HEARING RESUMES ON MONDAY 23 JULY 2012 AT 10.00 AM

MR RENNIE CALLS

JOHN BARRIE MANDER (SWORN)

MR RENNIE ADDRESSES THE COMMISSION – ORDER OF EVIDENCE

EXAMINATION CONTINUES: MR RENNIE

Q. Now Professor you're John Barrie Mander?

A. Yes I am.

Q. A New Zealand citizen currently a permanent resident of the United States at College Station, Texas. You hold the position of Zachry Professor of Design and Construction Integration with the Zachry Department of Civil Engineering at Texas A & M University?

A. That's correct.

Q. Now can you certify in accordance with the High Court rules that you read the code of conduct for expert witnesses and your evidence complies with the code of conduct's requirements?

A. Yes I have read them and I believe it complies.

Q. The matters on which you expressed an opinion are within your field of expertise?

A. Yes.

Q. In paragraph 4 of your brief of evidence you note a connection with three students, I correct myself with two students, and a previous witness in relation to professional matters in the past?

A. Yes.

Q. So therefore would you start from paragraph 4 of your brief reading from, "I have no interest or relationship..."

WITNESS READS BRIEF OF EVIDENCE

A. "I have no interests or relationships with any parties to these proceedings, although in the interests of transparency, I note:

(a) From 2008 to 2010, I funded and was the research supervisor of Mr Christopher Urmson as well as the major advisor for his masters of

compression meridians running through the joints this way and they'll run parallel and then this, the tensions in here weaken the concrete. Now to add insult to injury on the columns when everything reverses in the opposite direction, pretty much in the direction that Dr Fenwick has drawn his sketch then you'll have compression down this side and then you'll get cracks on this diagonal so you end up with these X shape looking cracks in the beam column joints and they really, really struggle to provide the loads that are demanded on them without the concrete in that joint zone deteriorating.

So on the first pulse of the Christchurch earthquake if the inertia force has pushed the CTV building from the left to the right the forces may, I have read this, but if the axial load in the columns is high as in the lower storeys in the CTV building at least some damage will be done. It is this damage promoted by the tensions in the off diagonal that leads to progressive softening or weakening of the concrete on subsequent cycles.

Calculations have been performed that show that the overall joint forces will restrict the potential input forces from the columns to about 70% of the potential maximum of that shown in figure 2.7. Therefore the columns, apart from during the initial cycle, will remain mostly undamaged. Yet the condition of the beam-column joints will continue to deteriorate as the cycling progresses.

The weaker joints in the CTV Building were a mixed blessing. The weaker joints actually will have acted like a fuse and therefore protected the columns from any further damage. However, over time the concrete will have worn down to the point where it could no longer sustain the axial load passing from the storey above through the joint to the storey below.

In spite of the deterioration in the beam-column joint zones, if the structure remains well tied together by the floor diaphragm, and also tied back to the shear wall system, the columns remain "trapped" and unable to fail due to a sideways action. Of course the joints must continue to be capable of transmitting the vertical load. Providing the axial load path can be maintained and the joint concrete does not crush excessively, the joints continue to function as a fuse. This initial phase of the partial failure, where the joint system acts as a fuse, is shown in figure 2.10."

So remember the previous slide that we had we had a displaced structure like this and we had plastic hinges at the ends of the columns, well I maintain that may have happened momentarily on the initial cycle but then beyond that what's going to happen is that it gets into the joint and all the damage is going to take place there because this is relatively weaker than the columns and so this in a sense acts as a fuse. You still have an axial path down to the ground.

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WITNESS CONTINUES READING BRIEF OF EVIDENCE

A. Based on an examination of all the beam-column joints in Figure 2.10, it may be noted that the exterior beam-column joints may have "failed"." I'm not saying that they have and that's because the demand on those exterior ones are not as high. "Failure of these connections is considered to be one of the primary triggers that "releases" the neighbouring columns, giving them room to move laterally (sideways) at one floor level with respect to the floors above and below. It is hypothesized that this is the "trigger mechanism" that eventually led to the collapse of the CTV Building. But it should be noted that for such a failure to occur after the "trigger mechanism" has released the beam, no further external loads need be applied, instead the gravity load alone is sufficient to collapse the structure.

HEARING ADJOURNS: 1.02 PM

HEARING RESUMES: 2.15 PM

EXAMINATION CONTINUES: MR RENNIE

Q. Professor, we were at page 35 of your evidence, section 2.6, "expected seismic performance of an exemplar structure in the Christchurch earthquake," and we have slide 29 on the screen.

A. Thank you Mr Rennie.

One might wonder how other buildings built in accordance with contemporary codes of practice perform in the Christchurch earthquake. This topic has been investigated and recently reported at the 2012 New Zealand Society of Earthquake Engineering, Mander and Huang, 2012. And I might add that the actual work for this, for the most part was done going back over some time, well before the earthquakes actually took place and basically we've put it into a context that is relevant to Canterbury.

For many years senior undergraduate civil engineering students at the University of Canterbury have been taught the principles of design of multi-storey reinforced concrete buildings, with a particular emphasis on seismic loading effects and the detailing of reinforcement for ductility. The exemplar structure used as part of the educational process is the so-called "Redbook" building. This is a 10 storey precast concrete structure, it could perhaps be considered a modern rendition of the CTV building. It's taller of course, it's 10 storeys.

A comprehensive computational analysis was undertaken for 20 different strong earthquake ground motions whereby incremental dynamic analyses were performed at increasing levels of seismic intensity until the structure collapsed. Now the slide here shows the results of such an analysis for the Redbook building and that was done, Dr Bradley actually worked on this particular project a number of years ago, and also incremental dynamic analysis is really the most advanced and contemporary way of doing such an analysis. It's not for the faint hearted. It takes a long time to do, largely because just to create that graph there are some 400 runs of analyses that are done and then the results collated and plotted accordingly. The results were then characterised in a probabilistic sense so that the median response and the dispersion of the outcomes identified in a risk-based format similar to that described in one of our recent papers in 2012. The computational analysis results of the general ability of the exemplar Redbook structure to strong earthquakes were then compared to the seismic demands imposed to similar structures in the Christchurch region. The outcomes were characterised in terms of a damage ratio with respect to the distance to the epicentre of the Christchurch earthquake, that is the cost of repairs or replacement to that of a similar structure constructed under stable economic conditions prior to the earthquake.

The analysis was also expanded to investigate the ramifications of the likelihood of fatalities arising from a collapse and the expected downtime due to the earthquake-induced damage. Additionally, several swing analyses were conducted to examine the sensitivity of the structural strength and reinforcing details on the general seismic performance. A summary of certain key findings from the investigation are described below, full details may be found in the attached paper that was submitted by Mander and Huang.

Q. Go to slide 30 please.

A. Figure 2.11 presents a so-called damage attenuation relationship for the Redbook class of building to the Christchurch earthquake. The results from the advanced computational simulations are presented in a probabilistic fashion, so that an idea of the spread of potential outcomes can be viewed. It should be noted that one cannot be emphatic about a certain outcome as the results contain the uncertainties in the structural response, the uncertainties in the distribution of ground motions due to soil variability and the difference of the as measured earthquake signatures at different sites based on actual Geonet data from the Christchurch earthquake. Also, the volatility in the cost of contracting and reconstruction after an earthquake is accounted for in the modelling. Therefore, there is considerable variation in the outcomes.

