HEARING RESUMES ON WEDNESDAY 25 JULY 2012 AT 9.32 AM

JUSTICE COOPER ADDRESSES COUNSEL RE TIME-TABLING

BRENDON ARCHIE BRADLEY (SWORN)

CROSS-EXAMINATION: MR ZARIFEH

Q. Dr Bradley, I've just got a few questions for you, more clarification questions about your evidence yesterday. Firstly, the four sites, ground motion sites and in particular the use of the REHS site. Am I correct in my understanding that because we don't know what the exact ground motion was at the CTV site on 22 February in particular or on 4 September or other dates prior to 22 February, looking at the ground motions from those ground motion sites around the CTV is an attempt to get some kind of idea of what the ground motions might have been at the CTV site?

A. Yeah that's correct. I think the main aim there is that as I mentioned there is significant uncertainty so therefore considering just one site probably wouldn't give you a representative range of values.

Q. Right, and I think one of the reasons you brought in the REHS site, looking at your report, was because the other three sites that were close by or in close proximity had low spectral amplitudes in the one second vibration period?

A. Yeah so you can use a relatively independent methodology to work out the range of values you would expect of a CTV site and for that range of independently calculated values the three other records gave at or below average and therefore in order to have balance it seems appropriate to include the four sites.

Q. The recording devices that were put on the CTV site in was it May –

A. March 2012.

Q. – March 2012, they were also another attempt to check on the accuracy of those four sites?

A. Yes that's correct.

Q. I don't know if you were in the hearing when Professor Priestley gave evidence. You recall he spoke I guess you'd call it a caution about extrapolating from those recordings?

A. Yes absolutely. I completely agree. It's a commonly understood problem that extrapolating from small ground motions to large ground motions and that's why in my evidence I used the term, this means that it can't be rejected as different rather than saying as the same.

0942

Q. Right.

A. The only way to have a better understanding would actually be to conduct further site investigations at the CTV site to understand the properties of the shallow soil layers and then to actually model those explicitly.

Q. And I think you may have made the point yesterday that if you'd got the they call accelograms?

A. Yes.

Q. If you got them on before the say June and December that might have given more accurate recordings?

A. That's correct yes because the events were larger and you would have more non-linear soil behaviour in those earthquake ground motions.

Q. Right. And I notice that the three that you presented on the graphs were from faults off the coast of New Brighton?

A. That's correct.

Q. So does a different fault line have a potential effect as well?

A. Well every fault line would. Maybe, are you asking the question from different orientations?

Q. Yes.

A. So inevitably the wave propagation can be three dimensional and in the case in which its three dimensional it does depend on the orientation of which it travels towards the site, for example from the east, from the north, from the west or the south. That three dimensional effect is largely only important at very long periods of vibration so for example around two and a half seconds or longer. For relatively short periods of vibration the deep soil properties matter to a lesser extent and the shallow soil properties matter much more so the behaviour of the site say to approximately about 30 metres depth or 50 metres depth is far more important for the response of a structure of one second period than the deep property and they would be largely independent of the direction which the wave comes from.

Q. Right. And just on that shallow soil properties I think in your report you talked about the REHS or you agree I think with Mr Sinclair that the REHS site has, I don't know how it was put, but different soil characteristics than the other three sites?

A. Yes I think it's relatively well understood now that the REHS site and soils in that vicinity of the site have more organic matter than at the other three sites. Again I would come back to the comment that in lieu of not having a detailed ground condition properties at the CTV site it's prudent to still include it. If we had more detailed site investigation at the CTV site you could provide more robust arguments to exclude.

Q. All right. But just summing all of that up trying to achieve a representation of what the ground motion could have been with those qualifications though as to accuracy or otherwise?

A. Absolutely.

Q. The other thing I wanted to ask you about, you said in your supplementary material when you were talking about comparing the 4 September earthquake to design ground motions that you talked about the lack of observable damage from the 4 September earthquake as perhaps being or an explanation for that obviously one that there wasn't damage but if there was, if there wasn't damage that could have been because you said several locations of conservatism. Do you recall that?

A. Yes, yes. I think it's important to first recognise that the definition of a design earthquake is purely from a demand point of view. That actually the responsive structure and potential damage of structures is a function of the structure's capacity but the definition of a design ground motion is actually strictly related to the earthquake demand, but yes I agree with the comment that in design of engineered structures we employ conservatism and that means that on, in an average sense the strength of structures is larger than what's used in design calculations.

Q. And there can be a number of factors that make up that level of conservatism?

A. Correct.

Q. Is that the same thing as someone, I'm not sure who, can't recall now but someone's mentioned this term 'resilience' in a building?

A. I think in a general sense resilience probably refers more to how the structure responds after your primary manner in which you resist the seismic forces is degraded, so resilience may, you may also use a term like redundancy. How likely is the structure to survive if the main mechanism behind which it was staying upwards was compromised.

Q. Right. Whereas the conservatism you're talking about is more akin to something like overstrength?

A. Yes.

Q. Would you agree with me then that looking at buildings say post-1976 buildings following the 4 September earthquake and how they generally how they feared there must have been a reasonable degree of conservatism in that building stock?

A. I think every building is different and it's hard to say as a blanket comment, but on the basis of the manner in which they were designed I would suggest that's the case yes.

Q. You talked about the alpine fault and I just wanted to check something with you, check whether something's right and I could be wrong. In, if you've got your supplementary statement in paragraph 6.

A. Yes.

Q. About the middle of the paragraph. I'll just read out the sentence. "The CBGS ground motion from the 4 September 2010 earthquake has a longer duration of approximately 30 seconds of intense shaking but reduced acceleration amplitude as a result of the causal M_w 7.1 rupture at a short to moderate distance and then R-RUP"?

A. Yes so that R-RUP represents the distance from the Darfield earthquake fault which was of course located around Greendale and Darfield to the Christchurch CBD.

Q. Right it's the figure in brackets that I'm wanting to check: 14 K. Should that be 40?

A. No when this is a common I would say not misconception but misunderstanding that you often hear values reported as epi-central distance which is the epicentre is the projection on the surface of where the earthquake started, a single point whereas the actual earthquake rupture plane occurs over some large area. Now the ground shaking intensity is much more strongly correlated to actually the distance to the nearest part of the fault rather than to the part where the earthquake actually started.

Q. Okay.

A. So for example the Darfield earthquake had a rupture length of approximately 30 kilometres and the actual point at which it started was about halfway along that length.

Q. Okay. So you talked yesterday about the distance from the rupture I guess what you're saying to the site, building site as being relevant to the intensity at which ground motion was felt?

A. Yes.

Q. And obviously the further away the less both horizontally and vertically?

A. Correct particularly the vertical.

Q. Particularly the vertical. And in general terms then is that why on the CTV site the Darfield earthquake ground motions horizontal or vertical will be of less intensity than the Christchurch earthquake?

A. That was one of several factors but that was an important one.

Q. Dealing with this issue of vertical ground motions you said that Professor Priestley had said that the September earthquake vertical ground motions were not significant. I think that what Professor Priestley said and you would have been in the hearing was that he did not agree with Professor Mander's statement to the effect that the vertical accelerations in the September earthquake were exceptionally high and you'll remember there was some, a reference to Professor Mander's evidence, his report on page 38 talks about Darfield –

A. Yes I remember that yes.

Q. – being exceptionally high or implies that. And I think you'll recall that that was put to Professor Mander and I think page 6 of his statement, his report, the submission was put to him where he talked about them being at or below that .7 line. So would you agree with Professor Priestley that they're not exceptionally high?

0952

A. Well I think, I mean it's difficult using such a subjective term exceptionally high. I would say factually that they exceeded at certain vibration periods particularly from .05 to .1 seconds the value of two-thirds of the horizontal spectrum, so if by exceptionally high you mean exceeding the design spectrum then the answer would be yes. If by exceptionally high you mean exceeding the demand of the design spectrum significantly then I would say no.

Q. Right.

A. But I think the use of such subjective terminology's not really that helpful.

Q. No and I think you talked yesterday about absolute and relative in terms of September. Obviously if you compare the September vertical accelerations to the February, the February were much higher and obviously had more impact on structures, particularly the CTV.

A. Correct.

JUSTICE COOPER:

Q. The word exceptional – it's only subjective if it's been loosely used. Perfectly precise word really, referring to something which is an exception, it depends on how you use it as to whether it's subjective.

A. Correct, I guess my comment would be made in, without the notion of the exception to what, so if you can provide a relative comparison.

Q. Which you've endeavoured to do?

A. Yes correct.

EXAMINATION CONTINUES: MR ZARIFEH

Q. Just on this issue of finally of vertical ground motions. You referred to a paper, looking for the reference so that I can get the pronunciation right, is it Papazoglou and others, and thank you for giving me a copy, I had a read of it. What I wanted to put to you is it seemed that from reading that their hypothesis or thesis if you like was that building collapses and failures could be explained by vertical acceleration even when other explanations had been put forward. Would that be fair?

A. Yes I think so, I think – I wouldn't say that there is complete consensus in the role of vertical accelerations, I would say that without question vertical accelerations are not helpful but I would agree there are certain proponents who advocate that some observations could have occurred with or without the vertical acceleration.

Q. Right, and there's one example in that report, an over bridge, a freeway bridge that was damaged in the Northridge earthquake, where the author has put forward that it could be due to vertical accelerations but there was clearly an opposing view that it was the detailing or the poor detailing of the structure, and in fact in that case I think there was hoops they're called, so the spiral reinforcing or confinement on the reinforcing was something like 300 millimetres apart which was seen to be very poor detailing and responsible but obviously the vertical accelerations contributed to that, did the damage and it collapsed.

A. Yeah, I would agree completely that vertical accelerations you can think of as providing an incremental addition on the demand imposed already by a horizontal ground motions and that if you already have a structure which is particularly vulnerable, for example as you mentioned as a lack of ductile detailing, then the additional effect of vertical accelerations is probably more important than structures which are well detailed.

Q. And I think you might be familiar with a paper by is it Ambrase and Douglas in 2000 where they propose that failures that have initially been explained on the basis of vertical accelerations, inevitably found to have a more obvious explanation. In other words often the detailing or the poor detailing?

A. I think it should be put in the context they speculate that claim and they cite one or two other papers but actually the body of the work they present in that paper is actually just looking at a seismological point of view rather than the impact on the structures.

Q. Okay. But just going back to this point that you're making, that ground, vertical ground motions obviously can contribute and they can exacerbate a poorly detailed structure?

A. Correct.

Q. And perhaps we've seen a good example of that in this case, with the CTV building?

A. Yeah, I think particularly structures which are sensitive to, or have high gravity loads and weak vertical load resisting systems are therefore particularly vulnerable to vertical ground motions.

Q. Right, and if the detailing is lacking?

A. Then as I mentioned even more so.

CROSS-EXAMINATION: MR REID, MR ALLAN – NIL

RE-EXAMINATION: MR RENNIE - NIL

QUESTIONS FROM COMMISSIONER FENWICK:

Q. Yes just one or two points - a bit curious about. If you look at the response spectra for the September earthquake and that would be BUI.MAD249.0524A.7. Yes, and I can give you the next one. Don't bring it up immediately, but that's BUI.MAD249.0524A.4. If you look at that I'm just wondering if this is some evidence of non-linearity in the system. I understand all about the directional effects and the problem there. If you look at the CCCC results and you look at the REHS results and if you look at those and then you compare them with the February ones which were on the second one, can you bring those up, if you look at the peak values you can see that those two record results have shifted, the peaks have shifted quite a lot?

A. Correct.

Q. Is that evidence of inelastic deformation in your higher soils?

A. Upon inspection I would say yes.

Q. It doesn't seem to affect the other two results to the same extent, just those two. The other point is we don't have much information you say, much deep information about the soils on the CTV site but there are some quite deep bore holes on the building across the road and I suppose they're too far away to be meaning, but they were showing sands remember going from quite high depths I think, something like 25 metres or so?

A. Yeah particularly as you get close to the surface the horizontal distances over which soil characteristics change are very short.

Q. One quick final point, just while you're there, the duration of the earthquake, there seem to be different ways in which this is done, I mean one large – (inaudible 10:01:08) some time ago is looking at the rate of energy release and you count the duration from something like 15 percent to 75 percent.

A. Yes.

Q. Which gave quite a short period. Another way I don't know how GNS do it, but they've come up I think with a duration for the September earthquake of about 15 seconds, looking more just at the strong ground motion, but I see in your chart you've come up with something about 25 seconds, can you comment on the different ways in which this duration is measured (overtalking 10:01:39) -

A. Certainly, the values –

Q. – should use?

A. Yes, the values that I've presented are what we call the significant duration and they are essentially what you mention that they represent the duration of time over which a certain amount of the energy is released so I presented two different values. One was from the time at which after 5 percent of the total energy has been released until the time at which 75 percent of the energy and the other measure is from 5 percent to 95 percent. Those two measures, the difference or the reason that those are the most common two of that class of duration measure are from 5–75% is generally regarded as representing the S-waves or the shear waves which arrive in the ground motion and from 5–95% is generally considered as representing the shear waves plus the surface waves or the basin waves. So depending on the type of structure which may or may not be sensitive to those long period basin waves you may choose the other. I am not aware of GNS publishing separate results but from what you've described it sounds like they are talking about a bracketed duration in which its the duration from the first time at which a certain level of ground motion amplitude is exceeded until the duration at which the last time of that ground motion, so you might count from the time at which the acceleration becomes greater than 0.1g, the first time until the last time that occurs. That measure of duration is useful from the point of view that it considers both the amplitude and the length of time over which that amplitude occurs. However, the choice of what value do you use for that threshold acceleration is relatively arbitrary. The benefit of significant duration which I would say is widely regarded is because it represents the time over which the ratio of energy occurs. It's essentially independent of the actual energy of the earthquake itself. So therefore by having one measure, for example, response spectra which is just amplitude and frequency and another measure which is just duration you have two relatively independent sets of information.

COMMISSIONER FENWICK:

Q. And the value you've quoted is the 5–95%?

A. I can't remember exactly which point you're quoting me from but either it's either 5–95 or 5–70.

Q. It becomes particularly important in design when you look at the time history analysis and they say the period's got to be longer than 15 seconds or three or five times the natural period of the structure. I can't remember the actual details so one really needs to be able to tie this down for use in design by one term or other.

A. Yes, one way you can do that, rather than doing a detailed model first you could run a very simple model, for example, a single degree of freedom and you can look at the duration of the shaking over which the amplitudes are large and then choose from that basis.

Q. But it needs to be spelt out doesn't it in our standards which one you do. Just one quick final one – Magnitude weighting which is I think GNS are coming up with three different figures and they've come up with one for structures which is based on one of the early ones for liquefaction. Have you any comment on this magnitude weighting?

A. I do have several comments. I am, please stop me if I elaborate too extensively, the first is that I think that the concept of magnitude weighting is primarily used for soil characteristics.

Q. Sorry, the magnitude weighting as I understand it is a duration factor. The higher the magnitude earthquake, the longer it lasts.

A. Yes, yes. However, I think that effect in terms of the importance of long duration depends principally on the actual structure you consider. If you have a structure which is particularly brittle then it can fail on the first cycle, whereas if you have a structure which is particularly ductile then it can sustain many cycles. So my personal opinion is that the effect of duration is related to the specific structure and therefore should come under structure specific aspects of design codes rather than the actual seismic demand itself.

Q. So you wouldn't support the current approach then of applying this to the design response spectrum?

A. No I wouldn't.

COMMISSIONER CARTER:

Q. Just one simple matter relating again I think to duration. Just reconciling the eye witness accounts that we heard there were people who said they felt an immediate vertical punch upwards beneath them in the earthquake and then moved on to feeling the oscillation of the building itself, as two distinct features.

A. Yes.

Q. Is that quite consistent with what you recall.

A. Yes absolutely. When the ground motion arrives at the site it's comprised of multiple waves. The first of those which always arrives are the P-waves or the primary waves and at near-source distances they are particularly strong in the vertical component. So the eye witness accounts of large upward shaking to start was the arrival of those P-waves and then the large horizontal shaking was the arrival of the S-waves which probably was given the source to site distance half a second or less in difference in duration.

Q. So after quite a short interval of time the vertical effect is receding quite rapidly?

A. I wouldn't say it's receding. I would say that for that first approximately half a second you only have P-waves which are primarily in the vertical component and then you start having the strong horizontal component but the vertical component continues except because of this large horizontal component your perception of the vertical component may become diminished.

Q. And the measurements that are made and then later on picked up and used in the analyses, the dynamic analyses, quite consistent with the quality of the vertical measurements and the quality of the horizontal measurements so that those can be combined on a sort of a good, sound rational basis?

A. Yes.

COMMISSIONER FENWICK:

Q. Just so we can make sense of this for other purposes, can you let us know what durations, the way you've calculated it. If possible it would be nice to know if we can have the three different methods, three different values so we can actually –

A. Would you like me to tabulate values? You want me to explain now or to tabulate?

Q. No, no, if you can just tell us what method you've used to calculate these different durations. You don't have to do it now. You can come back if you want to later on but if you just let us know.

A. In all of them it was either the 5–75% significant duration and I think possibly I think in all of the figures I've presented it's that.

Q. Okay, the important point for me is that they are very much longer than we've been quoted by elsewhere. It was the 5–75, not the 5–95. You seem doubtful about that.

A. I think the important thing is when I make the comparison with the observed durations with then what we would be expecting. Then because you're using a consistent measure you have parity there.

Q. Yes, okay, so it was 5–95.

A. 5–75.

COMMISSIONER CARTER:

Q. Just one last matter. The character of the Canterbury earthquakes in comparison with other records that have been obtained, international sources, quite consistent or is this a unique event in a vertical sense or...

A. In terms of vertical shaking do you mean? I think based on the empirical evidence to date I would say no it's not unique. I would say the unique aspect is the large number of records we have at very short distances. Definitely having such a great density of instrumental records at these short distances is unique but the results we're observing are consistent with what have been observed elsewhere.

WITNESS EXCUSED

1012

MR ALLAN CALLS

TIMOTHY JOHN ENTRICAN SINCLAIR (SWORN)

BY LEAVE EVIDENCE AND REPORTS TAKEN AS READ

Q. Is your full name Timothy John Entrican Sinclair and are you a Chartered Professional Engineer and Fellow of the Institute of Professional Engineers New Zealand?
A. Yes.
Q. Are you a principal and past director of the firm Tonkin & Taylor? Have you been practising with that firm for the past 28 years?
A. Yes.
Q. Do you hold degrees in Bachelor of Arts and Engineering Science from Oxford University, Master of Arts from Oxford University, Master of Science degree from the University of London?
A. Correct.
Q. Do you specialise, sir, in geotechnical engineering which includes the topics of soil mechanics, rock mechanics, ground water assessment and foundation engineering?
A. Yes I do.
Q. And in that respect have you authored over 20 papers on issues relating to geo-mechanics?
A. Yes.
Q. Have you provided specialist foundation design and geotechnical earthquake engineering services for buildings in many parts of New Zealand and in other countries, including an 18 storey five basement centre known as the Deloitte Centre in Auckland?
A. I have.
Q. Do you offer specialist advice in what's known as soil dynamics and in that regard you advised on seismic hazard assessments for critical infrastructure including a number of hospitals up and down the country?

A. I have yes.

Q. Most recently, Mr Sinclair, have you been involved in foundation assessments for the remediation of damaged buildings in Christchurch area?

A. Yes.

Q. Now just in very general terms, Mr Sinclair, are geo-physical properties of a building site important because soil conditions at a site can affect the velocity of shear waves that are generated by earthquake forces?

A. Yes that's correct.

Q. And in that regard then is it important, if one is undertaking analyses such as non-linear time history analyses of a building to help identify a cause of collapse, to understand what might be known as soil stiffness or spring stiffness values of the site in question?

A. Yes they represent the connection between the ground and the building.

Q. So these values, I suppose, in lay terms might be described as values providing some sense of resilience of soil.

A. I would use the term resilience there yes.

Q. Now in that context, Mr Sinclair, were you asked to prepare a report for Dr Hyland and Mr Ashley Smith for the purpose of assisting their own investigations into the cause of the collapse of the CTV building?

A. Yes.

Q. In that regard did you prepare a report dated 11 July 2011 that you have supplemented since with two, what you call, addendum or addenda?

A. That's correct.

Q. And in that report do you provide for Dr Hyland and Mr Smith some soil stiffness values for use in their non-linear time history analysis?

A. Yes that is a dynamic stiffness value.

Q. As opposed to static stiffness values.

A. Correct.

Q. Static stiffness values being values that might be applicable to determine the soil conditions under a dead load of a building as opposed to conditions subject to dynamic forces of an earthquake.

A. That's right.

Q. And in your report did you also comment on the utility of ground motion recordings taken from various sites across the CBD as the previous witness Dr Bradley has just given evidence of?
A. Yes.
Q. And, finally, does your report also consider factors that may have contributed to the collapse, at least geo-physical, geotechnical factors that may have contributed to the collapse?
A. Geotechnical factors yes.
Q. All right, now your report, Mr Sinclair, was premised in the first instance upon data contained in the geotechnical report prepared in 1986.
A. That's correct yes.
Q. And was that prepared for the purpose of assisting the designers of the CTV building and formulating an appropriate design for that CTV building?
A. Yes that's my understanding.
Q. Now did the authors or the investigators of that, the authors of that 1986 report put down 13 bore holes on the CTV site?
A. That's right, of various types.
Q. Some hand augered or hand drilled and others machine drilled.
A. That's correct yes.
Q. And is it apparent from the results of those bore holes that there appeared to be some uniformity in the top four metres of the soil across the entire site?
A. Yes all the bore hole logs indicate that the top four metres are very consistent across the site.
Q. And that would be with silt moist firm you note in your report generally down to a depth of one and a half metres overlying silty fine to medium sand.
A. That's right.
Q. And you note in your report too that the water level was found within that sand.
A. That's right.

Q. Now does the report of 1986 interpret site conditions to differ below that level?
A. Yes.
Q. Such that there are stiff areas and soft areas within the site?
A. Yes the original interpretation was that most of the site had a gravel layer at that four metre depth but there was one area in the north-east, the north-east quadrant I call it, that that gravel was absent and, therefore, regarded as a softer area.

JUSTICE COOPER:
Q. Could you just repeat that. There was a cough at the crucial moment and I didn’t hear what you said.
A. The original interpretation of ground conditions at the site was that most of the site had a gravel layer at that four metre depth extending to some five or six metres but that that gravel was absent in one quadrant of the site, the north-east quadrant.

