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.

1425

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

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.

1435

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. 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.

1455

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 22nd 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 22nd 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

1620

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.

1640

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?

1650

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

INDEX