Now those looking at this who may have seen this sort of thing before, I need to point out why there's this dog-leg in here. On the lower curve down in here, this is essentially where the structure is in general damage and if you were aggregating the damage over the entire structure you might just take an average. The upper portion however up on the higher plateau, different criteria need to be used simply because whether a building is capable of surviving really is dependent upon the weakest link in the chain, so you may have one storey that's severely damaged that could lead to collapse or it's not possible to repair so if this is irreparable damage then the structure would be essentially written off and it's worth pointing out again that the average of this cost is going to be bigger than one or 100 percent of the static condition costs like before the earthquake and that's due to price surge. We all know that Fletchers charge a lot of money these days for labour, and it's not that high before the earthquake but it's a lot higher now and a lot of work is done on essentially penal rates and which inflates the price and the cost. It is possible to have – some buildings may end up being a little cheaper to build simply because from before the earthquake not all the structure is damaged so for example a given structure may be damaged but the foundations can be redeemed and the new structure built on what was previously there so that would be some cost saving. Now there's considerable spread this way in the damage and the cost outcome, but there's – and that you might say comes from the structural effects and the cost of contracting and so forth. In this direction though the variabilities are more to do with the ground motion and how the earthquakes changes the radial distance increases out from the epicentre. The point to note here however is that the CBD is somewhere less than about eight or 10 kilometres where a lot of the concern is, and you can see in this region here that there's, well if we take this beam line here right on one, then there's an 84 percent chance that the building is going to be not redeemable after an earthquake, so it is for that reason it's not surprising buildings that were held up as being exemplars after the earthquake, such as the IRD building, are actually now going to come down as a consequence. Now this doesn't necessarily mean that there's going to be collapse and people will get killed, it just means that the building is irreparable and it needs to be taken down. Now one can also draw similar graphs for the probability of fatality and also the outage times, in other words the down time necessary to invoke repairs. Where did I get to Sir?

JUSTICE COOPER:

Q. I think you had summarised the balance of page 36 and go to page 37.

A. Thank you.

EXAMINATION CONTINUES: MR RENNIE

WITNESS CONTINUES READING BRIEF

A. "The extent of the CBD ranges from some five to nine kilometres from the epicentre. From an examination of figure 2.11, it is evident that most structures, at least some 70 percent, particularly those closer than 10 kilometres to the epicentre would not theoretically survive, and would require demolition and reconstruction. In fact there is already sufficient anecdotal evidence to support this analytical result. Thus in spite of modern buildings being constructed to textbook standards, they could not have been expected *a priori* to survive the Christchurch earthquake. Another question arising from this work is could one expect to see fatalities as a consequence of the damage arising from the ground shaking? Analysis results show in Mander and Huang, 2012, that deaths are not likely providing the structure conforms to the present day code based design. However loss of life and limb cannot be ruled out and the modelling results show there is about a 10 percent chance that if a structure collapsed occupants could be killed."

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EXAMINATION CONTINUES: MR RENNIE

Q. Now Professor we're now going to go from page 37 of your first brief to your second brief. Do you have that available?

A. Yes.

Q. And if you turn to page 1 of that there's firstly confirmation of your identity and a reference back to your qualifications and we can move on from those to paragraph 3. Do you have that? "I am providing this.....

WITNESS READS FROM SECOND BRIEF OF EVIDENCE AT PARAGRAPH 3

A. "I am providing this second statement of evidence to introduce the results of further investigation and analyses I have been completing. As far as my other commitments have allowed, I have followed the first two weeks of the Royal Commission hearing via the online live stream and archive. These further investigations and analyses have been prompted primarily as a result of evidence I have seen presented by other witnesses, in particular Dr Heywood and Dr Kehoe.

In carrying out my further analyses, I have worked closely with my former mechanical engineering PhD student, Dr Geoffrey Rodgers, currently a Post Doctoral Research Fellow at the University of Otago Medical School and an adjunct to the Department of Mechanical Engineering at the University of Canterbury. Dr Rodgers has primarily assisted with retrieving and collating the records from the four recording devices around the CBD that are referenced in my report and running the computational model to generate the fatigue spectra.

The new evidence presented in my report helps supplement evidence presented in my first statement regarding side-sway as to why structures fail. The methodology and reasoning I present are not well known or widely understood. Much more work could be completed to further advance the analyses but time has not permitted this. The project should, however, be considered a work in progress.

My report is annexed.

Supplementing the information in my annexed report, further details of the timing, peak accelerations and locations of the significant earthquake events from 4th of September 2010 and up to and including 22 February 2011 are given reference there.

Q. If you next turn to your document – "Progressive Damage Accumulation in Earthquakes within the Context of the CTV Building Collapse". Do you have that?

A. Yes I do.

Q. Can we have Slide 1 and we now go to Slide 2. – "Contextual Background" – Can you read from there.

A. "There are two key factors that lead to the overall damage effects on structures in earthquakes. The first is the maximum response displacement or drift that arrives from side-sway effects during shaking. The second, and often neglected effect, is the duration of the earthquake and the cumulative damage effects caused by the repeated cyclic loading.

Cyclic loading demands and their effects can lead to fracture or failure of key structural elements, and thus act as a trigger that will either lead to a lack of serviceability (such as, for example, excessively high floor vibrations) or a general collapse condition. Such phenomena come under the general category of fatigue loading. Fatigue can be considered in a disciplined way by separating the phenomena into fatigue capacity versus fatigue demand. This report focuses on the latter aspect in an earthquake engineering context. There are two types of fatigue demands that plague structures, high-cycle fatigue and low cycle fatigue.

High-cycle fatigue is the most well-known class of fatigue. (Slide 3 displayed) It occurs under normal day-to-day operational conditions and is a common problem in aircraft and other mechanical structures that are prone to vibration effects. The number of cycles to failure for this class of mechanically engineered system is generally in excess of one million cycles.

Civil structures, such as steel bridges, can also suffer from high cycle fatigue. To provide fatigue resistance, civil engineers strive to keep the double amplitude stress reversals below a so-called fatigue-limit threshold. For steel, this can be in the order of 150 MPa. High cycle fatigue generally occurs where the stresses remain in the elastic condition.

Low cycle fatigue can also plague structures such as buildings aand bridges under extreme loading cases such as in earthquakes. The number of cycles to failure is referred to as “low-cycle” because the material is commonly expected to be taken well beyond the yield stress or strain limits into the inelastic range of behaviour. Much work has been done in this area by the authors and others. See, for example, my list of three key references there.

Q. If we just go to slide 4.
A. Now we’re entering into a number of slides that is off the script a little bit so I’m going to talk through these and when we get to slide 12 I believe, we go back to the manuscript again. So this is something that came up last week obviously quite a lot and what I would like to do is to address this. Dr Reay made some comments about strain hardening and I do tend to agree with Professor Fenwick here. There’s a little bit of misconception surrounding this. I pose this as a question – Is strain hardening a surrogate for low-cycle fatigue? Well, what is strain hardening? So if we take a piece of reinforcing steel tested in tension so this one here, R13, is a normal reinforcing bar tested from zero strain out to its fracture, well almost beyond its fracture strain, out to just beyond the maximum peak stress. It goes up here linear elastically. This has always got the same slope of about 200 GPa and then it reaches a yield plateau. So this little straight horizontal line here is generally referred to as the yield plateau. So that’s yield but it’s not strain hardening. The strain hardening strain is right where the cursor is show at the moment and then –

JUSTICE COOPER:

You'll have to use words so that when we read the text we'll know where the cursor was. So you can identify it by reference to the two axes presumably.

A. Yes, so at about 330 MPa stress and a strain of about 1% or .01 on the graph that's the onset of strain hardening. So what you'll notice here is this curve starts to rise up and then it follows up around here and it gets up to the top.

