthquake at two to three seconds on the slide and the figure this can be seen as these large bulges which range from approximately two/two and a half seconds to about three/three and a half seconds. Such directivity effects are not present in the Tokyo ground motions due to the large source to site distance, that is 110 kilometres and would also not be present in Christchurch from an inferred alpine fault event.

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WITNESS REFERRED TO SLIDE 13

A. As previously mentioned elastic response spectral accelerations do not explicitly account for the duration of ground motion, which is important if the amplitude of the ground motion is sufficient to cause non-linear response in structures and soils.

The previous slide, sorry the current slide explicitly illustrates the significant duration of the ground motions examined in these three different events. So in this slide on the X axis is the three different events considered and the Y axis is the ground motion duration so in the Christchurch CBD 22nd of February earthquake, as I've previously mentioned, caused ground motions of approximately 10 seconds significant duration. From the 4th of September 2010 Darfield earthquake ground motions of approximately 25 seconds duration and from the Tokyo, the Tohoku earthquake, the ground motion durations ranging from 100 to 150 seconds.

So that is the ground motions from this large magnitude earthquake were 13 times the ground motion duration from the magnitude 6.3 ground motions in Christchurch and six times that of the ground motions recorded from the magnitude 7.1 earthquake in Christchurch.

WITNESS REFERRED TO SLIDE 14

A. As I've previously mentioned ground motion severity depends on its amplitude frequency content and duration and response spectra alone only provide explicit evidence of amplitude and frequency content.

However, one method to account for strong ground motion amplitude and duration explicitly is to consider the ground motions Arias Intensity which is a measure of both ground motion amplitude and duration. I should note that it is one method which is common for the consideration of the triggering of soil liquefaction but also is relevant to ground motion severity for short period structures.

A. The figures shown on the current slide provide a comparison between the Arias Intensities of the ground motions from the three different events that have previously been discussed. It can be seen that the Arias Intensities of the ground motions in the Christchurch CBD from the 22nd of February 2011 earthquake, which on average have a value of about 2.5 metres per second, are approximately twice that from the 4th of September 2010 earthquake, on average a value of about 1.25. It can be seen that the Arias Intensities of the ground motions recorded in Tokyo during the 2011 Tohoku earthquake are larger than the ground motions in the Christchurch CBD from the 4th of September 2010 earthquake but smaller than those of the 22nd of February 2011 earthquake.

Based on the Arias Intensities, therefore, it can be concluded that the ground motion severity, in terms of liquefaction potential, for example, as well as structures which may be susceptible to significant strength or stiffness degradation for the Tokyo ground motions are between those of the Christchurch CBD for the 4th of September 2010 and 22nd of February 2011 events.

Recalling that the source to site distance of approximately 110 kilometres from Tokyo to the source of the Tohoku earthquake is similar to that of Christchurch from a perceived Alpine Fault event 130 kilometres then the severity of the ground motions in Christchurch from an alpine fault event would be expected to be slightly less than those from the Tohoku earthquake in Tokyo, therefore, making them speculatively more similar to those from the 4th of September 2010 earthquake than the 22nd of February 2011 earthquake. Strictly speaking the severity will be a function of the vibration period of the considered structure with long period structures being subjected to greater demands than short period structures.

Q. Thank you Doctor and if you'd now turn to your discussion of the vertical ground motion effects on the 22 February and 4 September Canterbury earthquakes and we go to slide 15.