EXAMINATION CONTINUES: MR ALLAN
Q. Now that’s the interpretation that the authors of the 1986 geotechnical report had of the results of the 13 bore holes that they put down.
A. That's correct yes.
Q. Now your view of that interpretation, according to your report Mr Sinclair, is that it is reasonable and, indeed, most likely but you note that still uncertainties remain.
A. Yes I accept that.
Q. And is that because the hand augered or hand drilled bore holes would have been incapable of penetrating gravel layers and thereby incapable of determining the true depth of gravel layers at the site?
A. Yes, just explaining the hand augered (put down by hand) could not really penetrate the gravel but just detect the top of it and, therefore, you wouldn't know at those locations the depth, the thickness of that gravel.

Q. Since the 1986 report has there been a more recent survey undertaken relevant to the CTV site using what are termed multi-channel analysis of surface waves technique?
A. Yes that was part of an area wide survey across the CBD area for the City Council and it so happens that one spread for that was along Colombo Street adjacent to the CTV site.
Q. And the results of the MASW techniques –

JUSTICE COOPER:

Q. Just let's clarify this. Did you say along Colombo Street?
A. Yes.
Q. Well do you live in Christchurch?
A. No.

EXAMINATION CONTINUES: MR ALLAN

Q. My notes here, just seeing as His Honour has pointed this out Mr Sinclair, are that it was along Madras Street –
A. Oh, I'm sorry, it was along Madras Street.
Q. Madras Street.
A. Yes.
Q. Now the results of the MASW survey along Madras Street what effect do they have in terms of your view of the interpretation of the stiff area/soft areas that the 1986 report reached?
A. The profiles given supported the view that ground conditions can change, very similar to how it was interpreted at CTV site. In other words there were gravel layers present in places and not in others.

1022

Q. Now the range of spring stiffness values that you derived from the 1986 report are then supported by the results of this recent survey?
A. In my view yes.
Q. Mr Sinclair, the 1986 report offers some data to the designers of the CTV building in terms of foundation design and you note in your own

report that this was summarised on a chart setting out options for foundation design?
A. That's right.
Q. And that in your view that chart was appropriate, helpful and reasonably conservative for static design of the footings?
A. Yes.
Q. And as we mentioned before static design being design to accommodate dead load as opposed to the dynamic effects of earthquakes?
A. Sustained loads, yes.
Q. You conclude your report with a section containing conclusions and recommendations, one of which refers to this issue of foundation design in the future and you make the following comment, "Notwithstanding the issue of liquefaction consideration of other earthquake effects should become part of foundation designs in the future."
A. Yes I believe so.
Q. And do you conclude therefore that this should include the provision of dynamic response parameters based on certain modern surveying or mapping techniques?
A. I recommend that yes.
Q. In relation to the issue of liquefaction Mr Sinclair, in fact perhaps if we could have your report now, WIT.SINCLAIR.0001.14, and if we could have please the last two paragraphs under 4.2. Now I understand Mr Sinclair you have a correction or a typo you wish to correct, in this section so perhaps if you could read it through and point out when you get to the correction that you wish to be made.
A. I'll read this paragraph out.
Q. Thank you.
A. "In summary a thin layer between water level at 2.5 to 3 metres depth and gravel at 3.5 to 4 metres depth may have liquefied during and following the February earthquake. At the northeast quadrant this may have extended deeper. The limited thickness of the layer and the confining effect of the larger footings would mean complete bearing –

that word should be failure – would be unlikely, but “yield” with resulting settlement and differential settlement could have occurred.”

Q. All right, now you conclude there that differential settlement could have occurred, you go on in the next paragraph essentially to indicate do you that no evidence of that was found?

A. That's right.

Q. And that's based is it on the levels and planning, rather the levels and positional survey that was undertaken at the site in April 2011?

A. Yes and I refer to that later.

Q. Turning now then Mr Sinclair to the ground motion records and we've just had evidence in relation to this from Dr Bradley but you were called upon were you to provide to Dr Hyland and Mr Smith some analysis of the ground conditions at five stations of interest, being the five strong ground motion records at the Botanical Gardens, Cathedral College, Christchurch Hospital, Rest Home Colombo Street and Pages Road pumping station?

A. That's right.

Q. Now you mention in your report Mr Sinclair that the response characteristics of a particular site are generally governed by the soil profile to rock which would be about 300 metres in depth and so thin layers near the surface are not likely to make much difference?

A. Yes that was my opinion.

Q. What do you mean by soil profile to rock?

A. Just the make-up of the soil layers from ground surface to the bedrock layer level.

Q. And is that Mr Sinclair because soft soils that might overlie a rock base tend to have an amplifying effect on seismic waves?

A. That's right.

Q. You note that the exception to the soil profile to rock is being the factor that generally governs response characteristics could be the presence of deep liquefiable soils and/or the presence of peat?

A. That's right, I do.

Q. Now with that is your, excuse the pun, sort of bedrock of analysis, what affect did that have in terms of your advice to Dr Hyland and Mr Smith in terms of the suitability of ground motion recordings taken from the five stations of interest?
A. Well looking at those two aspects, first of all the depth to rock and I had no information on that but the geology would suggest that the depth to rock as you travel away from the Port Hills would be increasing, whereas the other three would be about the same and of course the presence of the peat at the log for REHS site, I recommended there they exclude the REHS record.
Q. All right, and your conclusion in relation to the Pages Road pumping station site?
A. Well likewise that was subject to severe liquefaction and so I excluded that too.
Q. Now you noted that the depth to rock hadn't been measured but you were drawing inferences were you from records and the fact of moving further away from the Port Hills would increase the depth of rock?
A. I made that assumption, yes.
Q. Now since preparing your report have you become aware of additional information regarding the potential utility of the Rest Home Colombo Street site?
A. Yes.
Q. And does that additional information include information concerning depth to rock as well as additional seismological analysis results that Dr Bradley has given evidence of over the past day and again this morning?
A. Yes. The depth to rock has been indicated from a geo-physical survey carried out by the natural hazards platform and I produce that extract from their report which would indicate that there is possibly another 100 metres depth at REHS to the CTV site.
Q. Now given that greater depth could it be expected that there would be some difference in the recorded dynamic responses at the site compared to the CTV site?

A. Well that was my assumption, if I had to include REHS it would – I would make the comment that you would expect a slightly different response there.
Q. Yes and the - I believe that in your report you suggest that the question is well how significant is the difference?
A. Yes.
Q. And is the significance of the difference or in answer to that question assisted by the seismological analysis results, obtained by Dr Bradley?
A. Yes of course, the test results that they carried out were immensely useful, if I had that at the time I probably would have made another recommendation.
Q. Yes, well you've indeed prepared an addendum to your report that takes this additional information into account haven't you?
A. Yes.

JUSTICE COOPER ADDRESSES MR ALLAN

What's the date of that?

MR ALLAN

The date of that Sir is 14 June 2012. It's WIT.SINCLAIR.0002 and the discussion of it commences at .5.

1032

EXAMINATION CONTINUES: MR ALLAN

Q. Now in light of that Mr Sinclair is it your conclusion to you now that the response to the REHS site is not unreasonably greater than measured at the CTV site and that accordingly those records do seem to provide an upper bound which helps to balance the suite of records that could be used in NTHA analysis?
A. That was my conclusion yes.
Q. All right. Since the report have you also become aware of sets of records from two other stations in the CBD, the Westpac building site and the police station site?

A. Yes.
Q. And you prepared an addendum that addresses those sites as well?
A. That's correct.
Q. And do you conclude with respect to those sites that based on the geophysical conditions those records might be warranted for use in NTHA analysis as well?
A. I did but emphasising that that was on the basis of ground conditions at those two locations.

CROSS-EXAMINATION: MR RENNIE

Q. Now Mr Sinclair the work that you did basically led to your providing estimates of the stiffness of this upgrade reaction values for the purpose of the ERSA and NTHA studies. Do you agree with that?
A. Yes.
Q. And by definition the values that you provide which are input into those studies will in turn lead to the calculation of the shear wave velocity values in those studies?
A. The reverse is true. The shear wave velocity leads to those spring stiffness values.
Q. So that the accuracy of the figures which are put in would significantly impact the outcome of the ERSA and NTHA studies. That's plainly obvious isn't it?
A. I wouldn't be able to say it's obvious but I don't know. In fact I have asked the modellers how sensitive it is to those.
Q. And were you given an answer as to sensitivity?
A. I was, yes.
Q. And did you apply a sensitivity adjustment yourself or did you leave to them to do?
A. No when I provided the, when I provided the information I gave a range.
Q. And just before we come to that range, now am I correct that in terms of bore holes your work has not included the sinking of bore holes by any means on the CTV site?
A. That's correct.

Q. In fact if I understand your report correctly it's dependent on the 1986 data except that in relation to the REHS site you did have a bore hole result available from the DBH investigations?

A. I did have a bore hole available on REHS although I don't think that is necessarily connected to your first question. Your question of whether I was basing those spring stiffnesses on the site information that would be correct but also on common assumptions.

Q. If we go to the 1986 report which the reference is BUI.MAD249.02039. It will come up on the screen for you. Now this is the plan which shows the location of the three types of bore holes which were sunk in 1986, you recall that?

A. Yes.

Q. And then in the next following pages as you recall it there is from 11 onwards the logs of the drill holes in relation to the findings which were then made?

A. Yes.

Q. And that's been part of the reference that you've used to the work that you've done?

A. Part yes.

Q. Now in relation to the diagram on page 9 the cable tool holes which are the solid filled segment were the three deepest holes, is that correct?

A. The two cable tool holes, yes, 15 and 14.

Q. Sorry I stand corrected. And in terms of the practice of the day would you regard that as a reasonable sampling of the conditions on the site?

A. I have given that opinion that was, are reasonable in accordance with the practice of the day.

Q. And in relation to the hand-augured holes, was it generally the position that they were not sunk as far as the underlying gravels which were then inferred in the study to be found there?

A. They were taken to what was inferred to be gravel.

Q. And in relation to the areas of silt in the north eastern corner which you referred to, which are the bore holes you considered to be particularly relevant to that?

A. Well all but number 14 in particular was a deep bore hole and it showed there just 100 millimetres of gravel and then continuing into sand, with sand below that.

Q. So that in terms of establishing values for shear wave velocity you have proposed likely lower bound and upper bound values in this information, is that correct?

A. That's correct yes.

Q. And we find those values in your report. Just go to that at WIT.SINCLAIR.0001.20. 20 is your most likely values, correct?

A. Yes.

Q. And you propose for the stiff area at 300 metres a second subject to the sensitivity which we're coming to and for the soft area which I take to equate to the silt area we've just been looking at 200 metres a second?

A. That's right yes.

Q. Turning to the next page at 21 we have the lower bound and your values alter there to 280 and 150 respectively.

A. That's right.

Q. And your upper bound values which appear on page 22 rise to 320 and 250 corresponding?

A. That's right.

Q. Now those are values at which you derived at the time of your report which was in 2011?

A. That's right.

Q. The sensitivities which you are adjusting for, do they include uncertainties in relation to the values that you are calculating or simply uncertainties in relation to the soil conditions?

A. They were really uncertainties in relation to the shear wave velocity which is soil condition.

Q. Would you agree that even under highly favourable conditions such as down hole or cross hole methods it's difficult to achieve a precision closer than 10 metres per second in establishing a value?

A. I am not aware of the sensitivity of that sort of testing.

Q. In terms of the level of uncertainty which you've backed into the values you've calculated would you accept that the adjustment you've made is small?
A. Yes, especially for the stiffer range, that's a fairly narrow band.

1042

Q. Now do you have an estimate as to the approximate depth of influence for the majority of the foundations at the CTV site, that's the stiff rather than the soft area?
A. I have used an assumption on the depth of influence of each footing.
Q. Yes.
A. Yes.
Q. So would you, you had a range of values. I think your highest value was somewhere round about 10 metres.
A. This is in reference to the static not the dynamic?
Q. Yes.
A. Yes.
Q. Now in this case the ground motions at the CTV site, as you have seen from Dr Bradley's evidence, have basically had to be inferred principally from adjacent measuring stations at other locations.
A. Yes.
Q. And do you accept, as I think you did when your evidence was led, that the shallow soil characteristics at the CTV site are sufficiently similar to those other four stations that they can be used?
A. I accepted that the records indicate that that could be the case. I don't accept that the ground conditions are similar for the fall on the basis of what I said earlier and that is that there's a presence of peat at REHS.
Q. Yes but in terms of setting the bounds within one would expect CTV results to fall if that data had been available that sets those bounds.
A. Yes, that's right.
Q. In relation to the shear wave velocities that we've just been looking at did you look back to the 1986 paper or other information to see what

velocities were commonly planned to in the Christchurch area at that time?
A. No.
Q. Are you aware of a paper published in December 2011 by Wood and Others in relation to shear wave velocities at the various measuring sites?
A. I'm now aware of that yes.
Q. And they, in fact, give values in a graphical form for the four sites that we've just been talking about don't they?
A. Yes.
Q. So if we can go to BUI.MAD249.0543 and then to what will be point 5 of that page. And if you look at the centre of that, if we could perhaps bring the three graphs up. These seek to graph shear wave velocity relative to depth for the ground measuring stations in Christchurch, correct?
A. That's correct.
Q. The values to be derived from those graphs in relation to the metre depth that we were just talking about are significantly below the shear wave velocities that you calculated, do you agree?
A. They are yes, it's the left-hand, it's the left-hand diagram that really counts and that over the top 10 metres that is less than my general range.
Q. Well, in fact, of the, putting REHS to one side for one moment, looking at CBGS, CCCC and CHHC the value at the 10 metre depth appears to be of the order of 150 rather than the 300 that you've postulated doesn't it?
A. Yes over the top 10 metres yes.
Q. Would you prefer that value to the value that you've proposed without that data in 2011?
A. Ah certainly if I'd had this information at the time I might well have had lower values in my calculations.
Q. And would you be reinforced in that if there were evidence available to you as to the values used by practitioners in Christchurch at that time?

A. Yes obviously I'm not aware of any values used. It wasn't a common practice at the time to look into the dynamics of foundations then.

Q. Now the soil modelling which we have been discussing as we identified at the beginning is an input into a structural modelling process?

A. Yes.

Q. And that you would accept is a complex and sophisticated process.

A. Certainly, I'm not familiar with the structural programme.

Q. The proposition I'm putting to you is that it is somewhat inconsistent that the soil was modelled simply with linear soil springs when much greater detail was given to the structural modelling at the next stage of the process. There were no variants or hypotheses put in to test the different outcomes.

A. The approach is a simplification of soil conditions, yes. It is a normal simplification and for the first model that was all that was asked for.

Q. And that was to be my next question, that you have supplied what was asked for?

A. Yes.

Q. Now you made some reference to liquefaction and some reference to the impact in the earthquake condition of water presence in the subsoil, do you recall that?

A. I do yes.

Q. Now the evidence to date appears to be that there was no or no significant surface manifestation of liquefaction but would you accept that excessive core water pressures would still have been generated which would lead to a reduction in shear stiffness, particularly as a result of deformations associated with the lateral vibration of the structure?

A. I have commented on that and there is, in my view, no evidence of a build up of core pressure which could lead to liquefaction. I'm saying that I've already said that that is quite possible. If that were the case we would expect to see more deformation in the foundations. My understanding is that there was no differential settlement in the foundations so I didn't feel that it was a significant factor here.

Q. To be clear, Mr Sinclair, the deformation in the foundations would be identifiable at the base level of the foundation, foundation unit by foundation unit.
A. It should be.
Q. If you were looking for it what would you expect to see?
A. A differential settlement over the foundations between one foundation and another.
Q. And that's not a matter that you were asked to investigate at any time?
A. No. I was engaged after that investigation was done.
Q. Have you sighted any photographic or surveyed record of the positioning of the foundations before they –
A. I've seen the photographs yes.
Q. And is it your perception that those photographs do not show settlement of –
A. You can't tell from the photographs.
Q. So in reality whether the phenomenon that you've just referred to – differential settlement – did or did not occur is, in the strict sense, an unknown.
A. In the strict sense but that is what's reported.

JUSTICE COOPER:
Q. What is reported?
A. That there was no differential settlement measured or the –

CROSS-EXAMINATION CONTINUES: MR RENNIE
Q. And in the sense of reported means that you have seen that stated by persons other than yourself but you don't personally know the position?
A. That's correct.

1052

Q. Now you referred to the study that your firm has done for the Christchurch City Council which is a very extensive study with multiple methods used to ascertain the subsoil structures?

A. Yes.

Q. That is what you discuss in part in your second brief in relation to the ground measurement sites, the four ground measurement sites and the CTV site?

A. Yes I used one, one plot from that.

Q. Yes. So was it essentially a question of comparing that plot from that general study to what was known about the four specific measuring sites to see if the two were consistent?

A. Yes I did that.

Q. And what level of agreement do you consider you found?

A. Well when I did it I averaged shear wave velocities over the two extreme example profiles I took from the chart and found that they were compatible with my average values, my most likely values.

Q. You've not at any stage been asked to carry out a more intensive investigation of the CTV site itself?

A. No.

Q. If that were undertaken what would you expect to learn from it?

A. Similar results to what is provided here perhaps. We can't tell.

Q. You say you can't tell?

A. No.

Q. Lastly, you made some statements from your experience, which is respected, as to 1980s process and practice in relation to the investigation of foundations?

A. Yes.

Q. And my understanding is that you generally see the work in that area as being consistent with a typical practice of the time, is that correct?

A. That's right, I've seen many sites around the town now and this is similar level of practice.

Q. Speaking of the period immediately before the Darfield and Christchurch earthquakes would you say that practice had altered or developed since the mid 1980s? Standard practice?
A. In detail yes. Possibly in principle no.
Q. And by "detail" are you referring to technique or level of investigation or both?
A. I'm really referring to technique.
Q. And does that represent the availability of more sophisticated techniques and equipment than those which existed in 1986?
A. Yes in particular the type of drill rigs and the penetration test.

QUESTIONS FROM COMMISSIONER FENWICK – NIL

QUESTIONS FROM COMMISSIONER CARTER – NIL

CROSS-EXAMINATION: MR ZARIFEH – NIL

QUESTIONS FROM JUSTICE COOPER – NIL

RE-EXAMINATION: MR ALLAN

Q. Mr Sinclair you were asked questions concerning the source of the comment in your report that there was no evidence of differential settlement and is that a matter that you have addressed in the statement of evidence WIT.SINCLAIR.0001 that you have prepared in order to produce and supplement that report?
A. Correct.
Q. And you mention in that statement, Mr Sinclair, that you rely there upon the levels and positional survey carried out in April 2011 as reported in the Hyland building site examination and materials test report?
A. That's right.
Q. Section 7, conclusion 7 and you cite there do you the parts of that report the discuss the absence of settlement, cracking damage and signs of liquefaction?

A. That's correct.

WITNESS EXCUSED

JUSTICE COOPER:

Mr Rennie, these questions you asked about differential settlement of foundations, is evidence contemplated on that matter?

MR RENNIE:

Not from the party I represent Sir.

JUSTICE COOPER:

So we now reach the nonlinear time history analysis?

MR MILLS:

We do Sir and we'll have to do a bit of organising at this front row here and so on to make room, so I wonder if we could just take the adjournment now.

JUSTICE COOPER:

Yes we can adjourn so that we can set up.

HEARING ADJOURNS: 10.58 AM

HEARING RESUMES: 11.25 AM

MR ELLIOTT ADDRESSES THE COMMISSION

For the next two days or so the Commission is going to deal with non-linear time history analysis and ERSA matters. I'll just give the Commissioners a brief introduction to orientate those watching. Firstly I will just indicate that the Commissioners now have Mr Tony Stuart who is to Your Honour's immediate right; Dr Barry Davidson, next to him Mr Derek Bradley, next to him, they are all from Compusoft, and Professor Athol Carr front row there. Next to him Mr Smith and next to Mr Smith, Dr Bradley and I'll say a few words about some of those experts shortly by way of introduction.

JUSTICE COOPER:

Mr Jury and Dr Hyland are they to come in due course?

MR ELLIOTT:

Yes Your Honour they will and I'll explain that. Your Honour firstly on the issue of non-linear time history analysis. Just to put it simplistically, it's possible apparently to develop a model, a computer-based model of a building and a model of the CTV building was in fact developed by the members of Compusoft who are sitting to the Commissioners' right and that was done for the Department of Building and Housing. The model can be and was subjected to simulated earthquake forces and the purpose of that is to determine the behaviour of the building and its parts in the 4th of September and 22 February earthquakes. The results of a report produced by Compusoft were then taken into account by Dr Hyland and Mr Smith and members of the Department of Building and Housing Panel in considering possible collapse scenarios of the building. I think all experts on this issue would agree though that NLTHA is not necessarily definitive but it does produce very helpful information which, when considered with other evidence, helps to identify the building's performance and the other evidence includes things like the state of debris after collapse, witness' statements and so on, the sort of evidence which the Commission is hearing.
The process of putting this computer model together first involves identifying what we've called "inputs" and there are many many inputs apparently and they include things such as the strength of particular building members to allocate to the model. There are other inputs such as the state of the ground shaking which must be simulated for the exercise and obviously the nature of the shaking the building was subsequently exposed to.

In the course of preparing for this hearing Mr Ashley Smith who was primarily involved in liaising with Compusoft in relation to the inputs he was asked to prepare a statement setting out the inputs. A copy of that statement is at Tab 9 of the Commissioner's bundle and Mr Derek Bradley from Compusoft was also asked to prepare a similar statement. Your Honour should have two folders I hope. I wasn't intending to refer the Commissioners to any particular part of those statements, just to point out that they are there for reference.

As the Commissioners would appreciate, there is scope for debate about the inputs that should be used in a model like this and as a result of that the Commission made an order on the 18th of May 2012 directing experts to confer about those inputs and a copy of the Commission's direction is at Tab 1. I will just bring up a section of that to illustrate the main purpose of the process and that's BUI.MAD249.0532.2, paragraph 4.

"So the purposes of the experts conferring were to endeavour to reach agreement on the input data to be used to conduct an NTHA of the response of the building which would provide the most reliable model of the response to the earthquakes on 4 September and 22 February."

And then the experts were asked to identify where agreement could not be reached which inputs that related to and the reasons for the disagreement and the ultimate purpose was to produce NTHA results which would provide the most reliable model of the response for the building to those two earthquakes.