Q. That's the R13 curve.

A. R13 and this top point had a strain of about 0.13 and stress of about 570 MPa is defined as the peak stress or the ultimate stress and the associated ultimate strain. Don't confuse that with fracture strain. That typically comes later at about .18 or .2. Now this whole region from here right through is called strain hardening and what typically will happen with members that have fairly light levels of axial load you will go up into this region well up into here and so often strain hardening can be used as an indirect, and I must emphasise this is an indirect measure of damage done in terms of the remaining life in terms of fatigue of the material.

The corresponding companion graph there, R2, this is the same type of steel cut from the same bar but tested in compression and what happens in compression is if the bar is transversely supported at a spacing of less than about or equal to about six longitudinal bar diameters, so for example this was done on a 16mm bar and so if we're about, you know, whatever six times 16 is, out here we get out to a 5% strain. That becomes the maximum compression stress at about 5% strain but it should be noted that there are often in the field tell tale signs where this happens. Typically what one will notice is that the cover is spalled off and the longitudinal bar will have buckled and it'll be quite noticeable and there is an example of that we can show from Mr Frost's evidence.

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Q. Do you want to go to that now or later?

A. I think bring it up when it comes up more naturally perhaps.

Now those tests that were shown I've just discussed (Slide 5) were called monotonic tests. They are monotonic because they just go in one direction. If we now move on to cyclic loading tests the first graph here is where what we've done is cycled between a constant strain amplitude of plus or minus .008. So the point to emphasise here is if this steel is embedded in concrete the concrete is probably going to start crushing on the cover if it's contained in a well confined cage and it's going to perform pretty nicely and it's going to go out and in this case the test went out to about 152 cycles as given on the right-hand graph before it started to reduce its strength. Now what's happening at that point is that there will be some sort of microflaw or crack will appear in the metal then under repeated cyclic loading that crack propagates, grows until it finally fractures. Now the example here is where you have cyclic loading and fatigue but without strain hardening okay.

Now the next three graphs are kind of the opposite to that. They are cyclic loading with strain hardening and the three graphs across the top of the page show you might say mild, medium and large strain amplitudes. The first graph is cycle between plus or minus 2% strain. It gets out to 9.2 cycles. The next graph is cycled between plus or minus 3% strain and it only gets to 4.1 cycles and then finally the third graph gets to plus two minus 6% strain. It's very heavy in compression but it gets to two and a half cycles so as we can see in general simple terms as the strain amplitude increases the fatigue life or in other words the number of cycles of earthquake excitations, that the structure can survive through decreases or more still you might say the bigger the earthquake the less the fatigue life so the duration effects do very much affect the fatigue life of structures and the intensity of the earthquake of course as well. So just returning to that slide the question was is strain hardening a good surrogate for low cycle fatigue – my answer is possibly yes but the phenomena should strictly be referred to as low cycle fatigue. Now we can algorithmize all of this and there are a number of equations that one can go through and this relates to the second of the references that are listed at the back of this where the fatigue capacity can be related to the first equation there is a general fatigue capacity and is essentially characterised by two parameters. This exponent here becomes significant C=1 or -.5 in this case. This is a lower case strength and then this parameter Theta hoop is really just a scaler that one can relate to this quantity on the left which is plastic curvature. Okay now this can be applicable for both concrete and steel under the number of cycles of loading and both will fail differently and independently actually so if we look at this all these equations can be best explored through this next graph on figure 9.

The flatter of the two slopes this broad band through here, and in fact it's done as a broad band because there is quite a bit of uncertainty with fatigue life and that's drawn accordingly to represent that, is the low cycle fatigue of the longitudinal reinforcing steel. This type of failure mode by the way is essentially non negotiable. You can't change this by design because we are, we have to live with a type of steel that we receive and different steels, are fairly consistent with these results. What you can do though is you can change the position of this steeper line of the two and if you put tightly spaced transverse reinforcing into the structure then this curve will move up which means that there's less chance of getting a concrete failure indirectly by a heat fracture of the transverse reinforcement but there is a point where that's not going to be of particular use to you. If you have basically no transverse reinforcement such as the case in CTV this curve here would move way down so that it's most probably below the flatter curve of the two.

Now I just want to point out that my following remarks are going to relate to this form of the equation up above so we can characterise these curves in here by this simple equation that the number of cycles until you get a fatigue failure is related to sum coefficient times either the strain, the plastic curvature or the rotation in a member raised to the power of C, and that value of C if it was the concrete failure that would be one and if it was for the reinforcing steel failure on this line here that would two. So that's some more of the theory and what I would like – this is the third paper and we did a study in 1999 (Dutta et al) on the retrofit for control and repair of the linear damage and that was done through some quasi earthquake displacement techniques method of full scale testing. It was all related back to the theory and we found much to our surprise that things weren't all as they seemed. So what was different? Well in the experiment we were also mimicking the vertical load effect and the vertical load effect made quite a marked difference and so we modified the theory to cope with that so that's equation 13 here on the slide and if one was to put in there Delta T which is the fluctuation in the axial load as was the case for the CTV building then one would note that the fatigue life is only going to be about one third of the normal case so this really brings things to a more sudden and dramatic halt and you will have a lot of potential for fatigue fractures and again this is not surprising that many buildings in Christchurch have been condemned because of these fatigue effects. Vertical motion makes the significant difference.

Now what I've talked about up until here is really the capacity side of the equation. Now we need to move into fatigue demand and then starting back some 20 years ago I had some students work on this particular problem and then we repeated some of the work as more earthquakes came to hand and I might mention like back then there were very few earthquakes that one can use and so this particular graph – I'm sorry it's a very poor reproduction from the ASC paper but two curves do stand out quite well. One is the envelope around here which is we were going to incorporate this in design this would be like an upper bound that would be worth taking note of and then this heavy black line through here which is a bit of a wiggly line –

Q. Line marked Pacoima?

A. No I'm sorry Mr Rennie this is not the Coiler Dam this is actually the heavy line for El Centro. Now El Centro has been mentioned already in these proceedings by Professor Shepherd I believe was really the benchmark ground motion. That's all we had back in the day to concoct a design spectra with and indeed the New Zealand spectra basically along with other world codes such as in the United States essentially used this as developing the code and so this will follow on in some of my remarks.

So the cycling demands of earthquakes this has been qualitatively appreciated since the 1970s. Not well quantified due to lack of ground motion records prior to 1989, but one thing that we were all aware of and I was certainly aware of it back in the seventies and the early eighties when I was a graduate student at Canterbury University was this: the clause C 3.2 of NZS3101.

Q. Should that in fact be 4203?
A. Yes I'm sorry Mr Rennie. It sure should've been NZS 4203.
Q. We will correct that clause 3.2 of NZS4203 on slide 11.
A. The structure should be capable of sustaining four fully reverse cycles of loading without losing more than 20% of its strength capacity and when we used to do testing in the labs down the road we often used this as a bit of a gold standard as to whether the cumulative ductility factor was about 32 or not and so indirectly we were taking this into consideration. Is this a realistic measure? Well the part of the difficulty here is we have to be careful as to how, how it is measured. A real earthquake is like shown in the top slide, it's a very random motion, in fact that's one for the Kobe earthquake, one second period, and if we were to transform that into equivalent constant amplitude number of cycles of loading then the middle graph with the green curves show that there is some 5.4 effective cycles of loading that would've caused the same damage. So the same damage between the black curves at the top and the green curves beneath.