WITNESS REFERRED TO SLIDE 15

A. This slide shows on the left-hand side the ground motion response spectra recorded in the 22nd of February 2011 earthquake at three locations. The response spectra are the same as ones you've previously seen, however, the scale of the axis is logarithmic in both the X and Y axis. The reason for doing so is to be able to illustrate the different nature of the response spectra for horizontal components which is shown on the left-hand figure with solid lines and vertical components which are shown on the left-hand figure with dashed lines. On the right-hand figure represents a basic ratio from the results of the left-hand figure where for each vibration period the ratio between the response spectral amplitude of the vertical component is divided by the response spectral amplitude of the horizontal component. On the right-hand figure, in particular, the results are for three strong motion stations. The station in red is the Pages Road pumping station, PRPC, located to the east of the CBD. The line shown in blue is the CHHC ground motion recorded at the instrument near Christchurch Hospital and the line in green is the ground motion recorded at RHSC which is Riccarton High School located to the west of the CBD. As well as those three lines representing specific locations in Christchurch I've shown a dashed black line at a value of 0.7 which represents the design standard in terms of permitted ratio between vertical and horizontal response spectra. So one thing to note, looking at the curve on the right-hand side, would be that the design ratio is constant and independent of the period of vibration whereas you can see that evidently the observations suggest otherwise with particularly large ratios at short vibration periods.

So with that I would state that in both the 22nd February 2011 and 4th of September 2010 earthquakes the ground motions in the CBD had vertical response spectral amplitudes which exceeded the vertical design spectra based on the Code rule of two-thirds of the horizontal ground motion which, for the New Zealand loading standards, is actually a ratio of 0.7.

I note that other structural engineering experts have commented during their testimony, for example, Professor Priestley, among others, that the peak ground accelerations in the vertical component in the CBD during the 4th of September 2010 earthquake were not significant. That is presumably to say that they were below two-thirds of the horizontal motion. This statement is true for peak ground acceleration which, on the current slide, corresponds to a vibration period of zero seconds. However, the statement is not true for vibration periods in the range of 0.05 to 0.25 seconds which often corresponds with the potential important vertical vibration modes of structures.

WITNESS REFERRED TO SLIDE 16

JUSTICE COOPER:

Q. Is that a statement that applies to structures generally or particular kinds of structures?

A. Sir, of course, every structure is slightly different but I would say it to be generally true that the vertical vibration modes for conventionally designed structures are significantly shorter, in terms of vibration period, than the lateral vibration ones.

Q. Yes, so it's a statement that's generally applicable.

A. I would, I would suggest so yes.

COMMISSIONER FENWICK:

Q. And this would overcome would it normal design criteria where you take load factored actions, the dead load plus live load, and to use the strength reduction factor, compared with when you're looking at vertical excitation you take just your dead load plus a small fraction of your live load so there's less action there. I mean to get up to that you'd have to have a pretty high vertical excitation wouldn't you?

A. I'm not sure quite what you mean sorry. Are you meaning from a capacity viewpoint or a demand viewpoint?

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Q. I'm pointing from the point of view of the capacity to upset – I mean we don't normally design vertical excitation, except where you've got cantilevers, that's the only case. I mean there are two cases in the standard where you're required to consider it, one is if you do an inelastic time history analysis, I don't know why it's in there but it's required there. It's not required for any other conditions except for where you've got cantilevers and the reason for this is there are limits, you know, it's assumed that the gravity load conditions will dominate and to overcome them you need to have very high accelerations.

A. I would accept –

Q. When you come to columns there's another factor which is slipped in, to make sure that you're well below the capacity of the column.

A. Yeah, I would accept that under – ground motions which are not directly near the earthquake source and which ground motions are not large that vertical ground motions are not important for structural response.

JUSTICE COOPER:

Q. What do you mean by near –

A. Yes, maybe the – look it maybe point 3 which I'm about to say will elaborate on that.

Q. All right.

A. Either – I'm very happy to answer directly afterwards if it doesn't ...

Q. Okay, but in the meantime I interrupted Commissioner Fenwick who had another question.

COMMISSIONER FENWICK:

No, it's all right, carry on.