Professor Athol Carr was appointed to facilitate the process and I'll just now introduce Professor Carr as he hasn't yet appeared before the Commission and his CV will be placed on the Commission's website as is the process with other experts but just by way of brief introduction Professor Carr is a Professor Emeritus in the Department of Civil and Natural Resources Engineering at the University of Canterbury. He graduated from the University of Canterbury in 1964 with a Civil Engineering Degree with First Class Honours and then graduated in 1966 from the University of California with a Master of Science in Engineering and then in June 1967 he graduated from the University of California with a Doctorate of Philosophy. He is also a Fellow of the Institution of Professional Engineers (New Zealand), a Member of the New Zealand Structural Engineering Society, a life member of the New Zealand Society for Earthquake Engineering and a Member of the American Society of Civil Engineers. He has an Adjunct Professorship at the Earthquake Engineering Research Centre, University of Iceland and his areas of specialist professional expertise are in structural dynamics, non-linear time history analyses of structures and finite element analyses of structures and he has authored and co-authored many journal papers, conference papers and reports and a book.

So Professor Carr, as I say, was appointed to facilitate the process, experts conferred and a joint report was produced and that appears at Tab 2 of the Commissioner's bundle. What they do in that report is to identify areas of agreement on NLTHA inputs and areas of disagreement and shortly I'll invite Professor Carr to present that report.

In addition to that Commissioners, Professor Carr also produced a separate individual report which is at Tab 3 and that separate report comprises really a record of the communications between the experts throughout the process of conferring and including reference to documents and it wasn't intended that Professor Carr would read from that report or anything but it's produced to the Commission as a point of reference, setting out the detail of the process that was followed.

So shortly Professor Carr will, as I say, present the joint report on the process. He will take the Commissioners through the areas of agreement and disagreement. Ultimately, of course, the Commissioners will need to reach a conclusion on the areas of disagreement and in order to assist with that process a "hot tub" has been convened. I have been asked what a "hot tub" actually is and there's no tub – hopefully it's not too hot – but as the Commissioners are aware it involves all of the experts being present at the same time and it's really an alternative to individual cross-examination by lawyers who may be even more inept than usual when dealing with technical expert issues and so the purpose is for the experts to express their views, views to be canvassed in dialogue with the Commission and for that process to lead to a resolution of complex issues and it's a process which is used, as Mr Mills mentioned in opening, in Courts very commonly.

So Commissioners, as Professor Carr identifies each area of disagreement it will be up to the Commission to ask the experts to expand upon that as the

Commissioners see fit. The outcome of the process that the experts followed was a second Compusoft report. So in fact there are a few versions of that, but the first Compusoft report from the DBH process is at tab 7. A further Compusoft report, which I think the Commissioners have had now for a few days is the one entitled, "First Draft Issue, July 2012," and the Commissioners will also see another report on your desk which is a further report from Compusoft which was produced yesterday and they will be able to explain why there are two. I don't think there are any great differences between them but they can outline that. Following conclusion of the input discussion, Compusoft will actually present their reports to the Commission. The reason that we have three people from Compusoft here is so that they've all contributed to the process so they can answer questions about any of their reports. They've been asked in particular to identify key differences between their first report and the more recent reports, and also to comment and to answer questions about the main features of the building's performance in the earthquakes as identified by their more recent report in particular.

JUSTICE COOPER:

Mr Elliott there are statements of evidence in these files. Is the intention that we take those as read rather than have them read out?

MR ELLIOTT:

There are two categories Your Honour, the first relates to the inputs I've just mentioned and it wasn't proposed that they be, they be read out. In the case of Mr Bradley and Mr Smith they, they just account for the inputs which have now become a subject of discussion in the expert panel, and I think those documents were available to the expert panel. I think in both cases though they've also produced a shorter brief where they give views on interpretation of the first Compusoft report and they could certainly affirm the contents of that but each expert who has expressed a view has been asked to file a further document on whether their views have changed. I will go in to explain that in a moment. So that issue of interpretation is the subject of a separate hot tub. Once the further report from Compusoft was produced, counsel assisting invited experts who've given evidence on interpretation of the data to consider areas of agreement and disagreement about what that report says about the performance of the building and that has resulted in a second joint report. And so after Compusoft have presented their reports there will be a separate hot tub at which time Mr Jury and Dr Hyland will join the hot tub and Professor Carr will present his report on interpretation to follow the same process where experts will outline areas of agreement and disagreement about the interpretation. And Your Honour each expert, as I say, has been asked to produce a report to the Commission saying if their views have been altered by the more recent Compusoft report and those reports are all in the bundle as well under tab 13. I'll just now briefly introduce those experts who the Commission hasn't heard from before.

Firstly Mr Stuart, Tony Stuart, he's a senior engineer. He has been with Compusoft Engineering for over 10 years. His major role has been modelling building structures for Compusoft and its clients who are other consulting engineers and in that context he works with clients to solve problems for them when they use SAP2000 and other software products. And as I've mentioned, he assisted Mr Bradley in developing some of the models relating to the CTV building.

Dr Barry Davidson in the centre of the group, he is an engineer. He has an Engineering Science Degree with Honours. He has a PhD in Engineering Science. Over 30 years of technical engineering experience. He has been actively involved in analysis and design of most structural forms. He's a past president of the Structural Engineering Society of New Zealand. He's been involved in developing industry standards for earthquake engineering design and he's published widely in fields relating to earthquake engineering. From 1978 to 2005 he was a senior lecturer in structural dynamics and earthquake engineering at the Department of Civil Engineering at the University of Auckland.

Then Mr Derek Bradley holds a Bachelor of Engineering with Honours in the civil discipline. He's a chartered professional engineer and qualified as an international professional engineer. He has over 15 years of structural design experience both in New Zealand and abroad and his experience includes structural design analysis and management of commercial, civil and industrial projects. And he's been employed by Compusoft as a senior engineer for eight years, undertaking structural analyses for in house designs and external structural design consultants.

The Commission's already familiar with Mr Ashley Smith and with Dr Brendon Bradley so I won't say anything further to introduce them. Unless there are any questions on the process so far I'll –

JUSTICE COOPER:

They may emerge.

MR ELLIOTT:

They may emerge. I think I should let the Commissioners know that all of those present, including those who are not, Professor Shepherd and Professor Mander have really put a great deal of work into each of these processes under considerable time pressure and including working on weekends and they are putting other matters to one side. So Commissioners at this point I'll ask for them to be sworn and then I'll invite them just to confirm that they've read the code of conduct and then it will be over to Professor Carr to present his joint report.

JUSTICE COOPER:

Well rather than swear them I think we'll affirm them because it is easier to do that in bulk. So what I'm going to do gentlemen is put a proposition to you and then once I've done that I'll just read each of your names and I'm going to ask you in broad terms to promise to tell the truth, and if you would just indicate yes when I call your name or no if you want to leave at once.

TONY STUART (AFFIRMED)
BARRY DAVIDSON (AFFIRMED)
DEREK BRADLEY (AFFIRMED)
ATHOL CARR (AFFIRMED)
ASHLEY SMITH (AFFIRMED)

BRENDON BRADLEY (AFFIRMED)

MR ELLIOTT:

And Your Honour I'll invite each expert to confirm that they've read the *Code Of Conduct For Expert Witnesses* and agreed to comply with it in terms of the evidence they're giving today.

MR ELLIOTT:

It is over now to Professor Carr to present a report.

PROFESSOR CARR:

Commissioners, Your Honour, this is the joint report of the nonlinear time history analysis panel. The introduction, this is a joint report pursuant to clause 9 of the Order of the Royal Commission dated the 18th of May 2012 in relation to the nonlinear time history analysis NLTHA.

The purpose of the experts conferring were to endeavour to reach agreement on the input data to be used to conduct a non-linear time history analysis of the response of the CTV building which provides the most reliable model of the response of the building to the earthquakes at 4.35AM on the 4th of September 2010 and 12.51PM on the 22nd of February 2011. Where agreement cannot be reached on the inputs to identify (a) the inputs which cannot be agreed and the reasons for the disagreement, and then to produce a non-linear time history analysis results which provide the most reliable model of the response of the building, to the earthquakes at 4.35AM on the 4th of September 2010 and 12.51PM on the 22nd of February 2011 and which can then be analysed and interpreted. The topics, the report addresses the following topics, areas of agreement, areas of disagreement including the reasons for the disagreement, results of the further non-linear time history analysis and additional issues the panel was asked to address.

Move onto areas of agreement, the first area is the strength, or concrete strength. After some discussion it was accepted that the concrete strength be taken as 1.5 times the design strength and not the design strength plus 2.5 kPa as used in the original Compusoft analysis. Mr Smith accepted this on the understanding that you could still compare the results with those from the original analysis to check the potential effects of variable strength. Professors Mander and Shepherd and Dr Bradley considered that examination of the sensitivity of the results of the analysis to assume concrete compression strength is necessary and it noted that this will be a future hot tub discussion between concrete experts at the Royal Commission to discuss the various concrete test results and it is hoped this might provide further guidance on the likely concrete strengths.

JUSTICE COOPER:

I should comment, I'm not too sure about the derivation of the phrase hot tub, the intent of some lawyer probably who invented the term was to contrast the situation where people come along individually and give evidence on oath and be questioned and then perhaps several days later in a hearing an expert in the same subject matter with a different point of view would come along. That's the traditional path at hearings in Courts that are normally taken but the thought is there are many subjects which would actually result in better understanding being developed by the decision making body, the relevant people were all gathered together and spoke but roughly at the same time, and also that it would give the opportunity for people in a process of discussion where ideas are being transferred for perhaps even greater areas of agreement or to develop or the reasons for disagreement to be placed in sharper focus as a result of a kind of conversation really and that's the intent of it.

MR CARR:

The next item was the column plastic hinge modelling, the panel was unanimous in the requirement that the columns should use an axial force bending moment or PMM yield interaction instead of the bending moment or MM yield interaction surface used in the original analyses. The vertical ground accelerations were large and the variations of the axial and the axial forces in the columns could be very important and that therefore should be taken or included in the column hinge model. Masonry infill panels, the previous analyses, this is the first Compusoft report, incorporated two cases for masonry and for the walls. The first case with the masonry panels isolated so as not to interact with a concrete frame at any level of drift and the second case for the masonry panels in direct contact with each other and with the concrete frame with lateral stiffness and strength of the masonry based on flexural yielding at each floor level. It was agreed that these DBH analyses gave an adequate insight into the potential masonry interaction effects. Due to time constraints for future analyses only one of these cases could be considered. The one taken would correspond to the masonry infill panels being completely isolated from the structure. This could be considered to reflect the design intent.

Floor diaphragm modelling – in general the modelling used to represent the overall behaviour of the structure was satisfactory but if there were localised floor deformations then the stiffnesses used need to be decreased and the finite element mesh may need to be further refined. These effects were regarded as being of much less significance than the modelling of the column plastic hinges and the modelling of the beam column joints. The diaphragm model was adjusted to better represent the behaviour near the north tower and along the beam lines. Potential fracture of the mesh has not been modelled and will need to be considered when evaluating the results.

Ground motions used in the analyses – the use of the Resthaven, REHS ground motions as part of the analysis suite for the February earthquake was accepted. These have been incorporated in the revised analyses. There is also agreement that some analyses should take the September earthquake and then follow this with the February earthquake to see how using the damaged structure for the February earthquake would differ from starting the analyses for the February earthquake with the original undamaged structural model. It was agreed that the sequential analysis should include the Christchurch College, CCCC ground motion and it was suggested by Professor Shepherd that it could, time permitting, be done for all four ground motions. These results for two ground motions including the CCC ground motion are reported in the revised analysis report.

JUSTICE COOPER:

So what are the two then?

MR CARR:

Botanic Gardens, the CBGS, the Christchurch Botanic Gardens and the CCC which is the Cathedral College.

Duration of shaking. There was agreement that the duration of shaking included in the September and February earthquakes needed to be further investigated. The start and finish times for the revised non-linear time history analyses were adjusted and they are reported in the report.

Damping model – there was agreement that the damping model is accepted but that the level of reduction in the specified damping for the February earthquake needs to be adjusted to give a more realistic damping level for the vertical modes of free vibration. This is reported in the results of the revised non-linear time history analysis. And Mr Elliott has suggested that maybe it's a point when I might make a further comment, if you read the report, WIT.CARR.0001.1, at about page 7, I have made some comments about the damping models used by the programme that was used in the analysis model. Most of the software in the world uses the Rayleigh damping model which is SAP2000 users. We became aware of limitations of that model about 1979. We found that for non-linear structures it could have a very strange detrimental effect and there are some figures that follow, I don't want to push them at this stage, but they're there for the Commission to maybe examine at some point if they feel so inclined.

JUSTICE COOPER:

I'm looking at a document which is in our – the Commissioners have a tab, tab 3 WIT.CARR.0001 capital B.

MR CARR:

Yes, that's the one, I only saw the one on the front.

JUSTICE COOPER:

And it's starting at .7 which has a heading 2.7 Damping Models.

MR CARR:
Yes that's correct.

JUSTICE COOPER:
That's what you've just been referring to.

MR CARR:
Yes. The problem with the damping model is with the Rayleigh damping you can specify the ratio of damping you want at only two frequencies, and in between those frequencies the damping is lower than you actually want, and beyond the second frequency it's higher than you actually want and that's the part that's higher was the concern we have on non-linear time history analyses. The people were concerned about the part that's below for the vertical modes which are in the range of periods in the Compusoft model which is in between, I think, 120 Hz and about 1 Hz where the first mode is located. I let that slide at that point because there's nothing we can do about it. The programme has a damping model and we have to accept it but I have concerns that the damping has an effect. It's largely ignored. It's put in the model and then forgotten about so it's just something I wanted to raise as an issue.

WITNESS CONTINUES READING BRIEF OF EVIDENCE
A. Move on to beam pull out and exterior joints.
There's an agreement that this needs to be modelled. This is further incorporated in the beam column joint model discussed below. Compusoft stated that this could be done, however, were unsure of the behaviour of the hinge and requested guidance as to the backbone curve that would suitably present the response. No guidance has been provided despite numerous requests so bar pullout was not incorporated in the revised non-linear time analysis. It's commented on later on. Compusoft also has concerns over attempts to simplify this mechanism

through a backbone curve as the pull-out strength will be greatly influenced by the axial compression in the joint. In addition we're not sure how this pull-out will manifest itself should bar pull-out take it with a wedge of concrete and the ability of the joints to support axial load could be compromised. If so there is little to be gained by modelling this failure mechanism as pull-out would mean total collapse which can be achieved through monitoring demands rather than through explicit modelling.

JUSTICE COOPER:

Q. You read total collapse. The word is local collapse in the text.
A. I'm sorry. I apologise it is local.
Q. Local collapse.
A. Yes local collapse.
Q. Right.
A. That may mean the same thing in the long run.
Q. Depends what local means.
A. Yes.

WITNESS CONTINUES READING BRIEF OF EVIDENCE

A. The beam column joints. The panel was unanimous in its request that the beam column joint model must be modelled as a non-linear manner. There was considerable discussion that went over about two weeks on the manner in which the beam column joints could be modelled and the way the degradation of strength could be represented. Compusoft had concerns that the methodologies promoted would not adequately capture the joint behaviour. And in material under Tab 3 there's a long discussion about modelling of joints and the inadequacy of most of the current joint models that are available.

Move now to areas of disagreement.

The first item is drag bar strength. There was agreement from the panel that consideration be given to the revised drag bar tensile strengths provided by BECA engineering. The BECA strengths are considerably less than those used in the Hyland Smith report, however, as noted by BECA, and there is considerable uncertainty in the estimation of these tensile strengths and the range of strengths given by BECA could not be considered to be either an upper or a lower bound. Mr Smith advised that one reason why the BECA values were lower is because they had used lower characteristic material strengths and not expected or nominal material strengths which are necessary for this non-linear time history analysis. Mr Smith advised that the strengths computed from the original DBH report were considered to be upper bound expected strengths and yet disconnection of the drag bars was still predicted in those analyses. There was no subsequent discussion. The original strengths were used in the revised analyses. The orders were assess the impact of this assumed drag bar modelling. Professors Mander and Shepherd and Dr Bradley argue the analyses should be a formed using both sets of reported values.

Column plastic hinge modelling. The panel unanimously agreed that the column hinge definition should provide for PMM or axial force moment moment interaction. In an attempt to consider this the column hinges have been modelled considering lumped plasticity fibre-based hinges at the column ends. In these hinges the interaction surface and hysteretic form is governed by the material properties and geometry. Both shear and torsion are considered to be linear and uncoupled from the PMM behaviour. This solution is consistent with the desires expressed by members of the panel to include the PMM interaction. However, many of the panel members would prefer a line model as they consider this would better represent the behaviour of the hinge. This is not available in this SAP2000 software.

Floor diaphragm modelling. Professors Mander and Shepherd and Dr Bradley considered that there was contention as to the equivalent flexural stiffness of the floor. Professor Mander proposed to use an effective stiffness of 0.1 EIG whereas Professor Priestly had contended that the effective stiffness is significantly greater. Given the inferred importance of vertical ground effects on the 22nd of February 2011 earthquake the sensitivity of the results to the effect of stiffness used should be examined.

Duration of shaking. Doctors Bradley and McVerry suggested that the whole length of the ground motion should be used for the analyses and this should include the first arrival of the P wave. The computational times would be considerably extended. Demonstration of the effects of the significant responses for simple structures showed that emitting up to 20 seconds of the original September ground motions had minimal effect on the responses and that once the major part of the ground motion had passed then little more would be learned by persisting with the later parts of the ground motion. Compusoft Engineering Ltd list in their report the start times and durations of ground shaking used for the revised analyses. Professors Mander and Shepherd and Dr Bradley argue that the omission of the start of the February ground motions will have an effect on the displacements and velocities and hence the damping and inertia forces acting in the structure. They also consider that the September records should have been run for a longer duration to see if further damage occurred in the diaphragm to north wall connection.

Ground motions used in the analyses. Professors Mander and Shepherd and Dr Bradley now argue that the September analysis should include the Christchurch Hospital (CHHC) and Resthaven (REHS) ground motion noting that as damage was indicated in the September earthquake the analyses to get a measure of sensitivity of the results for the ground motions.

JUSTICE COOPER:

Q. Can I just ask for clarification of that. It doesn't seem to quite be a complete thought. Just the last two lines there – "Noting that as damage was indicated in the September earthquake the analyses"…

A. Probably should be, it should include 'these other earthquakes'.

Q. Should include those sites?

A. Those records to get a measure of sensitivity of the results of the analyses.

Q. Gentlemen does everybody agree with that addition?

MR SMITH:

A. I just have a, I don't think it was, there was evidence of damage indicated, you know, there is a difference of opinion over the level of damage indicated.

JUSTICE COOPER:

Yes, no, no, this is supporting the view of Professors Mander and Shepherd and Dr Bradley but the report was not a complete sentence and I'm just saying, asking if it was common ground that those words should be added to make this an accurate reflection of the views of Professors Mander, Shepherd and Dr Bradley?

DR BRADLEY:

Yes, yes that's correct.

JUSTICE COOPER:

All right. I'm just conscious this is a document that everybody signed apart from two.

WITNESS CONTINUES READING BRIEF OF EVIDENCE

A. Beam column joints. There was still some differences of opinion between Professor Mander and Messrs Bradley at Compusoft on the beam column joint models but by the time the analyses had to be carried out these differences were very slight and it was still agreed to disagree. The model was analysed as it stood at that time. We were very aware of having to report to the Royal Commission by early July. We had to come up with some answers. The models used, as well as most of these modes will suggest, can only really represent the behaviour of the joint itself and not the effects of the joint behaviour on potential shear failure of the connection between the adjoining beams and the joints themselves. Such models are not available in SAP2000. It would require a large amount of time to develop and then calibrate against experimental results. Such time was not available at this stage in the process. It should be noted that there are no recognised modelling techniques for modelling two way interacting beam column joints. In addition there is little research on the structural typing configuration used in the CTV building so there will always be a considerable uncertainty as to the adequacy of any modelling of the joints.

Professors Mander and Shepherd and Doctor Bradley point out that Takeda modelling of the joint is peak oriented and that there is no cumulative degradation with only degradation due to post peak softening.

DR BRADLEY:

May I add just there for the record that should be pivot rather than –

PROFESSOR CARR:

Pivot yes.

DR BRADLEY:

At the end of the first line.

JUSTICE COOPER:

Q. Sorry Takeda the modelling of?
A. The word Takeda should be replaced with the word pivot.
Q. How is it spelt?
A. P-I-V-O-T.
Q. With a capital P or a small P. Is it somebody's name or is it a technique?
A. Technique.

DR BRADLEY:

It was a correction that came in after this was submitted.

PROFESSOR CARR:

If accumulative degradation is considered then more damage will concentrate in the joint and less in the columns. Hand calculations illustrate that a beam column joint has a lower moment capacity than the column moment capacity and hence that potentially the columns cannot reach yield. The analysis of the Compusoft report needs to be reconciled with this and there's a note at the bottom that Compusoft commented the joint model is a pivot model.

Results of further non-linear time history analysis. Compusoft Engineering Limited submitted the results of the revised non-linear time history analysis on the 13th of July, 2012. The report covers responses to the buildings subjected to ground motion inputs for all recording sites in the CBD for the February earthquake, for two of the sites in the September earthquake and for analyses using the September and February earthquakes such that the damage state of the building following the September earthquake was used as the starting state for the February earthquake. The results of analyses were not included in the terms of this panel and were presented by the representatives of the Compusoft Engineering Limited. Professors Mander and Shepherd and Dr Bradley argue that the Compusoft report requires discussion on the failure mechanism that is predominant, in particular one of the key outputs of the non-linear time history analysis as the collapse sequence. It is likely the collapse sequence will be different for different modelling assumptions.

Further issues. The panel was asked to consider some further issues. These are set out below.

Modelling of earthquakes. An email dated the 15th of June, 2012 Buddle Findlay suggested that the non-linear time history analysis should include modelling of significant earthquake events between 4.35AM on 4th of September 2010 and 22nd February 2011 so that the cumulative fatigue effect can be considered.

The only comments that came in from the panel was from Professor Carr and Dr Davidson. Magnitude 5 records are two magnitudes smaller than the

September earthquake ie released energies about 1000 times smaller than the energy released in the September earthquake. Most were at similar distance from the CBD as the September events, the exception being the Boxing Day 2010 earthquake. A study by myself Professor Carr using the Boxing Day earthquake on another building in the CBD showed that the building response to that earthquake when compared with the September and February earthquakes indicated the response that the Boxing Day earthquake was in the noise level of the earlier response. This may be a little different for the CTV building as its natural period of free vibration is shorter so the earthquake magnitude is still of the order of magnitude that is relatively small. Dr Davidson responds in a statement that yes it could be done but how many weeks were available to do and extract the results from the analysis. The suggestion was to carry out the September analysis then using the response spectra from the September to just prior to the February earthquakes give an estimate of the significance of the later shakes from the likely response of the structure. This has not been able to be carried out to date.