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JUSTICE COOPER:

Q. Shall we call that the dark green?
A. The dark green yes, because there is a light one below yes.

EXAMINATION CONTINUES: MR RENNIE

A. So this is for when the coefficient C = 1 which really is representative of concrete damage. If on the other hand we're concerned about steel damage and we were trying to exert the same number of cycles that would leave the same damage to the steel, then the right middle graph with the red, the big red saw tooth curves, this would show that there are roughly 2.5 cycles for this particular black motion up here. Now this, in both cases this is normed back to the peak. Notice that the peak on the black curve up here is 0.25. Both of these are 0.25. Okay, now most people will be aware who, who have studied earthquake engineering that the Kobe earthquake in January 17th 1995 was a pretty massive and violent earthquake and bigger than what we indeed got here in some aspects of it. But if we were to norm that back to what was a New Zealand, say, design standard in Christchurch of the day then the spectral amplitude as by design would be what we discussed earlier this morning. It would be a 0.38 g spectral acceleration and then once you convert that into a spectral displacement that comes out to be 94.4 millimetres, you can see that's pretty close to a hundred on the plot there, and so this would be the equivalent number of cycles, if you, so if you transfer from the dark green graph where everything's normed to 250 millimetres, down to the light green graph, then the number of cycles would increase from five to 14.2. The point to make here is that both, both greens are giving the same amount of damage. Okay, it's just to do with how you reference it. Now onto the right-hand side, then, similarly for the steel. The differences here are more pronounced because remember that coefficient C is 0.2 so you go from 2.5 cycles to 17 cycles will give you the similar amount of damage. So I don't want to go over the next slide because it'll probably have everybody asleep. This is the mathematics that supports all what I've just presented. And we did check this, that we were doing this correctly because there is some, quite a bit of data processing, so one of the methods we used is a signal processing technique based in RMS counting and then we used an area equivalent that Professor Pacer's group and including Dr

Rodgers at Canterbury University have used a lot in their signal processing. So we basically got good agreement between the two so we're confident in these results. If we move on to the earthquakes and I probably should start reading again.

Q. Now I'll just pause you for a moment because there's one matter you wanted to come to which was the photograph of Mr Frost's evidence that you said you'd come to at a convenient time?

A. Oh, yes that would be quite convenient now thank you.

Q. Quite convenient now? And the reference is WIT.FROST.0001.32

WITNESS REFERRED TO SLIDE

Q. Do you have that?

A. Thank you, that's what I wanted to see. Now this I believe is the south wall. I think you know this because you can see Les Mills in the background over here and the south wall basically would've rocked backwards and forwards in plane on a strong axis of bending, and then when the building collapsed it folded in. So I think the folding in of the wall and the cracks that you see running longitudinally down the right-hand side of the photograph here are perhaps an artefact of the collapse of the wall. However, one part I'm quite positive is not part of the collapse, and that's right down the bottom near the toe of the wall here you'll notice that there in the middle, so this is where it's not going to be bending much anyway, but in the middle you'll notice this piece of, little piece of reinforcing bar. And what this has done as the strains in this region would've been quite high and you can see the hoops that have confined it, and they are all working nicely, in fact you'll see sufficient evidence in my mind that these are straining quite hard because they're bulging. And then the longitudinal bar itself is buckled. So if you recall back to the slide I showed earlier where I mentioned about buckling of steel and six longitudinal bar diameters, this more or less is exactly what we're seeing here. So had the earthquake continued on longer, and we don't know, like sometimes these fractures take place behind what you see, they could be behind the hoop, there's likely to be fractures if not now then a few more cycles you would definitely see it.

Q. And by "fractures" you're referring to fractures in the steel reinforcing bar?

A. Yes and a fracture is in this context is more or less literally severed all the way across. It's very visible to the eyes.

Q. Thank you Professor. Now if you go to the top of the page which the reference in the Commission system is 2.5, "Although structural engineers are not explicitly required…" Do you have that?

A. Yes I do.

Q. Now to some extent you've discussed those three paragraphs when you were referring to the graphs. Can you just scan them and see if there's anything there that you haven't covered?

A. No I think I probably should start at the next section.

Q. Yes, let's suggest that, so we go to, "Fatigue demand analysis of the Canterbury earthquake sequence…" and we're on to slide 16.

WITNESS REFERRED TO SLIDE 16

A. This report seeks to quantify the degree of cyclic loading damage imposed on structures in the Canterbury earthquake sequence and to draw conclusions as to how the cumulative effects compare with the cyclic loading demands implied by the code NZS4203.

There were 15 earthquakes greater than or equal to magnitude 5 from 4th of September 2010 through 22 February 2011. A summary of these records in terms of their location and peak ground acceleration, shaking intensity, are given in table 1, that's on the screen now, and I might highlight that the red ones are all, are the earthquakes where the ones we've taken into consideration specifically as being high. And these are the five, well the –

Q. The five you've taken into consideration are the 4 September one on line 1?

A. Yes, yes.

Q. The 8 September one?

A. Yes.

Q. The, what has been called the 19 October one?
A. Yes.
Q. What has been called the Boxing Day earthquake?
A. Boxing Day.
Q. And finally the 22 February earthquake?
A. That's right. Now we did analyse all of those but we've got, we're presenting some of the graphical results for each of these. The Boxing Day 2010 earthquake, although less than magnitude 5, is also included because of its proximity and effect as I mentioned this morning, it was more or less directly beneath the CTV site.
Q. Go to slide 17?

WITNESS REFERRED TO SLIDE 17

A. The response spectra for the five earthquake events with the highest recorded peak ground accelerations between those dates are shown in figure 1. Figure 1 shows that while all of these five major events have notably high PGAs, the spectral response that equals zero – which is at the peak ground acceleration, I'll just refer to this, is what you read right in here on the left-hand side, so it's the, where it cuts the vertical axis.
Q. So close to the vertical axis in the second graph on the slide?
A. Yes. However all of these five events have notable spectral response in the T = one second period range which will produce ongoing cumulative demand on a structure with periods in the range of one to two seconds. Figure 2 presents the spectra of the effective number of cycles of three different fatigue exponents. I think these should be on the next slide here.

1455

Q. That's slide, logically it should be slide 18, but we don't actually have a reference to that, 26, slide 26.
A. Thank you, and there're actually three graphs and there are only two on the screen, the two that are more relevant for this piece of work which is the C=1 which is for the concrete critical fatigue and C=2 for the reinforcing steel fatigue. These fatigue exponents correspond to

different material classifications. I don't think I need to go through all these equations again.

Q. No.

A. Thank you.

Q. Where would you like to pass through to?

A. I think we probably should go back to slide number 13.

Q. Yes slide 13, that would be on the next page, for the results of figure 2 on –

A. Oh no, I think we've done that, let's go up to slide, sorry we have talked about that, slide 26.

Q. Yes. So to be clear you're now in the paragraph beginning with the results of figure 2 and you're going on to discuss the position –

A. Yes I'll read that paragraph because that's quite relevant to this slide.

Q. Thank you.

A. For the results of figure 2, each orthogonal ground motion record for every available record recording station from CBDS, CCCC, CHHC AND REHS were simulated and normalised to the reference amplitude, that is the spectral displacement for that specific record. Therefore the results of figure 2 present the record to record variability in terms of record duration and distribution of response cycle magnitude.

Q. And just go to slide 27.

A. The results of figure 2 are plotted on a log log scale, the normal mean and plus or minus one log normal standard deviation, that is the sixteenth and eighty-fourth percentiles are presented on the graph as red lines. The black lines represent the fitted trend lines to these lines, therefore these results give an indication of the record to record variability indicating the range of equivalent fatigue cycles relative to the peak response for that record. Now the slide that's on here under slide 27, basically what we're doing on the left-hand side under C=2 is comparing the reinforcing steel fatigue for the Canterbury earthquake sequence along with what was recorded or computed should I say back in the late 1990s and you'll notice that the two graphs are plotted to the same vertical and same horizontal scale. Now of particular note I might say is if we consider one cycle, sorry one second period, where the cursor is on the right-hand side on the black, the El Centro line crosses over here at about four cycles and so perhaps that's where the four cycles come from, that are mentioned in the code NZS 4203 and if we look at what we get here for the Canterbury earthquakes it's a little less but quite similar, about three and a half cycles. So in many respects there are no surprises in these earthquakes, one cannot say that they are quirky and different just because they happened in New Zealand. They are the same as geology in other parts of the world, both, there's earthquakes listed in there from Mexico, the United States, Japan and also in Bucharest, and now all are showing similar trends.