EXAMINATION CONTINUES: MR RENNIE

A. Thank you. It is well acknowledged based on observations of multiple earthquakes worldwide since the 1994 Northridge earthquake that the rule that the vertical acceleration spectrum is two-thirds of the horizontal spectrum is highly un-conservative in the near field region for short vibration periods. So by the term near field region I mean when the distance from the earthquake source to the site of interest is relatively small. For example in the Christchurch earthquake the distance from the earthquake source to the CBD was approximately 4 kilometres so that is most definitely a near source. The distance from the Darfield earthquake source to the CBD was 14 kilometres and that's on the border of near source I would say.
This un-conservatism was evident in both the 22nd of February 2011 Christchurch and also 4th September 2010 Darfield earthquakes. As the two current figures on the slide show the response spectra in the vertical direction on the left-hand side for the 22nd of February 2011 earthquake and on the right-hand side for the 4th of September 2010 earthquake that those vertical response spectra go above the dashed black line which represents two-thirds of the NZS 4203 1984 design spectrum.
The point to note there would be that that point at which the ground motions exceed that response spectrum is at short vibration periods, however when looking at the response spectrum you have to make the link between the vibration period of the particular mode of the structure that you're interested in and the ground motion intensity at that mode. So while the structure has a lateral mode of vibration of approximately one second, that looking at the vertical response spectra at a period of one second in my opinion is relatively meaningless and only looking at the vertical response spectra at periods of vibration similar to that at which the vibration to floors occurs in the vertical direction is appropriate, which for reference based on the analyses of Compusoft are around .2 to .25 seconds, the vertical vibration period.

Q. Slide 17.

A. This slide illustrates essentially the same results as the previous figure except rather than presenting the result for one location across a range of periods, what I've presented is the values for different sites for a given vibration period. So for example the top left-hand figure represents the V over H or the vertical over horizontal ratio for peak ground acceleration. So that's for a vibration period of zero seconds. On the X axis of that figure is the source to site distance, so what we're looking at here is how does that ratio change as you move from right beside the earthquake source at a distance of zero kilometres out towards a distance of for example 40 kilometres.

What this figure illustrates across the four different plots. If we look at the top two figures in particular which correspond to peak ground acceleration as I mention for a period of vibration of zero seconds, and spectra acceleration for a vibration period of .1 seconds, that actually that ratio exceeded 0.7 in the majority of cases at source to site distances less than approximately 20 kilometres. That is to say that for those two vibration periods within 20 kilometres you can expect that the vertical to horizontal ratio will exceed the value currently given by design codes.

Looking at the bottom two figure panels, the one on the left at the bottom is for a vibration period of .2 seconds, the one on the right for a vibration period of .3 seconds, and what these two results illustrate compared with the two on the top panel is the sensitivity to vibration period. They're unlike the top panel where the values are very large. In the bottom panel the values are a lot smaller. On the bottom left-hand side at a vibration period of .2 seconds you can see that within 20 kilometres the results still largely exceed a value of 0.7, however at a vibration period of 0.3 seconds the bottom right-hand panel, you can see that almost all of the events, even at very short distances are below that ratio of 0.7.

So the unconservatism of the design code is a function of both the distance, particularly within 20 kilometres, and also the vibration period particularly those below a value of .3 seconds.

Moving from the actual vertical ground motion demand to its significance. The significance of vertical ground motions on structural response is well recognised as can be ascertained from the following quote from Elgamal and He 2004. Papazoglou and Elnashai 1996 drew attention to the significance of studying vertical ground motion and its damaging effects on structures. At this point I'll turn to slide 18.

Q. Slide 18.

A. Indeed field evidence from recent earthquakes and by that recent is in the context of the publication 1996, has shown that many buildings and bridges experienced significant damage attributable to higher vertical earthquake ground motions. Papazoglou and Elnashai collated such data, such damaged building and bridge case histories during the 1986 Kalamata earthquake, 1994 Northridge earthquake and the 1995 Kobe earthquake. Figure 1 from their quote which is shown here on slide 18, is among the many examples of damage due to vertical motion presented by Papazoglou and Elnashai 1996. It shows the collapse of the California State University Northridge three storey parking structure. Inward bending of the lateral force resisting system occurred as a result of interior column collapse. Very likely due to vertical ground motion, with reference to Papazoglou and Elnashai 1996. In this regard vertical motions may increase axial column forces causing an increase in moment demand, shear demand, plastic deformation and extent of plasticised zones in the beams/columns, and they cite several references for their statement.

Q. That completes your presentation.

A. There's one sentence left. Vertical motion may also reduce the ductility level in columns and moment shear capacity in beams.

HEARING ADJOURNS: 5.01 PM

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