Masonry. On 15th of June 2012 counsel for the Royal Commission asked the panel to consider comments from Dr Clarke Hyland and the suggestion by Dr Hyland that there is the need for additional SAP analyses using the base Compusoft input files. These should be modified appropriately for considerate masonry wall and contact with surrounding beams and columns at the start of the February aftershock and calibrate the input motions to allow assessment of purpose and demands on the south wall and masonry infill prior to drag bar failure using the capacity set out in the BCR, I wasn't sure what the report was referred to now. Compusoft does not support scaling input motions to calibrate analysis.

JUSTICE COOPER:

Mr Smith, do you know what BCR refers to?

MR SMITH:

Building collapse report, DBH report.

PROFESSOR CARR:
The modelling of contact between the masonry infill and the columns could be modelled using gap elements to represent the contact no contact status between the masonries and the column. The panel have agreed earlier that the two limit states used in the earlier analyses was sufficient. The earlier Compusoft model had discussed such contact. However to do what was suggested would require a considerable amount of both computing time as well as man hours to set up the modelling, its calibration and then extract a solutions. This was not possible in the very limited time that was available.

JUSTICE COOPER:
In our folders this document has been signed by everybody except for you Mr Smith and Mr McVerry, but Mr Smith is that because you have not had the opportunity to sign it.

MR SMITH:
I thought I had signed another copy just a minute ago.

JUSTICE COOPER:
Okay so that is signed by you. And so far as you know Mr McVerry is signing it or did he not effectively participate.

PROFESSOR CARR:
It was a very minor participation. He was part of the panel. That's why his name is here. I presume it's how we get this document to him and back again is the moot question. I assume I'll leave that to the Commission.

JUSTICE COOPER:
Mr Elliott, is that in hand or is it not thought necessary to pursue Mr McVerry?

MR ELLIOTT:

I think he should sign it Your Honour and I will make sure he does. Certainly I don't think he has any areas of disagreement.

JUSTICE COOPER:
All right well we proceed on the basis that that's agreed. Professor Carr thank you very much for that and probably not the first time may I express our gratitude to you for conducting this process. We're very grateful.
Well Mr Elliott what should happen next?

MR ELLIOTT:
Well Your Honour firstly if the Commissioners had any questions about the areas of agreement I think they could be asked. If not then I suggest working through each of the areas of disagreement one by one and just asking Professor Carr and others to articulate their views and to pose questions if necessary.

JUSTICE COOPER:
Was it not thought appropriate to hear from the Compusoft representatives about the amended report?

MR ELLIOTT:
If that would assist to do that now before discussing the end questions.

JUSTICE COOPER:
If Dr Davidson or Mr Stewart or Mr Bradley could just explain, can I just ask you where we've got to, we've got another one this morning.

1215

MR BRADLEY:
Essentially what that is, is since we issued the first draft on the 13th we have post processed and a number of other bits of data and produced that and

distributed it amongst the panel and essentially that document just collates that and puts it in a document which is easier for reference purposes.

JUSTICE COOPER:
So as far as you're concerned anyway could we simply substitute this latest document for the other amended document that we've had.

MR BRADLEY:
Yes.

JUSTICE COOPER:
This is calling itself the second draft issue. Is there anything in the first draft issue that we need to be aware of which isn't in the second draft issue?

MR BRADLEY:
No, the intent is it does get substituted, it's just – produces.

COMMISSIONER FENWICK:
Are the page numbers the same?

MR BRADLEY:
No they won't be and nor will the figure numbers and table numbers.

COMMISSIONER FENWICK:
This involves a certain problem when this arrives on the desk this morning and the previous one was at a time when I could actually study it so my questions are noted in the old one.

MR BRADLEY:
That's fine, there's just additional information, the information in the previous draft report is unchanged, there is just additional so if you want to go off the first draft that's fine by us as well.

JUSTICE COOPER:

What I think would be helpful, well there's two things I'd like to ask you Mr Bradley first is this says, second incomplete draft issued. Does that mean there's another iteration of this to follow?

MR BRADLEY:

Essentially what we've done is we've completed the analyses that we undertook as part of the second review but we haven't finished post processing all the results for all the records and the reason it says incomplete is because we have, and we do intend to complete processing those and publish those results at a later date.

JUSTICE COOPER:

So what are the categories of information that is yet to come?

MR BRADLEY:

There's just – we have covered essentially all aspects, just not for all records so we've produced information as regards a column behaviour pullout but we haven't produced plots for all critical columns or diaphragm actions for every single record.

JUSTICE COOPER:

So the material that's yet to come is in the nature of further elaboration of material that's already here?

MR BRADLEY:

Correct.

JUSTICE COOPER:

And secondly, is it draft for that reason?

MR BRADLEY:

The draft is because for that reason and because given the time constraints that we have we hadn't had time to comment on a number – the information that's been presented as we normally would and we wanted to flesh out the report later and add some conclusions.

JUSTICE COOPER:
Is that, for what purpose would you be doing that?

MR BRADLEY:
Just for completeness more than anything else.

JUSTICE COOPER:
And when would that happen?

MR BRADLEY:
We'd work on it straightaway and publish it as soon as possible. I mean if the Commissioners are satisfied that what we have there suitably represents the trends and behaviour of the building, then it might not be necessary to publish all results for all records. I guess we'd take a directive from yourselves with regard to that.

JUSTICE COOPER:
Well the best thing from our point of view is to have a document that we can rely on now. I infer from what you're telling me is that we so far as you're concerned we can rely on this document but that there might be another iteration of it which is simply more complete. Is that – we in that category?

MR BRADLEY:
Yes.

JUSTICE COOPER:

So it's not anticipated that there would be significant change between the conclusions that are either already expressed or able to be inferred from this document.

MR BRADLEY:

No, not in my opinion.

JUSTICE COOPER:

Mr Stuart do you wish to –

MR STUART:

But I would like to point out that while we anticipated that there would not be significant change, we certainly cannot guarantee that, as Derek said, we have not processed a significant portion of the data still, so this should be considered to be a work in progress in many respects.

JUSTICE COOPER:

Dr Davidson.

DR DAVIDSON:

The first report issued on the 13th followed outputs of the trends that we were encouraged to look at from the DBH, what we would call the DBH analysis. Obviously they were easier for us to process because we had post-processing mechanisms put in place and also it meant that those results could then be directly related to the results that we achieved last year for the Department of Building and Housing. However this new model if we like that we've, under this panel does have additional features. Obviously that was the intention of it and we have gone to some extent to report the results of those features and what has happened but I don't like to say it, but we have been under a lot of time pressure and we have – why it's been listed as incomplete is that we are just a little fearful that there may be some features we have not gone around looking at the – and we may find something just a little different. What is here we can say with quite a lot of confidence is correct, but there may be more things to be added to it if we had time.

JUSTICE COOPER:
Thank you.

COMMISSIONER FENWICK:
I'm just curious about one thing – the decision to replace the fairly simple column and the beam column joint with a more complex one, I'm wondering what the logic behind that was. If I can sort of take the long way, I thought it might have been tackled, I can't see that the new model would have made any difference, appreciable difference to the inter-storey deflections you'd get. I would have thought it would have been more logical to take those inter-storey deflections and apply them to a much more detailed analysis of the beam column joint in the beams in the location, just to push it backwards and forwards between those regions. I'm just curious as to why you decided you needed to upgrade that model to something which could never be accurate but possibly one might have got more accuracy by taking it out and doing it as a separate analysis which would presumably have allowed the whole analysis process to proceed more rapidly. Can you tell me why did you go in that particular way?

MR DAVIDSON:
This second series of analyses were undertaken as a result of balanced views from the expert panel, as Compusoft acted if you like as a processing arm so a lot of the decisions were not necessarily our decisions.

COMMISSIONER FENWICK:
I'm sorry I put the question obviously to the wrong person. Perhaps I should divert it to Professor Carr.

MR DAVIDSON:

Well I would like, if I don't mind that's part A, but part B on the speed we would say that while there was man hours involved in changing the modelling the actual speed of computer solution and for those who are not aware some of these programmes run for a week, that order, so it was actually a little faster so the new modelling was faster so on the face of it, it appeared that it would be a slower model but in fact it was not, so there was a little bit of an upside from that point of view, but the reasoning maybe you need to do up to report back to Athol.

PROFESSOR CARR:

The a general feeling of the panel was it likely that the beam column joints were actually weaker than the columns themselves and it was therefore thought that the beam column joint had to be part of the model. If the beam columns misbehaved then the columns themselves might not get the same degree of deformation that you'd have got if the column joint was assumed to be elastic as was it done in the first analyses. There was a fairly general agreement that the beam column joint needed to be part of the model, the difficulty was in finding a suitable model that one could use. There are some models around, most of them are for two dimensional joints and that would be adequate for the interior joints in the CTV building because they are largely two dimensional, but the exterior frame joints not, they are two or three-way joints and therefore it's a bit more of a problem to get a model which is accepted and (b) which could be calibrated against the existing experimental data and that's even more in the case for this building as the beam column joints don't look like any beam column joints I've seen reported in the literature anywhere, and so there'd be a lot of difficulty in getting too elaborate a model at this stage. So what's been done is a fairly simple model. It's been done for steel frames for a long time as a pivot model at the intersection, but it has real problems in terms of concrete modelling. There's a suggestion I made in 2006 that to do the job properly you need to model the joint and the surrounding plastic hinges in the beams and the columns as all part of one element then the parts can talk to each other to find out what is happening. In a good object oriented programme one element doesn't know anything about its neighbours. It's not meant to. But if it's all in part of one model then you could do that. Unfortunately the post doc student we had in 2006 went off to GNS and that work's sort of been sitting on the back burner ever since.

COMMISSIONER FENWICK:
So the issue really remains doesn't it, that this is an area of considerable doubt about what went on?

PROFESSOR CARR:
That's undoubtedly true and I think that's what we've said and it's also said in the interpretation report that the beam column joints are an area of contention as to what actually is going on.

JUSTICE COOPER:
Mr Smith do you wish to add to that?

MR SMITH:
Not really, no.

JUSTICE COOPER:
Dr Bradley?

DR BRADLEY:
No not at the time thanks.

JUSTICE COOPER:
So shall we focus now on areas of disagreement and the first of which is the drag bar strength. Are you happy with that? Perhaps could you start us off on that Mr Smith?

MR SMITH:

Yes well I think it's recorded there that we were needing expected strengths rather than lower characteristic strengths which were the basis of the Beca calculations, and that is certainly one reason why the Beca values would be lower than what we had estimated. I accept that it would be best to do as requested by Professors Mander, Shepherd and Dr Bradley that we could do analyses with both sets of values. I accept that, but in the timeframe we only had time to do one set of values. So my recommendation was to use the upper bound values which I consider to be upper bound values. The results show that we still get some disconnections of those drag bars so my reasoning is that with a lower value we'd obviously still get disconnections so I think we did agree that in a teleconference with the group, so.

JUSTICE COOPER:

Yes, Dr Bradley do you wish to speak to this issue?

DR BRADLEY:

I would simply add the comment that one comment made on the Beca report was that the values were lower because the used characteristic strengths rather than nominal strength. Other comments for example from Professor Priestley argued that the reason the values would presumably be lower than those in the original Hyland Smith report is that failure could initiate from flexure of the connections rather than simply from shear failure which as I understand was the focus of those drag bar capacities first calculated. In terms of the comment just then from Mr Smith about what happened subsequently, I think it's important to understand that just because drag bar failure is suggested already with these upper bound values, with lower values you could get, you would presumably also get drag bar failure but the consequent effects might be quite different because presumably you would get that failure earlier in the analysis and that would change the dynamic properties of the structure.

JUSTICE COOPER:

With what result? What possible consequences are there of that different?

DR BRADLEY:

If for example the drag bar failure happened just taking for argument sake one second earlier, the deformation of the structure might be in the north direction as opposed to the south direction, and therefore the sequence of events which happens following that could be quite different.

JUSTICE COOPER:

Yes.

COMMISSIONER CARTER:

I think that would have kept the whole issue of sequencing here where we've got several uncertainties coming into play, so this just appears as if it's another uncertainty in terms of sequence?

DR BRADLEY:

Agreed.

COMMISSIONER CARTER:

Correct, okay, thank you.

JUSTICE COOPER:

Professor Carr, anything to add?

PROFESSOR CARR:

I agree in the general principle it will change the dynamic properties. How we're not quite sure. Just because you disconnect the torsional connection of the floor doesn't mean you immediately get large floor rotations. The late Professor Pauley got caught out with this. We did some work in the late 1990s modelling structures where we had, we were looking at torsional behavioural buildings and trying to get redundancy in the torsional response. He had come up with some simple design strategies. He had a postgraduate student run some analyses. The results agreed with his simple hand calculations and he wrote a paper on this. A little bit later he realised the student had left out the torsional inertia of the floor which changed the answers completely and he had to retract the paper and then write a new one. We had another PhD student who was analysing buildings again for torsion and he took an example of a building which has a wall and one edge only. Statically it's unstable. In the earthquake it didn't rotate. The wall yielded in shear and the floor basically translated. The rotational inertia is very large. It takes time for it to start moving. If it does start moving then you get some fun because it doesn't stop it but it depends on the earthquake excitation how it changes direction as to what the results will be and to a certain extent one sees in the Compusoft results the time history analyses don't show the same level of floor rotations that one got from the response spectra analyses and I think that's again that same rotation inertia considerations. If you've lost the ability to put that rotational inertia action into the floor then you can't get the rotation anyway. So it will affect the answers how ideally we will do sets of analyses on concrete strengths, on drag bar strengths, the modelling of the joints but do we want to wait a decade for the answers? It would be nice.

JUSTICE COOPER:
Well we can't.

PROFESSOR CARR:
No.

COMMISSIONER FENWICK:
Perhaps I can summarise that by saying we don't know what would happen, but the chances are that the release of the drag bars earlier on would not be positive, or can we say it would all be positive and we should've released them from the beginning? I mean it's, by and large you'd say well the earlier that drag bar went the more likely the structure is to fail isn't it?

PROFESSOR CARR:

I would think that the earlier that they fail, that the drag bars fail, the more likely that the structure will fail a little earlier.

JUSTICE COOPER:
Do you agree with that Dr Bradley? Mr Smith?

MR BRADLEY:
My only comment would be that the failure of the drag bars at a certain level would increase the demands on the remaining part of the slab connected to the north core. The influence of that would need to be calculated but it would increase the demands.

MR SMITH:
I think the chances are that it would bring the failure time earlier if we disconnected earlier.

JUSTICE COOPER:
Well the next identified area of disagreement is under the heading, "Column plastic hinge modelling," but I'm not sure whether this is a disagreement, properly so called. It seems to me about, a complaint about the limitations of the model that was used?

PROFESSOR CARR:
Yes it is and it's, it probably is not terribly significant in the circumstances. If it's other types of plastic hinges I might be concerned but I think in these cases probably not.

JUSTICE COOPER:
Yes. Dr Bradley?

DR BRADLEY:
Yeah I would agree. I think the significance of the uncertainties in the beam column joint are far larger than the column.

JUSTICE COOPER:
Mr Smith?

MR SMITH:
I agree.

JUSTICE COOPER:
Mr Bradley?

MR BRADLEY:
Yes, I agree.

JUSTICE COOPER:
Anyone else?

MR STUART:
I agree.

JUSTICE COOPER:
Thank you, that was Mr Stuart. I should explain we've got people off site who are transcribing what they hear through these microphones so if I'm a bit pedantic sometimes that's why. I'm trying to make sure that they are in a position to give us an accurate record of who was talking.
So then we have floor diaphragm modelling and an issue about equivalent flexural stiffness of the floor and here the reported conclusion with which you all agree is that the sensitivity of the results, the effect of stiffness used, should be examined.
1235

DR BRADLEY:
A. Yes it's one of those sensitivities.
Q. Yes so that's a work for the future is it?

A. Yes, in the future.

Q. Right, there's nothing the Royal Commission can do about that?

A. Not unless they direct so.

Q. Well we have to bear in mind we've got a reporting date in early November so what is the, as a lay person compared with all of you, what is the implications of this issue in terms of why the building collapsed?

A. This one thought is that if the floors are vibrating up and down on the beams that will actually increase the transfer of inertia forces to the beams and hence to the columns. So there's a what you might call a trampolining of the floor is a possibility. There's reports of sensitivity of floor movement so that is a possible argument that one might follow there. There's the other argument about in terms of the reinforcing, you know, connections. Do we actually have other sorts of failures in the floor itself that are not currently modelled very well in the analyses. So, but as I said before, it's in the future largely I suspect that this building is going to be the research topic for the next decade amongst researchers worldwide. We might actually get some little more detail in the future but we've got to come up with some reasonable model at the present time and I think that will not have a great effect on what we see at the present time in the sequence of failure and the failure mechanism. But it is again one of those variables that is a little uncertainty just to how much effect it's going to have.

JUSTICE COOPER TO DR BRADLEY:

Q. Dr Bradley do you wish to add anything?

A. No, just it was not, the sensitivity wasn't considered because of time considerations in terms of our reporting to you but it would be, the value used was that advocated by Professor Mander – a value of 0.1 of the maximum stiffness. Professor Priestley made the comment that he felt this value was too low and that it should be higher. If a higher value such that Professor Priestley was advocating was used that would make the vertical vibration period shorter which would then make the accelerations in the vertical direction that the floors felt larger and so you would have the potential for vertical effects to become more significant if that was the case.

Q. Under Professor Priestley's scenario.

A. That's correct.

Q. Yes that's interesting thank you.

COMMISSIONER FENWICK:

Q. Can you just confirm 0.1 or was it 0.2?

A. According to Professor Mander's calculations and those which were then used it was 0.1 but I did notice also in this revised brief that a value of 0.2 was provided so I'm not sure of the reason for either of those.

MR BRADLEY:

A. I believe it was 0.1 that we used. I'd need to check that.

Q. Okay so that seems a low value. Of probably more significance too, what was the axial stiffness assumed to be?

A. I think it was 0.32 or 0.35.

Q. AG?

A. Yes. Sorry I'd just like to point out that we did actually explicitly model the stiffness of the floor adjacent to the beam lines and that was the only point we could be sure of the contribution of the metal decking, ie: because there was no, it's only just seated on the edge of the beams. We assumed that there was no contribution at that point and we could actually explicitly model the stiffness of the floor using the non-linear loaded shell approach and we did that so that has been accurately captured. I guess it's the mid-span of the beam that needs to have a sensitivity analysis done on it.

Q. But overall it's still 0.3. or 0.35, if I heard you correctly, AG for the axial stiffness?

A. For the main body, not the non-linear. We have non-linear strips adjacent to each beam line and the actual stiffness of that will be modelled on a fibre level if you like.

Q. What about normal to the beams.

A. Yes.
Q. At right angles to the beams.
A. Yep.
Q. 0.3?
A. In the mid-span outside of the non-linear strips we've used 0.35 the AG and at the non-linear strips the axial stiffness is considered implicitly in the model.

MR SMITH:

I'll just add the 0.2 figure that, I can't remember where it comes up in the statements, is 0.2 IG. It was a number that I had calculated based on the stiffness that was used in the model so, obviously, Dr Bradley may wish to check that but I was aware that Dr Mander had suggested 0.1 G but my calculation of the value that was used I thought equated to 0.2 G. So now the other thing I'd point out is the original DBH analyses did carry out a sensitivity study of the floor stiffness and the effect on column axial loads. If we relate it back to that I believe the values in the latest analyses are at the lower bound of what was used there so we did do further calculations with higher stiffness in the original report.

JUSTICE COOPER:

Mr Derek Bradley do you wish to respond to that?

MR BRADLEY:

Yes we did undertake a sensitivity analysis in the original DBH analysis. It was using a different floor stiffness to what we have in the new analysis but we considered a bound by doubling and halving the stiffness and looking at the response of the floors relative to that. It did indicate that some column axial forces went up and some went down as you increased or halved. It depended on the relationship and the stiffness of the beams that spanned into it, not just the floor itself. So there was no hard and fast rule over, you know, if you double the stiffness you're going to double the axial loads. It varied across the building.

DR DAVIDSON:

I think, just to add clarification. There was a criticism on the damping model and you heard the word from the first report and so for the second lot of analyses that we've just considered we chose a, made sure there was a lower level of damping, approximately two percent damping, which Professor Fenwick will understand, and that was aimed at what we might call a mode where it had the most significant amount of vertical movement but what we have to all understand is that all the floors on the building and all the floors across, all the floor spans over any particular floor will not necessarily be going up and down together because they have different spans. So that probably would help describe why the results that Derek saw in the first sensitivity, when you change the stiffness up or down the ratios will change because the beams are getting involved too, so it's reasonably complex. Can you remember when you went up and down a factor of two on the stiffness as you say changed but did they change a lot?

MR BRADLEY:

It was, it varied, some changed by 10 percent, others changed by about 25 percent as far as the axial load variation on the column was concerned and it varied from record to record as well given the different frequency components for each earthquake record.

JUSTICE COOPER:

Anybody wish to add to this discussion? Duration of shaking – the issue here appears to be, if I'm understanding this and I may not be, but is it, Professor Carr is the difference captured in paragraph 26 of the report is it?

PROFESSOR CARR:

I suspect that's the major part of the –

JUSTICE COOPER:

It's a question of the start of durations.

PROFESSOR CARR:

That's the major part, yes. If they'd run the September records for longer we might have seen a few more cycles of inelastic behaviour in some of the members that were being inelastic but the models that I used do not capture cumulative effects. They're basically measuring changes to properties on displacement maximum. We've already passed that. The smaller cycle stuff is not going to affect the shape or the envelope. The models that are used in SAP2000 do not have a cumulative strength degradation options. There are one or two programmes that do but obviously the possibly could be done but again would take time and we don't have that time or didn't have that time to contemplate those issues. As for the start of the beginning of the record my opinion is that it would have very little effect. Traditionally for seismic analysis we've never had the beginning of the records apart from the past few years. The instruments had to be triggered before they would start recording. You've lost the P-wave before the instrument was triggered and we've never got upset about that. In this case the record is there and may be for forensic studies we may include all of it but, as Dr Davidson has said, some of the analyses take a week as it is and we had very little time at that point to do these analyses. We had to be a little bit circumspect about how much of the record we included in making an engineering judgement.

DR BRADLEY:

I would agree that given the demands of time and so on and so forth that the assumption made was reasonable. I'd pick up on the comment Professor Carr made that because of the nature of the model used that the effects of cumulative damage could not be fully appreciated. I wouldn't say they couldn't be at all appreciated. I think the modelling of the column using a fibre element is such that it could pick up cumulative effects but certainly the modelling of beam column joints and so on could not pick that up.