Q. Just pause there for a moment professor.

MR RENNIE ADDRESSES JUSTICE COOPER:

Your Honour just for the record there is this sequence of slides a slide 15 and I mention it only to dispose of it. It's there by error, it should not have been part of this presentation.

JUSTICE COOPER:

Headed 'Fatigue – area equivalent.'

MR RENNIE

That's my understanding, that's right. It should just be struck through Sir, apologise for that.

EXAMINATION CONTINUES: MR RENNIE

Q. Now we go to, "Significance in the context of NZS4203."

A. It was considered to be of particular importance to compare the cumulative fatigue demand to the design code requirements as in NZS4203. Therefore in the subsequent analysis the reference amplitude used for the cyclic counting is the spectral displacement amplitude from NZS4203. In spectral displacement from the code based on zone B, soft soil for Christchurch with a ductility factor of mju =

4 from figure 3 of NZS4203 that should read, and a uniform force reduction factor of R=, that's the force reduction factor equals the ductility fact equals 4, as per Newmark's well known equal displacement factor. Taking peak maxima it only yields some equations and we've managed to do this across the spectrum and that's how the next set of graphs have been normalised so I think we are going to talk about –

Q. We now go to slide 19.

A. Slide 19 thank you. So just a reminder, we saw this slide this morning. This is the Darfield earthquake first of all, showing the –

Q. Did you want to go to 18 first?

A. No, this one's fine. Oh yes, 18 should be – okay thank you. Well this is the Christchurch earthquake and the next one that – or the one that we've just looked at, slide 19 is the Darfield earthquake. Now all of those, what we've done is analysed and aggregated the results all the way through, and if we go onto the next one which will be 20 I think. Okay, this is the same again, it's just all drawn on the same scale so we can pass by that thank you.

Q. Twenty-one.

A. So the next few slides are a review of recent evidence from Heywood and Frost focusing on the beam column and beam slab failures. I just wanted to put this is in as an interlude here basically to stress the importance on the vertical motion effects that the floor slabs generate onto the rest of the structure. The hypothesis here is that the very high vertical motions greater than 1G left the slab beam connections broken. That word was raised, it is my word, I chose it, I guess engineers are entitled to coin phrases. The Canterbury term which is probably the more formal one we should use here is that they're just munted. Note that the slabs in the vertical motion in the general, in general had higher vibration frequency of about a quarter of a second.

Q. Twenty-two.

A. Now Mr Heyward, this is his diagram, it is not mine. He took, extracted from the construction drawings provided by ARCL, this piece and then he drew the red line on here showing the typical failure surface observed as indicated and it is my view that the vertical excitation and hence the fatigue effects are particularly critical along that red line. The stresses so generated would be quite high but repeated loading will eventually cause delamination at that red line location. Most probably prior to the February earthquake and then most certainly in the February earthquake because of that pre-damage done would have most likely made the slab become airborne with respect to the columns. Next slide please.

Q. Twenty-two.

A. And we can see this clear failure plane –

1505

JUSTICE COOPER:

This is slide 23 Mr Rennie I think.

EXAMINATION CONTINUES: MR RENNIE

Q. I'm sorry Sir.

WITNESS REFERRED TO SLIDE 23

A. Yes and we can see the, in this figure here we can see the, the broken nature of the connection in here. It's very rough concrete which means that the aggregate is broken from a tensile fracture and disengaged the slab from the beam supporting it beneath. Beams and T beams such as this would normally rely on a measure of composite action between the slab and the beam. With this brokenness, if we call it that, we're going to sever that integrity and the two will start operating independently, and that is one of the reasons I believe that there was sufficient eyewitness evidence to show and continually comment on the fact of the liveliness of the building, and it was notably worse after the Boxing Day event, again stressing that the Boxing Day event was probably had a much larger verticals and horizontals directly beneath Cashel Street.

Q. Thank you, next one?

WITNESS REFERRED TO SLIDE

A. This is a slide from Mr Frost who basically says the same thing. This is another beam here. You'll notice that he's showing the wings that have been clipped on the end of the beam. The bars have been bent out. I don't think we can say that those bars got like that necessarily from the earthquake but rather from the fall. But it's very clear if that landed on its side as you see there, then the wings probably broke when they were in place rather than because of the fall. Next slide please?

WITNESS REFERRED TO SLIDE

A. And a final one here looking at the top, you can see that his concrete is you wouldn't exactly call it polished but there's a lot of dust and fine aggregate there which typically is evidence that this has been working pretty hard, and when you have concrete operating like that it tends to grind itself down into a powder, and so you get this very powdery surface due to the damage that's largely due to continual and repetitive cyclic loading. Next slide?

Q. Now I think in that point you, it's 25 isn't it you just had. You want to go to Mr Frost's notes, WIT.FROST.0001.23?

WITNESS REFERRED TO SLIDE

A. Thank you, the comment here was when I first read through all this, all the material on the website I basically read everything but I did, the significance of this page did not sort of hit me until about a week ago when I reviewed it after he had given his evidence, and I thought this was worth bringing up again to emphasise a very important point here, because it seems that maybe Mr Frost and myself perhaps more than others have, we're not out on a limb I don't believe, but we're out there saying that there is significant effects from the vertical motion and what we've done is looked at the empirical evidence on the ground and noticed some things. Now you have to look at the very top here, that Mr Frost noticed this Sunday the 22nd of February. Okay, this is fresh in his mind and it's worth mentioning that Mr Frost is a construction engineer. He's a man with, very used to having boots on the ground and dirt under his fingernails and he would not be afraid in going into a situation like this and analysing it from what he saw. That's the natural thing that that class of engineer does. And so he gives his impression, and it's probably worth mentioning what this says. "My impression, " right down the bottom of this page, "Collapse mechanism initiated by either one, slab failure non-ductile bond to tray deck or column or column beam joint failure, also non-ductile." Can we have the next page please? Then he goes on to say, "95% of beams are completely separated from column components. Slabs separated from all interior beams," and he emphasis "all". "Slab separated from most exterior edge beams. No tray deck, bond deck, metal found still bonded to slab. All interior columns 400 millimetres diameter. Exterior columns…," I can't read that, "…report to have been square or rectangular. I saw none on site since arriving on Wednesday night. Right along the southern side of the building slabs and edge beams have collapsed almost vertically, little or no north-south drift. At northwest corner of slabs and edge beams found several metres north of original edge of the building," and so he goes on. And then the final one is, "All strong evidence, slab collapse started near interior columns." And we'll come back to that later. So he's quite emphatic about that I think and I tend to agree for different reasons. I wasn't there on the site. I've analysed it and that's my conclusion.

Q. Thank you, so to the brief, slide 28, reading from the words, "Figure 3 represents the equivalent number…"?

A. Figure 3 represents the equivalent number of design amplitude cycles for the Canterbury earthquakes. Three curves are plotted on each graph. The lower blue curve represents the number of NZS4203 design demand cycles experienced as a result of the Darfield earthquake on 4th of September 2010. The mid red curve represents the total damage done by the Darfield earthquake, plus aftershocks, plus the aftershocks prior to the 22nd of February. I might add that there is quite an increase there and much of that increase, but not all of it, would've come on the Boxing Day event. And the upper green curve includes all NZS4203 equivalent design demand cycles for a structure to survive the Christchurch earthquake of 22nd of February 2011.

Now just holding this slide and referring to specifically the one relating to reinforcing here at the right-hand of these two. If we look at the gold standard of the day as given by NZS4203, the four, magic four cycles, what do we have? Well the blue which is September is about three and a half. Now in my book, Mr Holmes claims that this is not important, but in my book this is pretty similar. It is a very similar earthquake in terms of magnitude, as we mentioned this morning, and it's also a very similar earthquake in cyclic demand to the design earthquake. I don't believe that, I think we can, we can confirm and continue to say that this essentially is the same as the Darfield earthquake and the design code. Now if we add to that the damage, now remembering that subsequent cycles in terms of drift and sway are going to be smaller than the Darfield event, up until we reach February, it's still doing damage and it doesn't do a lot of damage for this small level of shaking, I mean this, this period. In this period it goes out to about four and a bit cycles. And then when February comes along there's a massive jump up until about 18 cycles per one second. Now remember I've also mentioned earlier that the number of cycles of loading are somewhat dependent on the period and it's fairly flat across here, so you might say that the amount of damage done was about four cycles prior to the February earthquake and then if the building was to survive the February earthquake it would be expected to have survived a total of around about 20 cycles of loading. So four into 20 goes five times. That's like saying five equivalent earthquakes, the structure would've had to experience five equivalent design earthquakes the structure would've had to experience to survive and remain standing at the end of February. And again I emphasise, it's not surprising many buildings in Christchurch have had to come down because of this. Like they were never conceived nor designed for this level of cyclic loading. Now if one was to look at the short period range where the structure is really being hammered by this high frequency vertical effects the number of cycles is markedly higher no matter how you do the counting this is up around about 30 for the steel effects. This probably wasn't affecting the steel too much. It's more affecting the concrete and if you were to norm it back this almost sounds outrageous but nevertheless it's consistently done with respect to what one would have expected by design it's in the order of about 200 cycles. Now I need to emphasise these are design cycles as per the design code. So I think we're going to move on now talking about C = 1 is that right.

1515

Q. That's correct.

A. For C=1 which is really the left-hand graph and I want to emphasise this this pertains principally to concrete critical fatigue where concrete wears down under repeated cyclic loading. In figure 3 the upper graph is for the value of C=1. This places the linear weighting on all amplitude cycles such that two cycles at half the design amplitude equal one design amplitude cycle. Thus the graph for C=1 is used to determine the effective cyclic demands on components that possess a constant reserve of energy absorption capacity. This relates to damage to the components where the concrete is prone to failure as per Dutta and Mander 2001. In the context of the CTV building it is the connections that were prone to failure due to damage to the concrete. In particular the slab to precast beam connections and the beam to column joint connections are elements that are prone to concrete failure as pointed out by both Mr Frost and Dr Heyward.

For the slab systems in the CTV building the vibration period is thought to be in the order of about 0.3 seconds thus from figure 3 this implies that from all earthquakes prior to 22 February some 85 New Zealand NZS4203 code demand cycles would have been experienced by this structure. It is thus not surprising that the floor slabs felt quite lively by occupants clearly evident of repeated cyclic loading damage.

The beam column joint regions where the concrete was expected to be the sole mechanism to provide shear and bond resistance also experienced significant cyclic demand. Given the effective sway period would be in the range of one second it is evident that prior to the 22nd of February Christchurch earthquake some 20 cycles of loading would have invariably caused damage to the beam column joints. Note that this demand exceeds the design expectation of four cycles by a wide margin.

For the CTV building to survive the 22nd of February Christchurch earthquake there was a considerable additional cyclic demand. Given the effective period during this event would have shifted to 1.5 to two seconds the total cyclic demand would be in the order of 30 cycles. With such demand it is not surprising the joint zone concrete showed complete destruction being pulverised to dust due to this repeated action.

Now Mr Frost I believe and Dr Heyward also mention that they were unable really to find any evidence of joints that remained anywhere and when I had a look around the landfill site out at Burwood this was basically the same. You go hunting and looking for crumbs of dust from joints and you just can't find anything but you do see quite a few connections to the bars separating the elements they connected but no evidence of joint concrete.

So that's the concrete and then we move onto C=2. I'd like to emphasise that that's the right-hand graph there on the screen. C=2 is the reinforcing steel fatigue. When C=2 a cycle with 50% of the design amplitude is assumed to provide fatigue demand equal to one quarter or 25% of a full design amplitude. This conforms to low cycle fatigue capacity of reinforcing steel as given in Mander el al and Dutta and Mander. Reinforcing steel in plastic hinge zones are prone to premature low cycle fatigue failure. In the context of the CTV building therefore plastic hinges were observed to initiate in the structural walls.

Moreover other buildings with more conventional ductile detailing are reinforcing steel fatigue prone. It is therefore not surprising that other well designed and well detailed buildings have been condemned because of their uncertain remaining fatigue life. The ARCL design IRD building would fall into this category where some 20 full design amplitude cycles of loading would have been experienced as a consequence of earthquakes up to and including the 22nd of February Christchurch earthquake.

In the case of the CTV building a critical region where the reinforcing was prone to low cycle fatigue was the beam column joints. This is because the concrete through the joints relative to the surrounding elements was relatively weaker leading to very high strain amplifications in the column joint steel. From figure 3 it is indicated that some 20 cycles of NZ4203 code demand would have been experienced by the critical reinforcing steel.

It should also be noted that one cannot easily observe evidence of fatigue damage. Although cracking may be an indicator of a fatigue prone location the crack size cannot be used to infer the extent of fatigue damage. One must conduct a rational analysis using the principles or mechanics to understand the extent of fatigue damage.

And then finally C=3

For the sake of completeness for C=3 a cycle with amplitude equal to 50% of the design cycle would be deemed to contribute about 12.5% of the design cycle damage. This category is applicable to the fatigue damage in the steel components.

Now some implications of these results. In accordance with NZ4203 if one assumes that there are to be four cycles in the design spectral displacement amplitude via the structure capacity then a capacity versus demand evaluation can be made.

The CTV building was exposed to cyclic demands considerably greater than what one would expect to observe back at the time structures were designed in the 1980s. Three implications arise:

First older buildings could not be expected to survive the cyclic demand exposed prior to and during the 22nd of February Christchurch earthquake.

Given the forces that the building experienced on the 4 September 2010 earthquake and followed in close proximity by a significant aftershocks, it would have been prudent for all concerned to have been suspicious about the ability of the CTV building designed as it was since 1986 to have withstood the force of the 4 September earthquake and immediate aftershocks without a material loss of fatigue capacity in fatigue prone regions such as column bars and also its associated loss of strength in the concrete damage prone elements, in particular the beam column joints. Those suspicions could only be allayed by the performance of a structural analysis. Now this doesn't mean non-linear time history analysis because even for that you have to do post-processing. This is just a simple back of the envelope analyses that a structural engineer familiar with this area would be able to carry out.