JUSTICE COOPER:

And again for my purposes what's at stake here in terms of how the building collapsed.

DR BRADLEY:

Certainly, I think if you take the postulative of Professor Mander, for example, who places a large emphasis on cumulative damage then the modelling limitations are such that they are not picking up the full extent or the presence or absence of cumulative damage.

JUSTICE COOPER:

But his argument, as I've understood it, goes back before the February earthquake doesn't it, which is at this point?

DR BRADLEY:

Yes so, for example, in the September earthquake the analyses was stopped after the large part of the shaking had been completed in that September earthquake. If you carried on the analyses for example another 10 or 20 seconds that would involve several more cycles and therefore potentially more cumulative damage. As Professor Carr mentioned, the nature of the model used is such that it's not picking up that type of damage anyway so therefore for the purpose of the analyses I understand the truncation of the ground motion.

JUSTICE COOPER:

The next point is Ground Motions Used in the Analyses and the argument from Professor Mander and others that the Christchurch Hospital and Rest Haven site ground motion should be included. Would you care to expand on that?

DR BRADLEY:

Yes certainly. That was simply a time issue Sir. As we've mentioned, in re-running the second phase of analyses we considered the fact that the structure could sustain some slight damage in the September earthquake and then subsequently examine what would happen in the February earthquake. However, to date that has only been considered in two cases and this statement of the evidence there is just to say that it should also be done for the other two.

JUSTICE COOPER:
Right, so is anything happening about that?

DR DAVIDSON:
Well it has been only a time issue. We did not disagree with that request but it just basically we had to turn up today.

JUSTICE COOPER:
So those sites have only been included with respect to the February analysis. Is that right?

DR DAVIDSON:
Correct, the REHS.

MR BRADLEY:
The REHS has only been considered for the February analysis, yes.

JUSTICE COOPER:
And what about the Christchurch Hospital?

MR BRADLEY:
For the February event only.

COMMISSIONER FENWICK:
Was there any significant difference in the two analyses you did carry out where they were carried out one after the other?

MR BRADLEY:

For the CGBS record there was no, I would say no significant difference with regard to the main performance indicators. The CCC record indicated that there would be considerable inelastic demand in the columns under the Darfield event, the September event and because of that it became difficult to infer anything for subsequent analyses because the Darfield one essentially showed that some of the columns may have spalled, possibly failed under the Darfield event. There was, with regard to diaphragm disconnection, sorry the CCC event did indicate that there was a slight difference in times but again this was influenced somewhat by the column performance and the CGBS record there was no change to diaphragm disconnection times, no significant change in diaphragm demands, displacement demands were essentially identical or qualitatively the same.

DR BRADLEY:

I agree with the interpretation of the analyses simply to say that of any aspects of the building which are likely subject to cumulative damage they again would be the beam column joints and because of modelling limitations that wasn't considered.

JUSTICE COOPER:

You need to be a bit closer to the microphone again I suspect.

DR BRADLEY:

Sorry, would you like me to repeat that. Again I agree with the interpretation of the analyses as they've been stated but just simply to note that if there are any evidence of cumulative effects they would be pronounced of anywhere in the beam column joint and because of the nature of the modelling the complexities that have already been noted that type of cumulative damage wasn't considered.

JUSTICE COOPER:

Beam column joints – Now Professor Carr can you just summarise, you say in your paragraph 28 that the differences of opinion on the beam column joint models were very slight and it was agreed to disagree. I take it that that means that it was not considered an issue that needed to be thrashed out.

PROFESSOR CARR:
It was a convergent process and at some point we never did enough iterations, if you like, to get the full convergence. There was some slight variations and the emails sort of dried up somewhat so it was a bit hard to wonder exactly what was happening. There were little differences I noted between the responses from Compusoft and the emails from Professor Mander but in general he was saying that they were getting close to what he thought was the right model and after that there was the deadline to get the analysis underway had been reached and there wasn't much further said.

JUSTICE COOPER:
But it was near enough.

PROFESSOR CARR:
That was my interpretation. It was near enough and the little differences (inaudible 12:54:05) days of iteration might have got there. We didn't have those.

DR BRADLEY:
I wouldn't say that was entirely my opinion. I got the feeling from the conversations by email that the views of Professor Mander were quite different. He advocated initially to using a beam column joint model based on an American design standard which he later admitted was arguably slightly conservative but from again my interpretation was that he still felt the model used was slightly conservative. I think that it is important to bear in mind that

while we proceeded towards some form of convergence, we all recognised that the model we were converging on was still a gross simplification of the actual behaviour.

MR SMITH:
I agree with that.

1255

JUSTICE COOPER:
Paragraph 29 as I read that in my layman's terms is that the potential complexity of these column joints is not something that SAP2000 could respond to and indeed because of the particular manner in which these elements of the CTV building had designed it would be difficult to find any model that did. Is that right?

PROFESSOR CARR:
That would be my interpretation. I would say there's two or three PhDs somewhere down the line to start looking at modelling of these sorts of details.

DR BRADLEY:
And getting a good joint model. We still don't really have a good joint model that is accepted.

JUSTICE COOPER:
Dr Davidson did you wish to add to that?

DR DAVIDSON:
I guess from Compusoft's point of view that we would not see ourselves as concrete experts or reinforced concrete experts so we were taking advice from Professor Mander and Professor Priestley in the main in trying to

interpret what they were telling us and they weren't necessarily agreeing themselves so we sort of had to do best with what we could do really.

JUSTICE COOPER:
All right. Dr Bradley.

DR BRADLEY:
I would agree.

MR SMITH:
I just have one comment that in that disagreement between I also relied on the advice of Professor Priestley and Mander in that regard. The first recommendation of Professor Mander was to simply limit the joint strength to a proportion of the column strength which wasn't really acceptable from my point of view because that predetermines that the joints are going to fail before the columns and I was encouraging and I don't know whether it got there in the end an independent formulation for columns versus joints so we could get an answer out of the analysis rather than predetermining the results but it seemed there was difficulty in doing that. We had one piece of advice from Professor Priestley was that in his experiments that he's conducted he normally observed the columns to fail prior to the joints and that was conflicting with the other advice so it was just hard to resolve yes.

JUSTICE COOPER:
Well he's not here is he?. He might –

MR SMITH:
I was just commenting on that though.

JUSTICE COOPER:
Yes Dr Bradley.

PROFESSOR CARR:

In terms of the concrete I prefer to Professor Priestley and Commissioner Fenwick.

JUSTICE COOPER:

Do you wish to add, ask a question.

COMMISSIONER FENWICK:

I'm just wondering if you all agree that the joints were weaker than the columns. Is that universal or is there some difference?

DR BRADLEY:

I certainly agree with that.

PROFESSOR CARR:

I personally would agree the joints don't look to be very good at all.

JUSTICE COOPER:

Which leaves you Mr Smith.

MR SMITH:

Well I'm relying on the analysis that we've conducted which almost shall we say was converging on agreement to modelling of the joints and that I believe showed some damage in the joints but not necessarily failure of the joints you know, so I'm just relying on the analysis that we ended up with.

COMMISSIONER FENWICK:

Which relies entirely on the modelling and you have confidence in your modelling least I'm not sure you do have confidence in your modelling reading what you have written there.

MR BRADLEY:

We are confident that it's been incorporated the way in which we intended but again given the amount of uncertainty and the performance of those joints and the modelling, the limitations of the model that we employed we can't say definitively one way or the other whether the joints would have initiated collapse or the hinges.

COMMISSIONER FENWICK:
It's very difficult to model those hooks in the middle of the joints.

MR BRADLEY:
Exactly.

JUSTICE COOPER:
Professor Carr, do we then, the rest of your report seems to be reportage rather than outlining areas of disagreement, is that right?

PROFESSOR CARR:
Yes the other issues yes they are reporting. We had all this discussion what happened afterwards. We run some analyses which Compusoft will report on shortly I hope and the final issue is the other issues that were raised after the panel had actually met and the discussions that followed on that.

HEARING ADJOURNS: 1.01 PM

HEARING RESUMES: 2.17 PM

MR ZARIFEH CALLS
DAVID FALLOON (SWORN)

Q. Is your full name David John Falloon?
A. It is.
Q. You reside in Selwyn?
A. Selwyn District, yes I do.
Q. You are a Structural Engineer and operate your own business, Falloon and Wilson?
A. I am.
Q. Have you got a signed brief of evidence in front of you, prepared by you?
A. I have, yes.
Q. Can I ask you please to read that from paragraph 2 and just to pause when we come to a document which will be brought up on the screen and I'll get you to identify it.

WITNESS READS BRIEF OF EVIDENCE COMMENCING AT PARAGRAPH 2

A. "I have a Bachelor of Engineering Degree from the University of Canterbury. I graduated in 1967. I am a member of IPENZ and have been since 1970.

The CTV Building:

In 2000 I was engaged by Cemac Partitions to provide structural drawings for two specific aspects of the fit-out for CHTV, as it then was, for its tenancy of the CTV building. Documents on the Council file record that this tenancy intended to be operational by 1st of June 2000.

The installation of a staircase from Level 1 to Level 2 was the first of two stages Falloon & Wilson was engaged to advise on. This involved the cutting of a hole in the Level 2 floor slab. The second stage involved the

creation of an entranceway in the south-east corner of the building on Level 1 and we were involved with the entrance canopy structure.
I no longer have my records relating to this matter because after the time of the 22 Feb 2011 earthquake my office was in St Elmo Court building and this has since been demolished due to earthquake damage. Accordingly, I am relying on my memory and the Christchurch City Council file in respect of this matter. A copy of the Council file has been provided to me by Counsel Assisting the Royal Commission.

Producer Statement:
I viewed the drawings prepared by Falloon and Wilson in 2000 for the internal staircase opening between Levels 1 and 2 at the south end of the CTV building.

EXAMINATION CONTINUES: MR ZARIFEH

Q. I will just get you to pause there and I will get them brought up so that you can confirm they are the drawings.
A. Yes that is the drawing.
Q. I think there's two pages of drawings, just look at the next page .3
A. Yep, drawing 1 is the hole, illustrates the hole in the floor and drawing 2, S2, the canopy on the corner of the building.
Q. So it's S1 that we're particularly concerned with?
A. Yep.

WITNESS CONTINUES READING BRIEF AT PARAGRAPH 8

A. "The bottom right-hand corner of the drawings show that the design work was done by CWT. I recall that this was a short term employee called Chris. I have signed the drawings as having checked them. That's my signature.
An application for a building consent in respect of this work was made by Amanda Coats of Warburton Team Architecture on the 19th of April 2000.

EXAMINATION CONTINUES: MR ZARIFEH

Q. Get that brought up. That's the application?

A. That's the application.

Q. Carry on please.

A. "At that time Falloon and Wilson were nominated as the structural engineers for this project but we had not done enough of the design work to submit drawings with the application.

I filed a Producer Statement with the Council on the 26th of April 2000.

Q. Is that the document shown there?

A. That's the one. This pre-dated my preparation of the structural drawings. This is not normally a process I would adopt but the intention was to progress the application through Council departments other than structural. However, I did advise the Council at that time that our design work was in progress.

Q. We'll have a look at that document, 107.

A. That's right, yes.

Q. That's your letter to the Council?

A. That's my letter to the Council.

Q. Paragraph 11.

A. "The drawings for the internal staircase, 2774/S1, were filed with the Council on the 4th of May 2000.

Q. We'll just look at that letter, 95.

A. Yes that's the letter signed by me.

"Stair Opening and Analysis

I recall viewing the original drawings in 2000 and determining that the floor structure was HiBond with a concrete topping spanning north-south between concrete beams running east-west.

In checking the design I carried out calculations to determine the size of the steel trimmer beams required to support the edges of the floor around the opening for vertical loads.

I did not consider that the lateral load paths from the floor diaphragm would be altered because the area cut out for the stair opening was only a small proportion of the total floor area. The rest of the floor diaphragm was available to transfer lateral loads to the shear walls. It was my opinion that Level 2 would not attract as much lateral action as the upper floors.

The design required the slab edges to be supported on steel trimmer beams and a steel post. This beam and post arrangement is shown on drawing 2774/S1. The opening was at the eastern end of the southern shear wall and between two east-west reinforced concrete floor beams. The south end of the trimmer beam to the west side of the stair opening was to be bolted to the side of the south shear wall near the end of the wall. The other ends of the north-south steel trimmer beams were to be fixed to the sides of the concrete floor beams. The stair itself was timber-framed and self-supporting.

On the 23rd of November 2000 I provided a Producer Statement – PS4 – Construction Review to the Council confirming that I had carried out periodic reviews of the work and that the work had been completed to the extent required by the building consent. I remember visiting the site to observe construction but do not recall any detail about that.

Q. I'll just get you to look at that document. It's .37, 0009.37.
A. That's the PS4 that I signed and wrote.
Q. Now can I just ask you a couple of questions in addition. You have I think been shown lunchtime and read the statement of William Holmes who has carried out a review of the diaphragm forces, if you like, if that's the right word, to determine whether there was any problems as a result of the hole being put into the first floor?
A. I have, yes I have.
Q. You've read that?
A. Yeah.
Q. Now you'll see that considerable calculation has been carried out in relation to those diaphragm forces?
A. I did yes.
Q. Did you do any calculations at all of that nature at the time of the alteration?
A. I did not, no I didn't.

Q. Right, but from what you said in your brief you gave the matter some thought?

A. I considered that the proportion of the slab cut out would not affect the diaphragm action of that particular floor.

Q. Because of the size of the hole?

A. Yeah, it was about 1.2% of the total floor area.

1427

Q. The other matter I wanted to ask you about was you'll have noted that in the last paragraph of Mr Holme's brief he says, "It should be noted that the connection of a trimmer beam with drilled in dowels at the end of a major shear wall should have included a note to carefully monitor drilling and avoid cutting of bars," did you see that?

A. I do, yes I did, yes.

Q. Now do you, well firstly do you accept that as a criticism?

A. I do, yes I do with hindsight I accept that, but we do try when we're, when we're drilling into existing concrete to place the holes so that they do miss.

Q. That was the other thing I was going to ask you about. I understand from your last paragraph that you remember going to the site to observe construction but you can't remember any details of that. Are you able to say whether you would've supervised things such as the drilling of holes into the shear wall to connect the beam?

A. I don't believe that I did but it's a matter of memory. I don't remember observing.

Q. And so you can't help at all in terms of whether or not there was a reinforced bar met with a drill or not?

A. Well in this report and obviously Mr Holmes had got access to some information that I haven't got access to which seems to suggest that the holes were shifted in the end plate of the beam to avoid maybe a reinforcing bar was struck with a drill and had to be shifted so it didn't penetrate the bar.

Q. So he's referring to photographs since the collapse?

A. Yeah.

Q. Yes, but you can't recall that occurring?
A. I don't recall, I don't recall that.
Q. Can you recall any problems with the construction at all, the calibration?
A. No none whatsoever, no.

CROSS-EXAMINATION: MR REID

Q. Yes Mr Falloon, I'm counsel for the Christchurch City Council, I just have one point in clarification. As I understood your evidence the producer statement that you provided was prior to the drawings that you provided to the Council?
A. That's correct.
Q. But it was your intention, I take it, that the drawings that you subsequently provided were to be covered by the producer statement that you provided (inaudible 14:29:50)?
A. That was the intention, absolutely.
Q. And the Council treated it in that way?
A. I believe so yes.

CROSS-EXAMINATION: MR ALLAN – NIL

CROSS-EXAMINATION: MR RENNIE

Q. Mr Falloon you've been in practice on your own account, Falloon and Wilson for what about 25 years?
A. Since about '82, yeah.
Q. So a little longer than that, nearly 30 years. And I think prior to that you were the in house engineer of a company known as Industrial Holdings?
A. That's right, yes.
Q. And that company was involved in the construction of a number of multi-storey reinforced concrete buildings in Christchurch?
A. Absolutely, yeah.
Q. One example would be the Contours building?
A. Yes, yes.
Q. And you were responsible for the design of that?

A. I was.

Q. Would you accept the description of that as an eccentric building in the sense of not being regular in shape?

A. I would, the shear walls were on the, I think, what would it be, one corner of the building anyway and there was a balan – well and a balancing shear wall was further out closer to what is it, Armagh Street, yeah.

Q. Do you recall approximately when that building was built?

A. Well it was, I believe it was prior to CTV. I'm not quite sure exactly, do you have the information?

Q. Not beyond what you've just stated.

A. No. It was very similar in appearance. I believe that it was the same architect.

Q. And this was a design that you personally did while with Industrial Holdings?

A. It was, yep.

Q. Another example would be the building in Victoria Street, six to seven storeys between the Caxton Press and Warren and Mahoney, do you recall that one?

A. That was, can you, was that occupied by Education House was it?

Q. It was Specific Brands I think in its most recent title?

A. Yeah, yes that's right and I was involved with that one definitely.

JUSTICE COOPER:

Do you have an address Mr Rennie?

MR RENNIE:

I can only tell you, Sir, Victoria Street between Caxton and Warren and Mahoney but I will get you a precise address Sir.

CROSS-EXAMINATION CONTINUES: MR RENNIE

Q. And that's a building where one wall shattered and the earthquake reinforcing buckled and the building was close to collapse. Did you know that?

A. I did, well I was told by someone who must've rung me that one of the walls was damaged, yeah.

Q. Did you look further as to how or why that shear failure occurred?

A. I did not, no.

Q. And without labouring the point you would've done a number of other buildings of reinforced multi-level concrete structure up until at least the mid '80s?

A. Yes.

Q. Another one would've been in Kilmore Street, what was known as the Goldcorp building?

A. Yes that was a steel frame building.

Q. And then you moved out into private practice, did you continue your interest in designing buildings of that type when you came into private practice?

A. Yes, yes I did, yeah.

Q. And would you say that when you came to be commissioned in relation to the CTV stairwell that you took into account your experience from your previous work with reinforced multi-level concrete structures?

A. Well I didn't take a detailed interest in the partic – yeah, you know, I know what multi-storey, how they behave, yeah.

Q. Yes because you see you indicated to my friend Mr Zarifeh that you didn't do specific calculations because you were confident that what you were proposing for that building would comply?

A. I believe so, yes.

Q. Although there's a little confusion, I think, in that regard Mr Falloon because if you turn to your brief of evidence, and if you have a look at paragraph 13 you actually say there that you did carry out calculations, do you see that?

A. That's right. Well I did, I'm talking about gravity loads rather than lateral loads.
Q. So would it be fairer to say you did some calculations, but not a full set of calculations?
A. Well I did, yes that's right, I didn't check the lateral load resistance of that diaphragm.
Q. Now you've indicated that, just for the record, 123 Victoria Street, you've indicated that you did have available to you the original structural drawings. You say that in paragraph 12?
A. Mmm, well I say we would've looked at the structural drawings relevant to that particular, because we've, our drawing that we did for the hole, illustrates adjacent beams and such like.
Q. So would we put it this way that you don't recall the drawings but you infer from the drawings your firm did, that they must have been based on sighting the original structural plans?
A. I, I can't recall but what we frequently did was go to the Council, check property files to get information that we needed for the job.
Q. In principle to do the calculations needed for this you would have also wished to sight the original designer's calculations wouldn't you?
A. No.
Q. No?
A. No.
Q. You don't see that as necessary?
A. No.
Q. Who would've gone to the Council, yourself or your employee Chris?
A. It probably was me.
Q. And your firm at that time comprised how many people?
A. Oh, there might've been four, five, something like that.
Q. And the short-term employee called Chris, the work designation and the qualifications of that person?
A. He was a Bachelor of Engineering I believe but, I mean I can only remember he's Chris, that he was Chris at this stage.

Q. You're not assisted by the initials on the plan from that (inaudible 14:36:25)

A. Well yeah no I couldn't, I wasn't, I had to be prompted by my draughtsman actually to, with his name.

Q. Now is it correct to understand, paragraph 8 of your evidence, on the basis that the work was simply given to this person and this person did that and then you signed off the drawings as having checked them?

A. I do that but I also take a personal interest in what is done. I mean it's a small firm, that's my role.

Q. But in terms of the origination of the design, your paragraph 8 could be read as reading that this was originated by Chris. Is that a correct interpretation of it?

A. I would say that is a correct interpretation.

1437

Q. Now you've indicated that your records aren't available now and you refer in paragraph 6 to your office having been in the St Elmo Court's building. Can you see that?

A. Yes I do, yeah.

Q. Was it not the case that this building became unusable after the September earthquake?

A. Sorry.

Q. Was it not the case that that building became unusable after the September earthquake?

A. No, no.

Q. So are you saying you practised in that building up to the 22nd of February?

A. No, we had to get out of it, I don't know or remember exactly when it was but it might have been the beginning of the following year, yeah.

Q. Prior to the 22 February earthquake?

A. Yes, because 22nd of February we were in 79 Cambridge Terrace.

Q. So can you account for having moved from St Elmo to Cambridge Terrace without taking your records with you?

A. I can, yeah, well if we did – the records that we've got in – presumably still in 79 Cambridge Terrace, are not – don't include this particular job as far as I recall.
Q. Well just taking that in stages are you saying that when you left St Elmo Courts you took all your records with you?
A. We didn't take all of our records with us.
Q. What happened to the balance?
A. Well they went down with the building.
Q. And was there any selection applied as to what was left behind and what was taken?
A. We took more recent files, that could be checked, but I don't have a record of it.
Q. And are you now saying that you have in your current Cambridge Terrace premises files which came from St Elmo but which you do not believe include the files relevant to this job?
A. I believe that to be the case.
Q. And have you personally looked to see whether that is the case?
A. I haven't been to 79 Cambridge Terrace recently to determine that.
Q. So in terms of the preparation of your brief were you asked to locate the files in respect of this piece of work?
A. I wasn't asked no.
Q. Does it remain possible therefore that you may in fact still have the files for this piece of work?
A. That could be possible but I doubt it, I would - if required I could check it.
Q. Now in terms of the inspection of the works where you've indicated you recall going to the building but not greater detail than that, just talking about your standard practice as opposed to this particular job would you have been likely to go once or more than once?
A. Probably more than once.
Q. And in relation to the opening up of the slab which you understand is the point of interest –
A. Yes.