Q. Would you reference for example plan specifications or prior calculations for that work?

A. Absolutely and one of the things that is most crucial particularly for the CTV would be to make a look, take a look at the plans. One could go zero in on the bases of the wall because really that was the part that was intended to be the primary energy dissipation component and then take a look at those locations, do some non destructive testing if necessary. If there's loose concrete in the plastic hinge zones you can dig out the cover, take it off, inspect the bars and then if necessary like if it's deemed to be okay you would just grout it all up a bit. In the case of the joints that's a little more difficult because the joints for the CTV building essentially were concealed. They were concealed by the fact that we have this circular hole in which the weaker concrete was poured. In the process of pouring that concrete it would have shrunk slightly and so you would get a very fine natural crack line around the end of the shiny beam ends that some have commented is not all that great, well it's possibly not all that fantastic but I think more of the problem is that the concrete on the inside shrinks a little bit and therefore any cracks that may have been in that very inner sanctum of the joint core are basically concealed from visual observation, but again an astute person who had the plans would have noticed this and inspected accordingly. Now, again I come back to what I said this morning. The human body is an absolutely fantastic motion sensor and this dates back to the good old days from when I was in the Railways. I would often hear wheel tappers going down with their hammer, pinging rail ties looking for cracks and these are fatigue cracks basically they are looking for. Now with concrete structures it's similar but you can use a classical engineer's type ball-pein hammer and then you can sound the concrete by hitting it and if you get a dull thud that typically means there's been some separation and worthy of a second look. There are ways of doing this just by simple observation and using it. We typically refer to these as ENH methods – eye and hammer, very simple, but remember that the human ear is again a fantastic motion synthesiser and you can understand ringing noises quite easily and it will have been possible to do that. I don't know if that was done by Mr Coatsworth. I suspect not.

Q. So to be clear the structural analysis that you are pointing to in the second point in your three implications – plans are, what, desirable, highly desirable, essential?

A. Absolutely essential to do this because one needs to know where to interrogate. You cannot do that necessarily by looking from the outside. Like, for example, I believe that Mr Coatsworth did his inspection in good faith, that the building probably was a normal sort of building and normal in his mind may well have been a well detailed ductile building but looks can be deceptive. It's a bit like being caught up by a cop for speeding down the road and you take a breathalyser test and it doesn't register as being bad and so the cop let's you go. The cop may have failed to do the blood test and so in this case, but this is not human, these are inanimate objects. They are structures and just let's assume that these things are guilty until proven innocent. The blood test was not done.

1525

Q. The significance to this structural analysis of access to Council records?

A. The fact that the Council wasn't there, didn't have them available, probably deserved the time-out and some further exploration as to where one could procure those records. I would be very surprised if nobody would have had the records and normally you would go back to the engineer of record who designed the structure who would have a repository of the plans and just used those. Anybody, any engineer in that situation is going to happily hand them over.

Q. The significance for this structural analysis of access to prior calculations?

A. They may not be helpful. In fact they might be a distraction. It is probably best to just take a fresh look. Again I point out these are not rocket science. I refer you to the three references I have there and in particular the ASCE one which is the first one there which gives a very simple energy based methodology. It's a relatively new theory but there are graphical explanations of what's going on. If an engineer doesn't understand that, they can read, and they can read what this is about and it's not going to take very long to do and they can look accordingly. If there's more than four cycles and they think this has gotten out into the danger zone, they should have been more cautious.

Q. And now to Slide 31 and your third implication.

A. Yes, third and finally building survival to the excessive demands of the Canterbury earthquake sequence can only be attributed to a measure of overstrength that exists in structures where the in situ strength exceeds the specified capacities by design. Ductility is not a substitute for strength, as a design concept it is shown to be wanting.

JUSTICE COOPER:

Q. Just in relation to this paragraph numbered 1 on the last page of the second brief – "Older buildings" – What are you referring to by the use of that term?

A. Well maybe I should modify that to newer buildings too. Like if IRD has now gone and I don't think I knew that at the time I wrote this and I was probably operating under the presumption that newer structures are going to be detailed a little more robustly but given the fact that this particular generation of building there were quite a few designers around Christchurch that took the liberty to use this kind of dual system where one half of the structure was seen to be earthquake resisting and the other half was seen to be going along for the ride, pulled along by drag-bars and remaining elastic, clearly in hindsight that's a lot to ask and so that should have been realised. My guess is that that should have been realised and knowing that there are those deficiencies and given the fact that they were able to survive, the design event which I maintain is Darfield, that doesn't mean to say that they will survive it again or a bigger earthquake.

Q. So what is the category of building to which this approach should be applied?

A. Well I'm saying particularly older buildings –

Q. – Yes well I've asked you what you meant by that. Now could you tell me?

A. I would now say that that is probably anything older than the present codes.

Q. So anything designed under a code earlier than NZS1170.5?

A. Well actually no. I will change that. I will say including 1170 buildings. It just occurred to me one thing and that is in the first reference I come up with a quite scathing comment, that got published in the ASCE because the reviewers obviously saw it as being correct, criticising the old NZS3101 which was the second edition I believe, 1995, where the code requirements for the amount of transverse reinforcement was relaxed from the original one that was in the original NZS3101 1984. The second edition the column confinement requirements were relaxed for transverse reinforcement for elements with low axial load. That remains, Commissioner Fenwick would be very aware of this because that remains in the current code, and he and I were both on that Code Committee, and it remains a concern to me that as fatigue resistance even of modern buildings is not sufficient because this has not been taken adequately into consideration. So maybe I should just say all buildings as evidenced by what has had to come down.

Q. So all buildings in Christchurch should have been subject to the process that you refer to in paragraph 2 – all commercial buildings in central city?

A. It's a relative thing and it's a guilty until proven innocent. I think one could –

Q. – Well what's the answer to my question?

A. I think it would be wise to take a look at all but some will need more scrutiny than others in particular –

Q. Which are they?

A. The ones such as the CTV building that have these dual systems where you have some of the structure designed for ductility and were detailed for ductility and some of them not so part of it not. And again, as I mentioned this morning, people like Brian Bluck probably have known those sort of structures off the top of his head based on his experience.

Q. Well he's dead isn't he so I'm trying to...

A. I understand, I understand. This is often about corporate memory and the continual memory of what's out there. It's very very important in my view.

HEARING ADJOURNS: 3.34 PM

HEARING RESUMES: 3.51 PM

EXAMINATION CONTINUES: MR RENNIE

Q. Now Professor Mander, before the break His Honour was asking you about the first of your three implications and the expression 'older buildings.' Do you recall that?

A. Yes.

Q. Just to try and amplify what you were saying there a little bit, in terms of buildings are you talking about all buildings, are you including residences, commercial, industrial, what do you mean by a building?

A. Well more specifically engineered structures such as concrete and steel buildings, and to amplify on that question I really like what eventually was done in Christchurch, I thought it was really a smart move to not necessarily address every single building but to have several so-called indicator buildings scattered around the city and in that way one can examine specifically the progression of damage in those indicator structures and if they tend to show a lot of damage then you can say all other structures that kind of fall under this class of structure should be looked at more closely. If they came through unscathed then you – I'm sure that there's good reason to still be cautious but you would put less priority on re-examining those. Now if those class of indicator buildings are not available, then another alternative would be to essentially again consider all buildings guilty until proven innocent and then what you then need to do is to go ahead and triage perhaps all structures and you could bin them by putting them into an expectation of mild, medium or severe damage based on what the knowledge, current knowledge is of the class of detailing that one would see in that class of structure, and of course the ramifications of any particular class of failure that might lead to sudden collapse. Those are the ones that you would really need to look at very carefully very quickly, and there is though one class of you might say non-engineered structure that of course is always problematic and that is those are masonry buildings where you can have falling bricks and façades that are often a danger to human life. They have to be taken a particular care and regarded with a measure of caution, because walls can fall down with fairly modest level of shaking, quite inexplicably.