Q. – we're not concerned about the quite separate part of the job relating to the new entranceway.
A. Yes.
Q. In relation to that would you have defined the size of the opening to be made in the slab or was that given to you by the architect?
A. Well we determined it from the architect's drawings, I would assume. It's all nicely dimensioned on our drawing that's for sure.
Q. You see one can draw a distinction between cutting the hole which can safely be cut in the slab and the architect has to do their best and cutting the hole that the architect wants cut for their design purposes, you see the two?
A. Yes I do.
Q. Are you able to say which occurred here?
A. I don't think there would be a variation between what was required by the architect and what we actually did.
Q. So when you went on the inspection on each of perhaps the two occasions which we've identified you are likely to have gone, speaking of your general practice what would you have been looking for?
A. I would be looking at – well I do recall looking at a pad foundation for a post which was part of the situation and I'd be looking at probably connections for the beams to the concrete.
Q. Would you in particular be looking to see whether the as found conditions on site matched the anticipated conditions in relation to reinforcing and mesh?
A. Yes, I guess that would be the intention.
Q. Do you have any recollection of making that kind of check in relation to this slab?
A. No I do not have any recollection.
Q. Now you've indicated that the – in your assessment, the rest of – this is paragraph 14 of your brief, in your assessment the rest of the floor diaphragm was available to transfer lateral loads to the shear walls. Do you recall saying that?
A. Yeah I do.

Q. Is that an assessment that you made from your reinforced concrete experience or is that something that you have separately calculated?
A. I certainly didn't calculate it. I would put it down to experience.
Q. And with that associated experience did that also define the method which you adopted in relation to the use of trimmer beams to transfer the load?
A. Sorry.
Q. Did you also rely on your experience to use the method which you adopted trimmer beams to transfer the load?
A. Which – what sort of loads are we talking about here?
Q. The transfer of the slab loading to take account of the hole which has been cut in it?
A. That's right, yeah.
Q. And that would be based on your years of experience in reinforced concrete buildings rather than any calculations which you did?
A. Well we would have – we did calculations for the trimmer beams and their connections so ...
Q. And are these calculations that you did or that Chris did or would someone else have done them?
A. But it would have been Chris presumably, yeah.
Q. Well you don't recall?
A. I don't recall doing the sums myself.
Q. No. If these calculations had been done and if your job file could be found would you expect them to be on it?
A. I would, yes I would.
Q. Does that identify to you the importance of having looked for the job file?
A. It does.

RE-EXAMINATION: MR ZARIFEH

Q. Mr Falloon, to try and locate that file if you can.
A. Yes.

QUESTIONS FROM COMMISSIONERS FENWICK AND CARTER - NIL

QUESTIONS FROM JUSTICE COOPER:

Q. You've told us that your employee did this design and you signed the drawings as having checked them. Was that a common way in which plans were issued from your office?
A. Yes, yes.
Q. So what was your role?
A. Well I was the principal of the firm and I would be doing detailed design as well on other jobs.
Q. Yes.
A. And I'd be watching, you know, employees with what they did and checking the ...
Q. Well why did you check their work?
A. Well it's what we always do, I mean it's a daily event. You'd be keeping an eye on work that was done and finally when the drawing's complete I would sign it as checked.
Q. And can you remember how much experience this fellow had, Chris?
A. He was a, I believe a fresh graduate.

QUESTIONS ARISING - ALL COUNSEL – NIL

WITNESS EXCUSED

MR ELLIOTT:

PANEL DISCUSSION CONTINUES

1447

Powerpoint presentation from Compusoft

JUSTICE COOPER:

Now I just need to acknowledge the affirmations that you made earlier. They still apply all right? Now I'll just ask whether in relation to this evidence to date whether there are any questions counsel have that they'd like to pose Mr Reid? Mr Rennie?

MR RENNIE:

No Sir.

JUSTICE COOPER:

Mr Clay I see you're here but I –

MR CLAY:

Nothing Sir.

JUSTICE COOPER:

Well then Mr Elliott.

MR ELLIOTT:

Your Honour I think we've exhausted the discussion on the –

JUSTICE COOPER:

Yes.

MR ELLIOTT:

Areas of disagreement identified about the NLTHA input.

JUSTICE COOPER:
Yes.

MR ELLIOTT:
So I think we could move on now to the presentation by Compusoft of their further report. They've been asked to prepare just a brief presentation which would address the key areas of difference between the more recent report and the earlier one.

JUSTICE COOPER:
Yes.

MR ELLIOTT:
And comment upon the key features of the building's performance as identified in the more recent report.

JUSTICE COOPER:
Yes.

MR ELLIOTT:
So they'll do that now and then answer any questions the Commissioners have about their reports.

JUSTICE COOPER:
All right. Thank you. So who is going to take us through this?

MR BRADLEY:
I will.

JUSTICE COOPER:
All right. Thank you Mr Bradley.

MR ELLIOTT:

I prepared that Your Honour so –

JUSTICE COOPER:

I must have been talking to you Mr Bradley.

MR BRADLEY:

This is just a brief summary of the two analyses that we did to aid in the investigations to the collapse of the CTV building. We essentially did two non-linear time history analyses.

The original analysis was undertaken for Structure Smith and Hyland Consulting on behalf of the Department of Building and Housing and we will refer to this as the DBH non-linear time history analysis and the second one was a refined analysis undertaken the Canterbury Earthquakes Royal Commission which we'll identify as the refined non-linear time history analysis. Both of these analyses were determined in consultation with several and different experts in the field of concrete and seismic engineering.

I believe Marcus has already given a brief description of what a non-linear time history analysis is. I'll sort of elaborate a couple of points. Essentially it's a computer simulation which aims to mimic the response of a structure when subject to ground, earthquake ground motion. It can consider the degradation of strength and stiffness of structural components and the corresponding redistribution of seismic action resulting from this, and when we do a non-linear time history we consider the expected performance of the building and by that we do the expected material properties at the time of the earthquake and expected soil properties and we also try and replicate as best as we can that the likely performance of the detailing that is present ie you know how well confined the columns are and the force displacement relationships of all the structural elements.

Now a non-linear time history analysis is considered to be a better predictor of the seismic demands than can be determined by linear analysis techniques.

Firstly I thought I'd explain the analysis philosophy we used for the DBH non-linear time history analysis. It was Compusoft's role to determine the most likely response of the building and produce a set of results that would enable multiple collapse scenarios to be examined and failure hierarchies to be determined. To facilitate this the approach taken was to ensure that the computer model could produce a result at time steps beyond which local element capacities could have been exceeded. By that we allowed the model to produce results and then it would allow these failure sequences to be determined and failure hierarchies. We were concerned that if you'd stopped, if the analysis had stopped at the first instability there wouldn't be information available for those who are required to predict what happens and what order. Now the critical elements for which this was applicable included the column hinges, the beam column joints and diaphragm connections excluding the drag bars, and I'll explain a little bit about these right now.

Now for the column hinges we elected to use interacting moment/moment hinges. These were adopted because we were concerned that the use of interacting axial moment hinges could lead to numerical instabilities and would not produce the desired level of information if the analysis stopped running at the point of the first instability.

The beam column joints we were concerned, we didn't, sorry we didn't model the strength or stiffness degradation and the reason we didn't do that is because as Athol discussed at length we were concerned about there wasn't an appropriate level of test data to quantify the performance of the joints and also the absence of a recognised analysis methodology.

Now diaphragm connections. We didn't allow the total diaphragm disconnection from the north core to happen and the reason we did this is because there are multiple failure planes in around the north core depending of what reinforcement occurs where. And also there are a number of unknowns as to the contribution of the metal decking in the floor to the strength of the diaphragm, so we deemed it appropriate to just produce a set of demands and then people could go through and determine at each failure plane what they thought the capacity was when the capacity was exceeded.

Now this analysis philosophy is suitable for capturing the global building behaviour and in performance of infrastructure but the performance of local elements isn't readily available for interrogation and it does rely on some results to be post processed and assessed later on.

The analyses we undertook. We did the Darfield and Lyttelton analyses were done separately, ie they weren't sequential. We only did one ground motion record for the Darfield analysis and that was the CGBS record and we undertook that because it seemed from the three records that were recommended as being suitable it seemed to be by and large be about average, at the average response in the period ranges we were interested in. The three stations, we used three stations for the Lyttelton earthquake which is the February one and that was CCCC (Cathedral College), CBGS (Botanical Gardens) and the Christchurch Hospital (CHHC). What we did then is we undertook a non-linear pushover analysis to troubleshoot the model and get a feel for the performance of the building and what we've got presented on the left here is the purple and green lines here are the east west pushover period. That's the force displacement relationship which is up and down the page, and the blue and red in a north and south which is across the page. Now as you can see from this that the east-west is response – it's considerably stiffer than the north-south and you can also see here that there is a considerable of non-linearity exhibited both north-south and east-west.

These two plots here show the contribution of each of the components to the overall building behaviour, the blue line being the overall force displacement relationship for the building, the red line being the north core, the purple line being the south wall and the green line being the contribution of the frames. Now this one on the left is for east-west response and you can see here that the north core is stiffer than the south wall and the contribution of the columns is quite a lot less than both of those, particularly as small displacements. Now in this north-south direction the same thing's exhibited except for the north core is seen to dominate the response again with the frames contributing maybe 10 to 15 percent on the overall building performance.

A lot's been said about some of the results, particularly by Professor Mander regarding the validity of some of the results that we produced in our previous analysis, and in particular he was concerned about the plot that was included in the Hyland Smith report and of – this is an adaption of that plot from that

report. Now I'd like to state that the hinges that we used, the interacting moment/moment hinges that we used, the (inaudible) hardening model and does exhibit some isotropic hardening and by that it means that when you go through inelastic cycling the capacity of the hinge can actually grow a little bit. We looked at this and the effect in this building is actually very small and for this particular plot here the relevant interaction surface is this outer one which is in red on the outside here and we've plotted, the adaption we've made to this is we actually plotted the demands as a function of time. Now the analysis that we did, the initial analysis for DBH we did, showed that or indicated that this column would perhaps fail or could fail just before five seconds. Now as you can see here the orange dots here are the demands up to five seconds. So you can see here that there's – they're only slightly outside the interaction service perhaps at this end and the reason that there's a few other dots out here is because as we've stated previously we allowed the analysis to continue past at which the point of its failure was predicted so that more information was available for those predicting failure sequences. The fact that the columns only contribute in the order of 10 to 15 percent of the global building response means that the effects of this overall building is probably less than a percent so we consider this to be – show that the analysis actually is valid and is suitable for the purpose in which it was used.
Since then we've undertaken an additional non-linear time history analysis for the Royal Commission and basically we made some refinements to the initial model and this was to examine the effects of local element behaviour. It wasn't explicitly modelled in the previous analysis.
Changes that we made to this analysis model included the incorporation of moment axial load interaction into the column hinges. We included non-linear floor elements adjacent to the beam lines and that was to more accurately model the response of the diaphragms and we've also slightly modified the damping parameters for the February analysis runs and the reason we did this is we believe we – the damping that we had for the vertical modes in the initial analysis was perhaps a little bit on the low side so we've set that to be approximately 2 percent for the vertical modes in the new analysis. We've also included the non-linear beam column joint strengthen stiffness

degradation in the new model. We've made some minor revisions to the slab stiffness. We've also changed as Professor Carr has mentioned the concrete strength and the corresponding stiffness modifiers to one and a half times the concrete strength that was specified. We've also done, undertaken some Lyttelton earthquake, that's the February earthquake analyses and we've undertaken these considering the damage state present at the end of the Darfield earthquake and we've included the additional ground motion record, REHS which is a Resthaven record for the Lyttelton event.

JUSTICE COOPER:

What was the damage state present at the end of the Darfield earthquake, September earthquake that you used.

MR BRADLEY

What we do is we basically, we ran the significant portion of the Darfield earthquake and any stiffness degradation that the model picks up during that and is present at the end of that analysis whether it be some walls are cracked, some columns have perhaps reached their peak or undergone some inelastic behaviour, whether the diaphragm drag bars are disconnected, that is if we continue from that and then undertake the February earthquake record based on the damage state from that, and I'll explain –

JUSTICE COOPER:

Is the damage state predicted by the model?

MR BRADLEY:

Correct, sorry, yes it's the damage state predicted by the model. Here we go, basically the specific analyses we undertook were we did a Darfield and Lyttelton sequential analysis for the two recording stations, CGBS and CCCC, and then we did the three records we'd done previously for Lyttelton, but, and in addition to that we did the REHS and these were only for the Lyttelton only ie that assumes an undamaged building at the start of the earthquake motions.

I'll briefly go through the key analysis data. This is a summary of the main – the behaviour types that we will talk about and that's the building displacement demands, drag bar behaviour, critical column actions, beam column joint behaviour, anchorage of the beam column joint longitudinal reinforcement, the effects of vertical accelerations and sequential earthquake effects.

What we've got here is the building displacement response to the CGBS record. What we see in this portion here was the Darfield or September event and in the blue is – this is the displacements on the level 6 which is the top floor plate, just underneath the roof, red is the east-west response or displacement response and blue is the north-south, so you can see here that at the roof in the corners we were getting up to about 160 mm and almost up to 200 in the other direction and while the east-west was in the order of I don't know, 80 mm, 80/90 mm whereas if you can see here compare this and this is the Lyttelton event right here you can see that demands for both directions are in the order of two to two and a half times that seen for the Darfield event and this pretty much indicative of all records. They all show similar trends.

Now this is a displacement response comparison between the original DBH analysis on the left-hand side here and the revised analysis on the right. Now you can see here that both records essentially they show some slight variations but for qualitative purposes the displacements are pretty much the same. What's interesting to note is, and actually all records exhibit this, is they show some moderate excitation and then roughly a few seconds into the record there seems to be – there's a large increase in the displacement demands and they – all records, this is the north-east corner, sorry the north-west corner, all of the records are characterised by a large displacement demand and also at the same time, corresponding time there's also a large increase in the east-west displacement as it's seen down here.

The drag bars. The table on the upper left is the drag bar, whether the drag bar's connected or not for the Darfield event for the CGBS record and the DBH analysis and you can see here that the DBH analysis predicted that drag bars would disconnect on wall DE, that's the easternmost wall of the north core at levels 4 and 5. Now on the revised analysis or enhanced analysis there is no disconnections predicted for the drag bar strengths that we used. Now the bottom two tables are the sequential analysis results for the drag bars. Now on the left, the CGBS, on the right is CCC. As you can see here the four, the undamaged, analysis assuming that the undamaged state. The drag-bar times are identical to the drag-bar disconnection times, assuming that there was some damage at the start of the Lyttelton earthquake. Now this is not the same as was predicted for the CCC record. You can see here at the end of the Darfield event the CCC drag-bars were predicted to have disconnected at level 4 for walls D and DE and then, as you can see, the disconnection times for the remaining drag bars were slightly different particularly on wall D. There's a second or two difference here.

What's presented in this slide is a selection of column actions and what we've got here on the left-hand side is what the analysis showed to be the worst case column under the Darfield CBGS record and the top plot which is up here presents the rotations, the hinge rotations, this is column F3, at level 4, the hinge rotations and also the axial elongation of the hinge. So as the thing rotates it's sort of growing a little bit, not a lot. It's the scale and on the bottom is the strain, we've plotted here the strain in the concrete at the extreme compression fibre of the hinge, and what's interesting to note the blue line here is the compression side and the red line is the tension side. So you can see here that for this particular hinge the maximum concrete strain reached was around .004 and the ultimate strain that we believe that this material could have taken was in the order of .00825. Roughly half of what it's ultimate strain would have been. Now this is predicted that there would have been some yielding of reinforcement. The yield strain is around about where my line is, cursor is right now so there would have been some inelastic behaviour but not enough to cause spalling.

Now the plots on the right are for the February event and this is for one of the critical columns that we examined which is the heaviest loaded column, or one of the heaviest loaded columns in the CTV building which is column C2 at the base. What we can see here is essentially the rotations are relatively small and we get up to around 6.3 seconds and it's when we reach that large increase in displacement demands the rotations increase dramatically and shortly after these rotations occur you can see the axial elongation of the hinge reaches a point and starts to compress and as that compresses it's basically showing that that hinge is failing in compression. And below that you can see we've plotted the strains, again the concrete strains at the extreme fibre and you can see here that again it goes along. It's well below the ultimate concrete strain until it reaches around 6.3 seconds. It goes past that point and then shortly after that you can see axial load carrying capacity is lost and the strains drop, displacement drops.

Here is a summary of the beam column joint information that was produced for the CGBS record for Lyttelton. I haven't shown the one-way joints that occur on gridlines 1, 2, 3 and 4 because essentially the analysis predicted there, the response to the elastic with no degradation of the stiffness or strength for those joints. What we've got here, firstly I'd like to explain the colour-coding. The colour-coding here it's difficult to see but there's light blue which is that colour there, essentially shows that there's elastic behaviour of the joints. So the joints would be predicted to behave in a linear manner and in the red next to it indicates that there is some slight degradation in stiffness but the joint is yet to reach its ultimate capacity and the green colour-coding shows that there's basically a yield plateau or a strength plateau and when you get into the purple that shows a degradation in both strength and stiffness for the joint and light blue means a considerable degradation in stiffness and strength. Now as we can see here on the left-hand side presented is the gridline A, two-way joints, and the top one is –

JUSTICE COOPER:

Q. How do we know that, looking at it?

A. I haven't put labels on unfortunately. I can assure you though that that top one is gridline A and it shows rotation –

Q. I've had the same feeling with a number of these slides that it's going to be difficult for us when we reflect on this to understand what we're looking at, difficult for me anyway.

A. I'll be happy to go back and put titles on for you.

Q. Is this material extracted from –

A. Yes it is.

Q. It's not a further distillation of anything. It's just a re-presentation of material that's in the main report?

A. It is, yes.

Q. That's helpful. Now take me to the page in the latest report where this appeared.

A. It's in the second draft report that you received this morning. We'll just find the page. The plot that is exhibited in the top left-hand corner is Figure 67.

Q. So that's at our page suffix.

A. Page 86

Q. It's our page 105, 552.105. There's no explanation there of the colour code.

A. Sorry, that is included on page, the preceding page, page 84 and it's Figure 47 in that report.

Q. Just help me with this. Figure 47 defines the colour coding?

A. Yes.

Q. Providing you know what you're looking at. Is that right?

A. Yeah basically what it shows is the colour coding for a certain amount of rotations, different rotation and states.

Q. So when you say, "Note the blue colour represents elastic behaviour" that's the darker blue?

A. That's correct. The bottom left plot is Figure 71 and the top right is Figure 69.

Q. Just tell me, Figure 69, GL what does that stand for?

A. That's gridline.

Q. Gridline F?

A. Yes and what that plot shows is the total number of beam column joints on gridline F for which there are 20. It shows how many hinges cumulatively throughout the course of the record start to exhibit x amount of behaviour as shown on that plot. So what it shows here at about four seconds there's one hinge that is showing a slight degradation in stiffness and then after about six and a half seconds we get another about four or five and it continues from there.

1517

JUSTICE COOPER:
Now the fact that there are 20 of these. Is that recorded somewhere else in your report or?

MR BRADLEY:
No, it is –

JUSTICE COOPER:
One has to know that?

MR BRADLEY:
It's, basically there's five levels and there's four joints at each level. So there's a total of 20 joints.

JUSTICE COOPER:
And is that recorded somewhere or?

MR BRADLEY:
No it's not, I can add a note to that effect. Essentially there's, there's 20 joints on gridline F, there is 20 joints on gridline A and there's 60 remaining joints for the rest of the structure. So what the plot shows, if you can see on the upper left-hand plot here, for east-west translation, a rotation about the north-south axis, we can see that there is no distress in the beam column joints until slightly after four seconds where four joints are shown to crack slightly and degrade and have a slight change in stiffness and then it's about six seconds into the record they, three of those joints start to reach their ultimate capacity

and they're basically moving on this yield plateau and then after about seven and a half seconds you can see three joints start to degrade in stiffness and strength. And in four the rotation about the east-west axis, so that's for a north-south translation, less joints are seen to degrade in total but the degradation occurs more sharply. So there's one joint that shows degradation almost straight away after it reaches its yield plateau.

JUSTICE COOPER:
So how do we know that that's east-west?

MR BRADLEY:
There is a, there is a key in the report.

JUSTICE COOPER
Where's that?

MR BRADLEY:
It's on page 85 of that report. Just above photo 66. And what's important to note in these plots is that there is, the joints for the CGBS record and using the joint model that we have in the analysis model the joints behave essentially elastically on gridline A until you reach about four, four and a half seconds, and then it's after six seconds the joints are shown to reach their ultimate capacity. They aren't degrading in stiffness. Now this actually occurs if you go back to our previous one, after the joint, the column hinge is shown to exceed its ultimate limit stake capacity or the ultimate strain. So this is indicating that the column hinge at the base of the structure would be seen to lose capacity prior to the beam column joints losing any of their strength. On the right-hand side we can see for gridline F there is no degradation in strength exhibited throughout the whole record. Now with the beam column joint, the anchorage of the beam bars into the beam column joints on, were their hooks which didn't have a lot of embedment and there's been concern that under bending actions these hooks could pull out and influence the behaviour of the structure. The capacity of these, these hooks is dependent

on the axial load in the column. So what we've done is we've post processed the results to indicate where we believe joint or bar pull out would be a possibility. And as you can see on this upper graph there is a red line, and that's the demand capacity over one. So everything above that red line shows that there is the possibility that the bars or the beams could've pulled out, and these, for this particular record which I think is the north-south translation, there is a few bars on at four seconds that are shown to slightly exceed demand, and then again later at around six seconds where that big spike in demand, displacement demands are shown. Now these, the columns that exhibit this behaviour are essentially the lightly loaded corner columns on the whole, although a couple on gridline A are also shown that they could be susceptible to pull out. And for east-west translation of the building, a fewer number of bars are shown to have the potential for pull out, and this again occurs at around the six second mark for this record.

The effects of vertical acceleration. We, as discussed previously we did some sensitivity analysis for the DBH analysis but we haven't had a chance to do this for the revised analysis, the refined analysis. But what I've presented here, just to give an idea of the maximum variance in axial load that is exhibited for column C2 through, for the CGBS record on the left and the CCC record on the right. Now you can see on the CGBS record that there's not an insubstantial variance in axial load, but this is much larger in the CCC record. There's a much larger variance in axial load and it actually goes up to quite high compressive forces throughout the course, during the earthquake. Below these plot, is a plot of the demands for that hinge or for that column and the red and blue lines are the moment demands in each direction, and the green line is the variance in axial load as occurrence and both plotted as a function of time. What's interesting to note is that the frequency of the vertical actions is a lot higher than it is for the rotations or the moment demands which are dominated by flexure or sway in the building. Now the peaks, the peaks in the, the peak axial loads don't necessarily correspond with the peak moment demands in those columns. You can see here that they seem to get, they close on occasion but they don't always correspond, coincide sorry.

This here shows the effects of sequential analysis and what I've shown here is this, for the CGBS record only and you can see here that on the, the one on the left is the displacement at the north-west corner for the CGBS record, assuming that there was no degradation in strength at the start of the record, i.e. the undamaged state. And the one on the right shows the results for when we continue the analysis sequentially. And you can see here that there are some minor differences in displacement, but it's not a significant amount, and the same for this is the south-east corner and we can see here that by and large the displacement demands are the same. And as we discussed previously the drag bar disconnection times are identical for this record. Now for the CG – CCC record the drag bar disconnection times were slightly different and the performance is slightly different but the important thing to note for the CCC record is that it predicts under Darfield that a significant, a few columns do undergo significant inelastic demand and were at risk of spalling. They certainly have shown to exceed their ultimate compression strain.

JUSTICE COOPER:

This will no doubt betray my lack of understanding but why is the displacement in the north-south direction less in the sequential situation than it is in the diagram on the left-hand side?

MR BRADLEY:

Some of it is to do with there's a residual displacement after the Darfield event, so we're starting from a little bit back. We're not starting from zero so the displacement as a relative term is essentially the same because you can see here that it's starting slightly below the zero line, and previously it was. So that accounts for a little bit.

JUSTICE COOPER:

Not all of it though.

MR BRADLEY:

But not all of it no, but we would not consider this to be a substantial change in response.

JUSTICE COOPER:
But it's in the wrong direction, one feels intuitively.

MR BRADLEY:
The wrong direction Sir?

JUSTICE COOPER:
Well I mean if there was a difference you'd expect the sequential displacement to be greater wouldn't you?

MR BRADLEY:
Not necessarily because what can happen is the building can soften up a little bit and it actually changes the response to the earthquake so you can get a slightly different response and that can manifest itself in either larger or smaller displacements because the building is not quite the same.

JUSTICE COOPER:
Dr Bradley?

DR BRADLEY:
Yeah I was just going to make the same comment. You'll see just before the large displacement up till approximately 0.4 metres in the negative direction the displacement is larger in the case of the sequential analysis so it almost reaches negative .2 and the effect of the large displacement in the opposite direction is what -
1527

JUSTICE COOPER:
Amplifies it all right thank you.

MR BRADLEY:

This is a brief summary of the results.

From the analyses that were processed to date we can see there is no significant change in the global building response as a consequence of the refinements made to the Department of Building and Housing non-linear time history analysis.

The analyses show that failure could or would have occurred with or without any pre-existing damage that may have occurred prior to the Lyttelton earthquake.

The analyses indicate that a number of column hinges would have experienced significant inelastic demands when subject to the Darfield CCC event and they would have exceeded some performance measures during that event.

For sequential analyses we can see that for the CBGS record sequential analyses showed no significant influence on the building displacement demands, diaphragm forces, column hinge rotations and beam column joint performance.

Sequential analysis had no effect on drag bar disconnection times.

Now for the CTC record it's difficult to infer what the effects of sequential analysis are for the Darfield event as there was a potential for column failure during the, through spalling prior to the Lyttelton event.

Now the columns, now depending on the failure criteria adopted and there were two schools of thought in the expert panel and one was the failure criteria being when the actual low capacity of the entire hinge is lost, and the other one is where the compressed strain on the other fibre of the column or nearabouts exceeds the ultimate compression strain therefore leading to spalling and potential failure. So depending on which failure criteria you adopt there's a potential for column failure through excessive flexural demands which primarily occur on gridline F or excessive axial and flexural demands which occurs primarily on the heavily loaded interior columns such as column C2.

JUSTICE COOPER:

When you talk about column C2 can I just understand have you numbered each column?

MR BRADLEY:

That's a grid reference.

JUSTICE COOPER:

That's the grid reference right.

MR BRADLEY:

I think I've actually cross referenced the ones on the drawing as well.

JUSTICE COOPER:

No I just need to understand what your methodology was. So that's the grid reference. Gridline C2.

MR BRADLEY:

Yes. And for the beam column joints the analysis shows the column failure would occur prior to degradation in beam column joint strength although given the uncertainty and the performance of the detailing present a beam column joint initiation of failure cannot be discounted. For vertical earthquake effects work is ongoing for the latest analysis although the initial results indicate that vertical earthquake effects would have influenced the performance of the structure however we would anticipate that the building would be unable to sustain the lateral displacement demands even if vertical earthquake components were excluded.

HEARING ADJOURNS: 3.30 PM

HEARING RESUMES: 3.46 PM

MR ELLIOTT:

Commissioners before we continue can I just indicate the presence of some guests here. You will be aware that a number of those who died in the building were from the Toyama Language School. We just have with us this afternoon firstly the Vice Mayor of the City of Toyama, Kunio Oizuki; we have Houiti Sasaki a member of the Toyama City Assembly, Mamoru Arisawa a member of the City Assembly as well; also Professor Tadashi Jimbo from the Toyama College of Foreign Languages and Ken Katayama the Assistant Director Secretary Division, City of Toyama.

JUSTICE COOPER:

Gentlemen, you are most welcome here to our proceedings and we are honoured by your presence.

COMMISSIONER FENWICK:

There are a number of points I'm a little curious about and I'm sorry you gave me, or I had a draft before and the new draft arrived on my desk this morning which didn't allow me time to go through it so I have to reference everything back to the previous draft. On page 38, sorry .38 the BUI.MAD249.0535.38 you have table 7 which gives an expression for the shear stresses in the beam column joints. I hope I've got those references correct. Now when we're doing a standard building we calculate the horizontal shear stress from the beams. So we look at the steel and the beams and we calculate the column shear and we produce the difference between the two. In this case of course we've got the columns which are yielding, not the beams. So when you've got BJH there was that the vertical shear stress or did you somehow tweak it round to give you the horizontal shear stress, as normally we know we calculate the horizontal shear stress and we take an approximation for the vertical.

MR DAVIDSON:

Different people may answer.

MR STUART:

The stress that's governs the performance of the joint is based on the horizontal joint shear stress and that has been back – it's worked out on a basis of a relationship of the beam moments themselves, like you're saying that the column moments are either the driver, but but we've gone down to a simple assemblage and driver relationship based on certain moment profile.

COMMISSIONER FENWICK:
So you put the plastic column moments in and calculated the beam moments from that and then calculate the shear stress.

MR STUART:
We've put in – a moment shall we say and worked out what the maximum moment we could get through a joint was and calibrated that back to – or based it sorry, we found the maximum moment of the joint could transfer between the beams and the columns on the basis of a limiting horizontal joint shear stress value for a given moment profile or (inaudible 15:51:17) profile which is presented in one of the appendices.

MR DAVIDSON:
Might be C I think, there's some calculations there as well.

COMMISSIONER FENWICK:
Okay, you've got an appendix, that's fine, I can find it.

MR STUART:
So appendix C does outline the approach taken to derive the, ultimately the bending moment that we do transfer at the joint zone but as I said, it is – the limitation as I said is the horizontal joint shear stress.

MR DAVIDSON:
On page 115 of our first draft report if you like is the beginning of the calculations.

COMMISSIONER FENWICK:

Okay, I can look that up thank you. Now the column capacities, displacement capacities. How are they calculated?

MR DAVIDSON:

Sorry –

COMMISSIONER FENWICK:

When you were calculating the limiting capacity of the column, limiting displacement capacity of the column given its axial load on it in the programme, how was that displacement limit calculated?

MR STUART:

The column hinge model is a simplified fibre based model so it does have multiple concrete and steel elements within that fibre, sorry multiple concrete and steel fibres within the hinge and the axial moment interaction relationship and hysteretic form is governed purely on that geometry and the (inaudible 15:53:03) of that element. Now there are a couple of things that are missing from that because of the difficulties of it which is the potential for bar buckling is not considered and because – with large spacings between the spirals, there is potential that – there is a couple in effect in terms of once the outer fibre goes it could propagate rapidly through the centre of the core as well so it's an idealised – an idealised plane sections relationship I guess you could say with no real, yeah, because they're an ideal behaviour for every individual fibre.

COMMISSIONER FENWICK:

You have a model plastic hinge length?

MR STUART:

We have taken that to be 200 millimetres I'm pretty sure.

MR BRADLEY:

That's correct, yeah.

COMMISSIONER FENWICK:

How was that calculated?

MR STUART:

So that's half the column dimension and that was valid from quite early on I think.

MR BRADLEY:

The 200 millimetres was promoted by Professor Mander.

COMMISSIONER FENWICK:

And this involves a yield extension into the column and a plastic extension outside the column.

MR BRADLEY:

My understanding it did consider strain penetration into the joint and X amount outside of it.

COMMISSIONER FENWICK:

So how do you then calculate your rotation from that? You're assuming plane sections remain plane over that plastic?

MR STUART:

Yes, we are taking the full 200 millimetres as being a joint zone shall we say and the rotation is between those two concrete columns.

COMMISSIONER FENWICK:

So you can have your concrete strain, limiting concrete strain might have this over that full 200 millimetres?

MR STUART:

Yes.

COMMISSIONER FENWICK:

So in fact it's not an analytical model. It can't be because you can't have that strain in your yield penetration. It's surrounded by concrete?

MR STUART:

Yes that's true, yes.

COMMISSIONER FENWICK:

So it's not an analytical model it's just an empirical model (overtalking 15:55:03) achieve experimental results?

MR STUART:

Yes, it's more of a sectional analysis I guess. I'm not sure where the strain penetration I can't quite recall where that was brought into it because initially, there the first report did have a bigger length didn't it?

MR BRADLEY:

I think we did, yes.

MR STUART:

So I'm not sure where along the lines it went down from I think it was about 290 millimetres down to 200.

MR STUART:

This is where it was just losing that stream of penetration.

COMMISSIONER FENWICK:

This model came from Professor Mander?

MR STUART:

The hinge length.

COMMISSIONER FENWICK:
So perhaps I should question him about it rather than you?

MR STUART:
Yes.

MR BRADLEY:
The model, the hinge length came from Professor Mander.

JUSTICE COOPER:
Can you throw any light on this Dr Bradley?

DR BRADLEY:
Not in its entirety but my understanding was because of Professor Mander's argument was because of the lap splice at the base of the columns he felt it was smaller, but I'm happy to be corrected by him at a future point in time.

COMMISSIONER FENWICK:
So the lap splice is above the beam column joint but not below it and of course the ground floor there's no lap splice presumably.

UNKNOWN:
There is at the ground floor as well.

COMMISSIONER FENWICK:
There is at the ground floor is there? So this means that the plastic hinge is more into the strain penetration into the design?

DR BRADLEY:
At that point you would be going beyond my expertise Sir.

COMMISSIONER FENWICK:

I mean the question I've got is I think it's an empirical model, I think it must be and were there any columns tested which is like spiral reinforcement to develop that model. If the case we've got a question of whether it's valid, so I mean clearly you can't people can't answer it and that's a question that should be put to Professor Mander.

155600

MR STUART:

The main focus I guess is as pointed out previously the displacement demands for the building are governed, well almost entirely by the wall structure. The frames do not really contribute to that to a great extent. The – so the key point was to make sure that it would yield at a suitable value and incorporate the PM interaction, the actual hysteretic response given the low level of displacement capacity detailing would sustain, it was ultimately a secondary consideration I guess.

COMMISSIONER FENWICK:

Are you saying the columns couldn’t have failed?

MR STUART:

No they can fail but the model will indicate failure at a much greater displacement I think than would be likely, so we had to alleviate with promoted three potential failure scenarios which would be or indicators anyway which is when the column loses actual capacity and then needs to redistribute load to other columns. It’s essentially a watershed one I guess. Nothing can happen beyond then but certainly before that columns can spall earlier than that when the extreme compression fibre does reach the ultimate compression strain capacity of the concrete.

COMMISSIONER FENWICK:

By the plastic hinge theory.

MR STUART:
Sorry.

COMMISSIONER FENWICK:
By the plastic in this theory when it spalls? You're spalling it by assuming there's a plastic hinge there and when the critical strain in that plastic hinge reaches the critical area you say the concrete spalls?

MR STUART:
Yes.

COMMISSIONER FENWICK:
I'm saying that plastic hinge is not a rational model. It can't be because it's assuming plastic strains into the beam column joint. So normally that wouldn't model, it wouldn't matter because one can model it against say it's an empirical way of doing it based on test results. So the question I have is if those test results go down this column with very, very light spiral reinforcements but I think it's a question probably would need to go to Professor Mander.

MR STUART:
Yes.

COMMISSIONER FENWICK:
But thank you for attempting to answer it.
The diaphragm forces. I was at a certain stage trying to look at the interaction of the floor with the north diaphragm particularly the area which I can describe as the toilet area in the floor where you have the north wall, line C, and then I think there was a wall at CD. BUI.MAD249.0486.1.

So you see on line five line C there is the north end, north side there is the toilet area. This is a slab that goes through into the north wall and my concern was, interest was, whether the actions induced on that slab could actually tear the slab. If it had torn the slab would it have torn a certain way and increased the flexibility of the structure? When I look through the report probably going back to previous one now, the forces, the maximum forces you get at the beam line on line four opposite the slab are listed as a bending moment. You can see it must be a moment because the wall, the tear is resisted in the wall on line five and you've also got the it forms a sort of a channel shape and so it's actually the centre of, spectral resistance or shear resistance is somewhere out to the right. So that, you're going to get a high moment in that slab at line four. Simultaneously you've got very high shear forces and simultaneously you'll have some actual tension from the north south direction. Now the problem is in the report of course you just give me the maximum values so there's no way one can confine them from the report to work out what the likely stresses and actions were on the reinforcement in that zone.

MR BRADLEY

What we actually did we were aware that was a critical location so what we did is we presented results. Have you got figure 39 there? First draft to the Commission.

COMMISSONER FENWICK:

39, what page please?

MR BRADLEY:

It is page 52.

COMMISSIONER FENWICK:

Right that lists the maximum values. It doesn't tell me the true –

MR BRADLEY:

No the figure actually shows it's difficult to see but there's actually a dark blue line there or a selection of finite elements and what we've done there is we've taken a section cut whereas we present the information across that interface and we present as a function of time the axial shear and in-plane moment.

JUSTICE COOPER:
Let's just get this displayed, it's 0535.68.

MR BRADLEY:
Yes that's it. As you can see there's some slightly darker shell there in darker blue which is essentially the interface we're talking about and we've taken a, what we were capable of doing is actually taking a section cut along that line and what it does is presents all the axial moment and shear information as a function of time for that interface and whilst we presented a table that you can see just underneath there, table 25 and 6 that's the, that produces the maximum information over a set period of time. We have also in the appendices produced that plot as a function of time so you can actually work out or do you own equation to determine when failure would have occurred. Now for an example I'll show the plots that I refer to. If you go to page 169 of that report.

COMMISSIONER FENWICK:
That's your numbering.

MR BRADLEY:
Yes.

JUSTICE COOPER:
So this should be .185.

MR BRADLEY:
168 actually sorry.

JUSTICE COOPER:
.184.

MR BRADLEY:
Figure H20.

JUSTICE COOPER:
What appendix is this in?

MR BRADLEY:
This is appendix H. Page 168 I refer and also page 169. This is the appendices for the – sorry the other one. You can see there on figure H20 what's presented there is that interface. That's the diaphragm actions in the north-south direction as a function of time for each level. Now if you go to page 169 and see in figures H21 that's the east-west or the shear actions that occur across that interface as a function of time and finally the figure H22 presents the in-plane moment demand that occur at the same time as a function of time for each level. We've presented those so that hand calculations can be made to ascertain when or if that would have failed.

COMMISSIONER FENWICK:
I think I need an electron microscope to line those up. To get the readings.

MR BRADLEY:
I can actually provide you those in a much better format or an alternate format if you like.

COMMISSIONER FENWICK:
I might at some stage come back and ask you for them at a particular time. Just what happened at a particular instance. The other feature if we can go back.

MR BRADLEY:

If you did that you'd be able to tell absolutely.

COMMISSIONER FENWICK:

It's captured in your, you don't have to run another analysis for it do you?

MR BRADLEY:

No it's already there.

1606

COMMISSIONER FENWICK:

Can we go back and have a look at your page 68 which was the first one, there it is on the left-hand side. Now you'll see that the North Core diaphragm, north-east actions, well it doesn't matter, north-east or south-west, the actions, the forces you get are very short period.

MR BRADLEY:

Yes.

COMMISSIONER FENWICK:

Almost instantaneous, I don't know how many hundredths of a second each one lasts but they're very very short.

MR BRADLEY:

Yes.

COMMISSIONER FENWICK TO MR BRADLEY:

Q. Now presumably those forces are very very sensitive to the lateral stiffness of the slab. I mean if you had a crack width there which could grow to half a millimetre those forces might disappear to a quarter of what they are now. What was the stiffness in that direction?

A. For the second analysis what we did is we had elastic portions of diaphragm and we had non-linear sections of diaphragm, and adjacent to every beam line we had an inelastic or non-linear shell to represent the diaphragm and that is capable of actually considering the stiffness changes as a function of the axial load or the stress state of the reinforcement and/or concrete as a function of time so whilst we haven't done the whole, it's also incapable of unzipping, if you like, if the stresses exceed capacity in that location. Whilst we didn't have it over the whole floor, the reasons we didn't do that is because of difficulties in ascertaining what contribution the metal deck had to the diaphragm capacity we just presented it adjacent to the beam lines. I believe it's 750mm wide shell element so that actual did have some springiness and when we did that we saw that there was, from the initial deviation analysis where it was just elastic shells, there was a small difference in diaphragm actions but, you know, it was in the order of, you know, five to 15% and in some instances it actually increased, depending on where you are and at what point in time. There wasn't I wouldn't call a significant change in diaphragm actions and we've actually presented in the second draft report for the CCC record the, sorry, no sorry apologies, you can actually see from if you can compare the plots presented here against the equivalent plots for the DBH analysis you can see that the differences in response over time. What I observed is there was some subtle differences but not major.
Q. This was a smeared concrete and steel sort of model was it?
A. Correct.
Q. You don't identify them separately?
A. Yes they are identified separately.
Q. So you've got separate steel elements and separate concrete elements?
A. Correct, a concrete layer, a steel reinforcement layer and in fact we had I think about three steel layers to represent the mesh and the saddle bars.
Q. And the concrete that cracked?

A. Yes.

Q. So that didn't reduce your forces at all?

A. It did a little bit. It did a little bit. It varied. I mean if I had to make a global statement I would say in the order of five, 10 percent but, as I said, it did vary depending on location and what not.

Q. And you've got inelastic deformation there?

A. We, to be honest, we haven't had chance to post process all the results and go through those. I'm expecting that we did have inelastic demands and in fact from some test models that we did we were seeing inelastic behaviour.

Q. When I looked at your first report and the actions there, you know the shear was very high and then you had the flexure and the axial tension simultaneously it looked to me as that would start to tear but I couldn't identify simultaneous actions.

A. I agree. It is a difficult thing although it does present the things in time it is difficult to actually quantify, you know, exactly what's happening. I can provide you more detailed information. One thing to note is that because we only modelled the reinforcement adjacent or immediately adjacent to the beam line we included the reinforcement which was the saddle bars and the mesh, no contribution from the metal deck but we noted that just outside there, there is a potential failure plane where there is just mesh reinforcement which was not modelled so there still is the potential for that to open up and unzip along that interface.

Q. That's where you'd expect it to go.

A. Well we looked into doing that and the difficulties we had with that was it would have involved some sensitivity analyses doing contributions either an upper bound assuming all the metal decking or none and the true answer would probably be somewhere between –

Q. You can't tell from your analysis where it could have started tearing from that corner because I mean that would then change the stiffness dynamic wouldn't it?

A. Absolutely, yes.

Q. But it probably wouldn't have gone very far because you know once you change the bit you've changed the stiffness and you've changed the input but that wasn't something that you could really follow through with I assume?

A. Not with, I mean, we attempted to model some parts of the behaviour but because we haven't continued across the whole floor we won't see that effect or might not be able to see that.

Q. The pull out of the bars from the beam column joints. You're sort of indicating the higher the axial load the better the bars should be retained?

A. My understanding was the higher the axial load the more likelihood, more difficult to pull out. It was a higher bond.

Q. And where did that sort of come from?

A. That was based upon the, I think it's Appendix H, no sorry it will be D in the latest report. It's Appendix D in the second draft and it was also included in the first report in Appendix B of the DBH Analysis Report it's also included methodology which we used and I believe that was on some research papers which did some testing on deficient beam column joints and their anchorages.

JUSTICE COOPER:

Q. But it's not in the first draft of your latest report.

A. No it's not in the first draft. It's because we instead of, previously in the DBH Report we were presenting results and post processing the performance of the beam column joints and that appendices outlined the methodology which we employed to do that and in the second and for the revised analysis for the Royal Commission we no longer used that because we were explicitly modelling the performance of the joints so it was removed. However, in the second draft because we still used a part of that methodology to asses beam pull out only, it was then put back in as another appendices.

COMMISSIONER FENWICK:

Q. That model you were using did that have the, it wouldn't have had the hooks in the middle of the column though would it. They would have been on the far side or was it was of these early ones where they actually had the hook in the middle?

A. We considered both actually. We did consider the embedment that was available.

JUSTICE COOPER:

Now Mr Smith do you want to ask any questions of these gentleman.

MR SMITH:

No, I fully discussed their analysis prior to this.

MR ELLIOTT ADDRESSES THE COMMISSION

I think at this point we could move on to the issue of interpretation of the more recent Compusoft non-linear time history analysis. So for that purpose can I invite Dr Hyland and Mr Jury to join the panel.

1616

ROBERT DAVID JURY (AFFIRMED)

CLARK WILLIAM KEITH HYLAND (AFFIRMED)

MR ELLIOTT:

Commissioners the process here was that these experts who have so far given some evidence about interpretation of the earlier analysis were asked further questions which can be summarised in two categories. Firstly they were asked what do the results of the second Compusoft report indicate about the response of the building in September and February, and what do they indicate about why the building failed and the sequence of failure. That question resulted in a joint report which is at tab 13(a) in bundle 2, right towards the back. That is a joint report in which the experts were asked to address those questions and to then identify areas of agreement and disagreement. That process of conferring took place and resulted in this joint report and in a moment I'll ask Professor Carr to present the report in the same way that he did before and I just note that the copy I have doesn't appear to be signed so that once that's been presented by Professor Carr each person would need to just – or perhaps there is a signed version available, if not they'll just need to confirm that that is an accurate account of areas of agreement and disagreement. The experts were also asked to provide a separate report to the Royal Commission indicating whether any opinion they've given so far in evidence has been altered in any way by the second Compusoft report and each of the experts has produced a separate report which are quite brief and they appear in the Commissioners' bundles starting at tab C in the same section 13, tab C, D, E and so on. So once Professor Carr has presented the joint report – I don't think it would be necessary for each expert to read each individual report but they could be asked simply to speak to their report, and to summarise their position and I'd also, that Professor Mander has provided a report in the same way. Professor Mander is due to join the panel by video link tomorrow morning at 9.30 so at that time he can address his report and also address the questions such as those from Commissioner Fenwick and I expect Your Honour that then that would evolve into a discussion of further questions from the Commission about areas of disagreement that the experts have had. So I'd invite Professor Carr now to present the joint report.

PROFESSOR CARR:

Joint report in relation to the interpretation of the second Compusoft non-linear time history analysis.

Compusoft has carried out a non-linear time history analysis, the results of which are set out in a report dated the 13th of July 2012, the second Compusoft report. And of course you've just got a slightly later version of that. The counsel assisting the Royal Commission asked the following experts to confer, Dr Clark Hyland, Mr Ashley Smith, Mr Derek Bradley, Dr Barry Davidson and Tony Stuart, Professor John Mander, Dr Brendon Bradley, Professor Nigel Priestley and Mr Rob Jury, and I was asked to facilitate the discussions with the experts and oversee the production of this joint report.

The experts addressed the following questions,
(a) what do the results from the second Compusoft report indicate about
(i) the response of the CTV building to the earthquake at 4.35AM on the 4th of September 2010,
(ii) the response of the CTV building to the earthquake at 12.51PM on the 22nd of February 2011,
(iii) why the CTV building failed on the 22nd of February 2011, and
(iv) the sequence of failure of the CTV building on the 22nd of February 2011.

(b) Do the results of the second Compusoft non-linear time history analysis change any opinion of these experts have expressed in evidence given or to be given to the Royal Commission in relation to
(i) the response of the CTV building to the earthquake at 4.35AM on the 4th of September 2010 or response of the CTV building to the earthquake at 12.51PM on the 22nd of February 2011,
(ii) why the CTV building failed on the 22nd of February 2011, and the sequence of failure of the CTV building on the 22nd of February 2011, and then

(c) if the results of the second Compusoft non-linear time history analysis change any opinion that they have expressed and what way has that opinion changed?

The experts attempted to reach agreement in relation to the questions set out in clause 4(a). I now cover the areas of agreement and the first are really a series of if you like uncertainties so what is agreed on what was uncertain in the answers.

The first point is the non-linear time history analyses are highly dependent on the input assumptions made. Non-linear time history analyses are difficult to calibrate in a quantitative way to actual performance of the structures under severe seismic loading. While due care has been taken and expert opinion has been sought in determining suitable inputs, the non-linear time history analysis panel recommends caution in the way the results are interpreted with respect to the collapse of the CTV building.

The report on the revised non-linear time history analysis of the CTV building describes the changes to the computational model required by the panel of experts and the resulting responses of the building to two of the ground motions recorded in the Christchurch CBD for the September earthquake and for the four ground motions recorded in the Christchurch CBD for the February earthquake. Two further analyses for the February earthquake were also performed using the structures damaged state following the September earthquake and a sequential analysis.

It must be emphasised that these are the computational results for the structural analysis model of the CTV building and these computational results must be interpreted back to the actual CTV building. The results may not describe the actual building behaviour because of

(i) the limitations of the computational model, modelling of the structural components. This is in particular so with respect to the modelling of the beam column joints.

(ii) the actual ground motion at the CTV site is unknown and the results are for the motions recorded at four sites in the CBD, Christchurch CBD. It is hoped that these will give a measure of sensitivity of the results to this uncertainty.

There's in general agreement on uncertainties in the computational model and its data.

(a) it was agreed that the concrete strength be taken at 1.5 times the design strength for the purposes of the revised non-linear time history analysis. I make a note that there is to be a future hot tub discussion between the concrete experts at the Royal Commission to discuss various concrete test results and this may provide further guidance on the likely concrete strengths. (b) No allowance for the effects of the masonry infill on line A nor the precast concrete spandrel panels on lines 1, 4 and F were accounted for in the revised non-linear time history analysis. The use of EI effective of 1872 kilonewton metres squared per metre for out of plane stiffness of the 200 Hibond floor slab represents approximately 0.2 I_g or 0.29 I_{av} and this is that of the I average is the average of the cracked and uncracked section properties used for deflection calculations per British Standard 5950.4 1994.

DR HYLAND:
Can I just make just a little point there? I think that should be E EIG.

PROFESSOR CARR:
Thank you.

JUSTICE COOPER:
Where's that?

PROFESSOR CARR:
In clause, clause C. I've got 0.2 IG and 0.29 IG.

JUSTICE COOPER:
It should be?

PROFESSOR CARR:
It should be EIG I suppose for the correct definition.

COMMISSIONER FENWICK:
It would be 0.1 or 0.2 of, is it 0.1 or 0.2?

PROFESSOR CARR:
Well I was given 0.2.

MR STUART:

I'll take that if you want, the, it all comes down to what your base reference is I guess for EI gross. So we took, Professor Mander gave us a number, is the 1872 value and inferred that that was 0.1. We took that number on its merits and since then I think I'm not sure if Mr Smith or Dr Hyland have recalculated that to infer that that is actually 0.2 times the EI effect, EI gross sorry. It's just the base point.

COMMISSIONER FENWICK:

One number we can rely on is the numerical (inaudible 16:27:43)

MR STUART:

The 1872 is a real number, the fractions of EI is…

COMMISSIONER FENWICK:

I remember there was some confusion this morning about what value it should be.

PROFESSOR CARR:

(d) drag bar strength. Expected strengths used are probably an upper bound. The Beca values are possibly a lower bound being based on lower characteristic strength.

(e) the beam column joint model does not consider the time variation and axial forces on the joint behaviour. They used the initial gravity loads. And does not allow for loss of vertical load carrying ability or for the loss of shear strength in the adjoining beams.

(f) there is no in cycle degradation of strength capabilities of the beam column joint model.

(g) a better beam column joint model is very desirable, however it should be noted that the CTV joints are different from those discussed predominantly and tested in the available literature, i.e. precast channel beam shells, small seatings with cast in place in the beams. There was a potential loss of shear capability in the beam connections to the joints. An appropriate joint model is not readily available and so parametric analyses of the joint constitutive model would be beneficial in subsequent analysis. Most joint models that are proposed in the literature are for two dimensional joint models whereas some of the joints with high vertical loads are in the interior of the structure and are largely two dimensional in nature, the inelastic effects appear to be more dominant in the exterior joints which are three dimensional. Developing calibration of a three dimensional model would be, I've lost a line somewhere…

JUSTICE COOPER:
Does anybody have the continuation of paragraph G?

PROFESSOR CARR:
It's at the top of the next, take a considerable time and effort, even if experimental data on the type of joint was available.

JUSTICE COOPER:
Now where did you find that, where did you find that Professor Carr?

MR SMITH:
That was in my copy Sir.

JUSTICE COOPER:
Mr Smith's copy?

PROFESSOR CARR:
Yes, so it's probably in the copy that was sent out by email but it's somehow –

JUSTICE COOPER:
That'll be the one for everybody to sign then.

PROFESSOR CARR:

It just says it will take a considerable amount of time and effort, even if the experimental data of the type of joint was available. I make sure I actually pick up the next point which is

(h) though the joint behaviour and the computer results appears to be better than the computed column behaviour, this may be a function of the simplified beam column joint model adopted.

(i) the potential pull out of beams at the exterior joints in lines A and F, the bottom beam bars of interior joints is not modelled in the revised Compusoft model. A selection of joints had been post-processed to see what is the likelihood of the occurrence of beam pull out at the joints. This pull out appears to have occurred for some joints at similar time to the column failures. It should be noted that the fibre modelling of columns does not explicitly consider bar buckling.

(j) The definitions of column failure have been addressed as follows. The first criteria for column failure is deemed to occur when the concrete strain of the column cross section centroid reaches the crushing strain of concrete. This implies widespread spalling of the concrete and significant loss of the axial load carrying ability of the column. Two further potential failure criteria are being monitored at the locations of vertical column bars, including:
i) compressive strain equals 0.0033 as an indicator of potential column bar buckling and
ii) ultimate column concrete compression strain as an indicator of significant concrete spalling. I note that the concrete inside the column reinforcement is very weakly confined, so spalling will not stop at the line of the reinforcement. Thus columns with only moderate axial force could also degrade to the extent that they could not support the axial force, particularly when vertical acceleration effects are considered.

(l) there needs to be a check carried out to see if the floor diaphragm strength between walls C and CD is sufficient to maintain continuity for the north tower after disconnection of the drag bar. The cause of computational demands and time constraints analyses is used for displacement theory for the P delta adjustment. Hence geometric non-linearities are not explicitly considered which may be significant given the high axial loads on vertical load resisting elements. That covers the uncertainties.

Moving to item 2, the revised nonlinear time history analysis responses of the CTV building, the earthquake records at 4.35AM on the 4th of September 2010.

(a) both ground motions indicate inelastic behaviour in the line F columns, i.e. yielding of vertical bars and/or concrete compression strains exceeding 0.002.

(b) Christchurch College record (CCCC) indicates disconnection of the drag bars at level 4 only, whereas the CBGS (Botanic Gardens record) did not indicate disconnection though level 4 drag bar force on line DE reached approximately 95% of the disconnection force.

(c) The CCC record predicts columns in gridline F at multiple levels and heavily loaded columns, base level 1 exceed their ultimate compressive strain.

(d) there was inelastic behaviour in some of the two way beam column joints in the CCC and the CBGS records. Inelastic response is predominantly for rotation about the east-west axis, i.e. due to north-south building translation. However, none of these inelastic beam column joints reach their ultimate strength.

(e) the walls in the north tower show some inelasticity at the base of the wall.

(f) the walls in the south coupled shear wall show some inelasticity at the base of the walls. And

(g) the coupling beam at the south tower just above level 1 show inelastic behaviour but only for the CCCC record.

3. The revised nonlinear time history analysis response of the CTV building for the earthquake records at 12.51PM on the 22nd of February 2011.

(a) drag bar disconnection is indicated at all levels, 4, 5 and 6 for all four ground motions early in the response. Performance of individual column hinges had been assessed first with respect to the time at which the vertical load carrying capacity has been compromised. An indicator of this could be taken as when the compression strain at the column centroid exceeds the ultimate strain capacity of the concrete. There is a table, it should be table 1 . It represents the final which the first column hinges are seen to exceed this criteria placed on ultimate compressive strain between .006 and .00825 in figure 6 in the Compusoft report.

Table one shows column axial failure times the L1s are notes level one and B the weight of the columns and there's a table which indicates the times at which the different analyses, the four different records and the ones labelled CSEQ are the ones where this analysis was with damaged structure and you can see in a general sense the CBGS whether it's damaged or undamaged doesn't make a great difference to the times at all and there really column D2. The records for the CCC analysis are slightly different. The structure has slightly different properties because of the disconnection of the drag bar at level 4, and secondly the damage into the column, some of the joints in the analysis but the failure times are not greatly different and the other two records are Christchurch Hospital and the Resthaven records are shown there for comparison. It's very hard to use the actual times to compare one record with another except for the same earthquake record because the starting times on the records don't necessarily line up in physical time. They are different distances from the epicentre and I'm not too sure what the time basis of the records are and been set up in.

Come to point (c) there was failure of the exterior line F columns. Some records predict upper level columns received a ultimate compressive strain prior to the axial load carrying capacity loss in the level one columns. There are quite large variations in column axial forces which will contribute to column yield interaction. There is considerable damage to joints particularly in the two-way joints with more damage being seen in gridline A than in gridline F. It appears as if beam column joint strength degradation occurs after some column hinges are shown to lose their vertical load carrying capacity. Post processing of the potential beam column, beam pull out suggest that such failure is likely to have occurred. As a result ideally this failure mechanism would be modelled explicitly in any revised analyses to allow for effects subsequent to this failure to be considered and then understand whether it is important in the global failure of the CTV structure. The analyses using the sequence of the September earthquake record followed by the February earthquake record only show a noticeable difference in performance of the CCC record whether it was a loss of drag bar connection in level four in the September earthquake. This appears to have delayed a loss of connect to the last drag bar to fail as it now fails later in the analysis and is seen in the analyses for the undamaged structure model. The displacement history maintains a very similar form in magnitude irrespective of whether the non-linear time history model of the structure was assumed to be damaged or undamaged at the time of the February earthquake.

JUSTICE COOPER:

I don't want to interrupt you but in a lengthy way but can you explain why, what's the – is there a reasonable explanation for what's reported in paragraph G in terms of delaying the loss of connection?

PROFESSOR CARR:

The structure would have different natural frequencies and if it's lost some of its connection there's a, the properties of structure have changed so it would change the time (inaudible 16:40:26) of what happened in the shaking.

JUSTICE COOPER:
I see thank you.

PROFESSOR CARR:
Item 4. Why the CTV building may have failed on the 22nd of February 2011 based on the revised non-linear time history analysis.

(a) The failure of the drag bar connections at levels four, five and six is predicted for all four ground motions. This will add increasing demands on the remaining north core diaphragm connection.

(b) The inability of the heavily loaded interior columns that carry their imposed vertical loads when subjected to the effects of inter-storey drifts was sufficient to lead the loss or spalling of the cover concrete. This concrete spalling could penetrate further into the columns beyond the longitudinal reinforcement because of the very limited confinement. The loss of the cover, concrete and the very small amount of confining steel may also allow buckling of the longitudinal reinforcement particularly if it had earlier yielded in tension. A large variation of actual forces use the very large vertical accelerations may also have had a part to plan in the column failure.

(c) The beam column joints also indicate large inelastic effects, in many cases exceeding their maximum capacity at the same time or soon after the column failures.

(d) The possibility of beam pull out from at least some of the exterior columns cannot be excluded.

(e) the inter-storey drifts on line F without spandrel panel interference shown in the revised Compusoft report are greater than the columns can sustain.

5. The revised nonlinear time history analysis collapse scenario appears to be, and there's a wee proviso that most of the panel wanted to put in: the variability and uncertainty in physical properties in analysis processes do not allow a particular scenario to be determined with confidence, but the scenario appears to be:

(a) drag bar failure.

(b) potential disconnection of the diaphragms to the north core.

(c) inter-storey drifts greater than the column drift capacity.

(d) potential failure of upper level columns due to onset of spalling of concrete.

(e) the loss of axial load carrying ability starting with the column on line 2, (column A2 of the CBGS record and column C2 for the other three ground motions) at the ground level followed rapidly by the other interior columns. The time interval over which a significant number of column failures occurs appears to be less than 0.03 seconds. Some columns also indicate loss of axial load carrying ability in the upper floors, such as level 3, column C2. It should be noted, (again I seem to have lost it)…

JUSTICE COOPER:

We all have I think.

PROFESSOR CARR:

…computed results after vertical load carrying capacity has been lost should be reviewed with caution as the analysis only considers small displacement effects. I've again seemed to have lost two lines from the document.

(f) failure of beam column joints in lines 1, F and A. There's a possibility that this may not be relevant as the structure may have already failed and the responses computed after that point may be meaningless.

Areas of disagreement.

1, column failure criteria. Dr Hyland considers that the concrete column failure criteria should be given by the cover concrete reaching an ultimate concrete strain of E_{cu} greater than 0.004.

2, slab flexural stiffness. Dr Hyland is unsure of how the lower value of flexural stiffness used in the revised nonlinear time history analysis was justified.

3, the revised nonlinear time history analysis response at the CTV building to the earthquake at 4.35AM on the 4th of September 2010. Dr Hyland regards that the damage predicted by the revised nonlinear time history analysis does not agree with the level of observed damage following the September earthquake. The column damage indicator for the CCCC record should've been observable, particularly at the upper levels. The revised nonlinear time history analysis appears to overpredict the damage of the September earthquake though only slightly the CBGS record.

4, the revised nonlinear time history analysis response for the CTV building, the earthquake at 12.51PM on the 22nd of February 2011. Dr Hyland comments that the scenario does not include allowance for interaction with masonry infill on line A nor interaction for spandrel panels on lines 1, F and 4.

(5) the revised non-linear time history analysis collapse scenario appears to be (a) Mr Smith comments that there was failure at the line F columns, some records predict mid to upper level columns which exceed the ultimate compressive strain prior to axial load carrying capacity being lost in the level 1 columns.

JUSTICE COOPER:

Now I'll just confer with my colleagues for a moment. We think Mr Elliott that in fairness to everyone we should actually listen to the individual statements now being read and then that can be followed by a discussion to see where that ends up. So a slight amendment to what you proposed. – And we would begin then with Professor Carr again I think and so are you happy to keep reading for another –

MR CARR:

Most of my report is very similar to the one that I've already discussed, if you look at it many of the paragraphs that in fact are the same.

JUSTICE COOPER:

Well we'll take it as read, it can be part of the record. We would move on then to Dr Hyland.

DR HYLAND:

Okay, so item 6(b) do the results of the second Compusoft NTHA change any opinion expressed in evidence given or to be given to the Royal Commission in relation to the response of the CTV building to the earthquake at 4.35AM on 4 September 2010? My answer is no. The calibration of the NTHA to the reported damage remains problematic. No effects of damage to or effects of internal partitioning on floor response was able to be made on the horizontal or vertical response of the NTHA model. Both ground motions indicate inelastic behaviour in the line F columns, ie yielding of vertical bars and/or concrete compression strains exceeding 0.002. Figures 35 and 36 in the NTHA 2, this is the draft we had prior to today illustrate the need for caution in interpreting the NTHA results in an absolute rather than a comparative or relative sense. In figure 35 the inter-storey drifts in September 2010 would have achieved full contact with the infill masonry on line A, even if the specified gaps had not been compromised by mortar as reported by eye witnesses. In figure 36 the drifts on line F in September 2010 would have been sufficient to exceed the column EC strain of 0.004, failure criteria in the CCCC record. For both the CCCC and CBGS records the columns would have well exceeded their elastic deformation limits of concrete strainer point at 0.002 and from level 2 and above the tensile yield limit and the reinforcing. Once tensile yield initiated spalling would have been likely, obvious and significant levels of spalling would therefore have been expected to be observed from level 3 and above based on NTHA 2. No spalling was observed in any of the columns from level 1 to the underside of level 6 by those who inspected after September 2010 earthquake. In fact only column F4 at level 4 was identified as having a crack in it. The CCCC record indicated disconnection of the drag bars at level 4 only whereas CBGS record did not indicate disconnection, though level 4 drag bar force on line DE was up to approximately 95 percent of the limit force. Movement of the slab relative to the walls would have been required to exceed 2 millimetres prior to drag bar connection failure to account for bolt hole oversize allowance let alone that required for anchor failure. Such movement would have resulted in significant or observable cracking in linings at level 4 at the lift doors, however none was reported. There was inelastic behaviour in some of the two-way beam column joints in the CCCC record, however these inelastic beam column joints did not attain their ultimate strength. However no damage to the beam column joints was reported though it should have been observable. Drifts on line 2 compatible with the damage observed after the February aftershock had been estimated by a static drift analysis to be less than those necessary to cause net tension in the reinforcing steel of the bottom beam reinforcing steel. This indicates that the beam column joints may not have been subject to significant internal reversing shear demands and damage in the September earthquake or prior to collapse in the February aftershock. Walls in the south coupled shear wall in NTHA 2 show some inelasticity at the base of the walls though only one fine crack was reported, this is after September. The drifts shown along line 1 in figure 37 exceed those that appear to have occurred in the south wall prior to collapse in 22nd February aftershock, estimated to be .4 percent.

So part 2, to item 6(b) the response to the CTV building to the earthquake at 12.51PM on 22 February 2011. Does it change my opinion? No, though the vulnerability of the level 1 columns to enhanced demand due to two-way action has been highlighted in – highlighted if line A masonry and spandrel panel interference is ignored, sorry that's though the vulnerability has been highlighted more in this model. The effects of the masonry infill on line A nor the precast concrete spandrel panels on lines 1, 4 and F were accounted for in the NTHA 2 analysis. A lower bound cantilever flexural yielding masonry infill model was used in the original NTHA, this assumed vertical and horizontal disconnection from a frame of panels 2.39 metres wide. Shear strength and degradation model used assumed significantly lower capacities at 900 kilonewtons for the west wall than the NZS4230:2004 grade B masonry recommendations and allowing for concrete column shear contribution of a total of 2800 kilonewtons. Stiffness of the line A masonry infill is observed prior to the February aftershock including vertical and horizontal contact with framed columns and beams may have been a panel with effective length up to the full depth of the building of 22 and a half metres and dominated by shear and compression field behaviour until such time as the damage began to degrade its stiffness and strength. Effective stiffer masonry wall infill in this condition has not been considered in the NTHA. 3D static analysis and ERSA has shown the effect of a stiffer masonry infill to be significant on torsional response and would also be expected to provide some limitation to north-south drifts on internal columns affecting the two-way demands on them. NTHA 2 predicted drag bar connection, disconnection at all levels, however the collapse evidence showed that the drag bars only failed after collapse had initiated near the south end of the building. Eye witness testimony including that of survivors at level 4 is in agreement with the collapse evidence. This shows that the NTHA 2 model does not adequately describe in an absolute sense what happened during the collapse on 22 February 2011.

Item 3, does it affect my opinion on why the CTV building failed on 22 February 2011? No, the NTHA 2 shows that there were sufficient drifts for column failure to occur, and indicates a number of vulnerabilities were present such as drag bar disconnection and beam column joint fragility. This was also shown by the original NTHA.

Item 4, does it change my opinion in terms of the sequence of the failure of the CTV building to the earthquake at 12.51PM on the 22nd of February 2011? Again no. The sequence of failure presented in the BCR, that's the Hyland Smith report was developed considering convergence of the collapse evidence, eyewitness accounts, 3D ERSA, static analyses, nonlinear pushover analyses, drift compatibility analyses as well as the NTHA. It was recognised that there was difficulty in calibrating the NTHA results to the damage observed in developing the preferred collapse scenario.

At item 6C, if the results of the second Compusoft NTHA change any opinion they have expressed in what way has the opinion changed? No, there hasn’t been a change. NTHA remains useful but problematic in terms of calibration to the damage observed in the September earthquake and February aftershock. If treated as a closed system of modelling assumptions and input motions it reflects diligent and best current practice using the SAP software, however the results compared to report damage indicate the limitations of current NTHA practice in accurately modelling real building response to earthquakes.

JUSTICE COOPER:

Well that brings us to 5 o’clock or as close as we need to be. Mr Elliott can you, I’d like to have, the Commissioners to have the version with all the words in it of the joint report that Professor Carr read to us, and also if you could get it signed. Mr Rennie, that’s going to be problematic with Professor Mander isn't it but can you sign that on his behalf do you think?

HEARING ADJOURNS: 4.58 PM

INDEX