Q. To be clear by bin you mean to categorise or –
A. Yes.
Q. – classify, not to discard.
A. Yes.
Q. And when you spoke of concrete and steel buildings, am I right that you mean concrete buildings and separately steel buildings?
A. Yes, yes, like there's a common perception that steel buildings are immune from damage, but this is found to not be the case. There's quite a lot of evidence of damage to steel structures in Christchurch. After the Northridge earthquake many engineers thought that steel buildings were superior to concrete structures but only to be haunted some two months later when damage was discovered and then it was found to be quite

widespread but hidden in the beam column joint connections, often behind fireproofing or often behind concealed ceilings and they hadn't been inspected but then on closer inspection the problem was found to be quite severe so – and it was kind of interesting to me that Mr Kehoe said, "Well you don't have to take a second look at these, the damage is not going to get any worse." Well if, they were lucky, because they did not get a second large or larger series of aftershocks. Had they received that in that area of Los Angeles then the longer term outcome may well have been quite different.

Q. I wasn't proposing to take it further than that sir. We now go to section 3 of your first brief at page 38, "An alternative gravity dominated collapse scenario," do you have that and this is the third powerpoint, slide 1.

A. Well I would just like to start by saying that I chose this to be gravity load dominated, simply meaning that the structure is more or less going to be coming straight down as opposed to side sway where the implication from the DBH report was that it was a lateral side sway and I felt that the analysts were somewhat remiss in not considering this. Of course having lived many years in the United States and having lived there in the same state as the World Trade Centre when it was first attacked in the early 90s one can only be too aware of what happened in that particular structure and in the case of that particular structure we all know that planes flew into the building, the building eventually collapsed downwards but it was a stability type failure and many people have tried to attribute different things to it, but perhaps the most famous analysis was made literally three days after by a notable professor from North Western University, called Professor Bazant, where he wrote about a three or page note to the Journal of Engineering Mechanics, of the American Society of Civil Engineers and pointed out what he thought the collapse hypothesis was for that building. Now when you go back and look at that, it's strikingly similar to CTV and I was aware of this of course because I think engineers need to be from an educational point of view when they educate students, need to be aware of all the different types of classic failures that one might see out there, and this was indeed a classic failure, and it's also very similar to how one would implode a building. You typically take out the fittings and then you go into the vulnerable elements and put explosives on them such as the columns and near the connections, and then you blow the building up and it typically comes down in a ratcheting type fashion from top to bottom, there's very little spill out to the side and that's clearly a gravity collapse and when one looks at the photographic evidence of CTV and then also when one reads something like both Dr Heywood and Mr Frost wrote, it's very clear that this is not a side sway type of failure, and I think this is also in agreement with what Mr Holmes and Professor Priestly are saying, although they haven't quite come out and overtly said it such as I have, that this is a vertical thing which one can ascribe to classic stability and I might add that the computer programmes that one would use to do this such as sap2000 is currently being used, really, really struggled to do this sort of analysis where you actually use what's called geometric non-linearity where the deflections become quite large and then that drives the collapse. One would really need to resort to an alternative type of computer programme to capture this more effectively. It's known as the discrete element method, and there are researches in Japan that have developed programmes exactly for this sort of thing. So with those introductory remarks I would just recap that I'm not using this to dismiss other possibilities that may include elements of side sway, that could have been present, but I really feel that this is a possibility that should not have been dismissed, and it seems that it may not have even been considered and so that's where my starting point is.

Q. Thank you professor and if you'd now read from, 'During the Canterbury earthquake.'

A. During the Canterbury earthquake sequence commencing with the Darfield earthquake and leading up to the Christchurch earthquake, substantial damage had already been inflicted upon structures throughout Canterbury. The CTV building in particular had already met or exceeded the seismic design limits of its structural system. In the design of the CTV building the design engineer chose to transmit all seismic inertia forces accumulated by the mass distributed throughout the structure, back through the floor diaphragms to the shear walls and then in turn to the foundations. The remainder of the structure was detailed principally for gravity loads, and a check was made that the principal gravity load bearing components, the columns, were not put under excessive side sway displacements for the designed seismic loads.

It is evident however that the first earthquake in the sequence, the Darfield earthquake, exerted inertial loads that were either met or exceeded the design expectation. The Darfield earthquake, similar to the Christchurch earthquake, also had very high vertical motion acceleration components. Historically by design vertical accelerations have been expected to be about two thirds of the horizontal components. This was roughly the case for the Darfield earthquake but only over a relatively narrow frequency band. For high vibration frequencies greater than about three hertz the vertical acceleration components were exceptionally high, considerably more than the normally expected two thirds of the horizontal components.

These exceptionally high vertical accelerations tend to vibrate the vertical load bearing elements such as the columns and the floor slabs. While the exceptionally high vertical motions were not the sole cause of failure they certainly added considerably to the resulting damage. It is for this reason that people did not want to work in the CTV building. They were uncomfortable with their work environment. The slabs in particular were evidently not behaving as they should have by design, and it was for this reason that the CTV building should have been red stickered. The liveliness of the CTV building was the primary evidence that the structure had damaged connections, and that the CTV building was ill-prepared to survive further shaking, in particular an earthquake that was greater than another design level event.

As eyewitnesses from both inside and outside the building reported, when the Christchurch earthquake struck initially the CTV building swayed violently in all directions. After several seconds of violent shaking it seemed as though the structure had come to rest and then collapsed, and there's some eyewitness, I mean some eyewitnesses sightings. Although the H-S report is rather vague in its conclusions, it does allude to a collapse of the CTV building initiated primarily from side sway motions. The supporting Compusoft analysis was strictly unable to arrive at any other result because the dynamic hysteretic moment-axial load interaction effects were not properly modelled in the Compusoft analysis. For example the computational model simulations were unable to capture the possibility of a classic Euler type buckling of column failure due to the column compression overload induced by the exceptionally high vertical vibrations. Furthermore, the connections between structural elements were modelled as rigid blocks. This is a customary approximation made in design-based simulations for a forensic analysis when demonstrable damage of the beam column joints was clearly discernible, the assumed simplification was not sufficient.

In the remainder of this section 3, alternative collapse hypotheses will be presented. Where appropriate the hypotheses draw from the reported data in the H-S report along with eyewitness evidence to arrive at different conclusion. The analysis does not rely on the faulty assumptions inherent in the original H-S report. The specific erroneous assumptions were that the concrete as built was substandard and that the beam column joint zones were rigid. In contrast the alternative collapse hypotheses used rational mechanics supported by eyewitness statements to deduce a type of behaviour that conceivably occurred which led to the collapse of the CTV building.

It should be noted that this collapse mode is not a radically new idea. Mr Holmes points out in his peer review of the H-S report, could we bring this up sir?

1601

Q. We could do that, BUI.MAD249.372.9

WITNESS REFERRED TO SLIDE

Q. Is that the reference you wanted or was it the one, there's a later reference on 14?

A. Page 14 is it?

Q. It's 0.14 at the top if we just go to that perhaps and just see if that was the one you wanted?

A. While that page is there I notice the second paragraph from the bottom it says, "The proposed change in emphasis represented by the proposed collapse scenario is important because column hinge failure, thank you, would logically lead to a side sway collapse and all evidence points more towards a more vertical collapse mode. The classic vertical collapse mode of columns including squashing or shear failure are not evident, nor indicated as probable by quick calculations, however the three possible results of joint failure listed above could certainly result in sudden and mostly vertical collapse."
Now I read this of course after I'd done a lot of my own work, so naturally it immediately resonated for me.

Q. We might just by page 14 also while you're on this point? Or 13, yeah it is page 14.

A. I think it's the top paragraph is that right?

Q. Yes it's the top paragraph.

A. Maybe we could have that enlarged so I can see it? Thank you, "However the leaning configuration of the lower slabs could be explained by a disconnection at level 3 that caused the sudden increase in drift ratios in level 2 and or level 3 to 4 frames." Now I postulate something that's in the same vein as this but perhaps for slightly different reasons, but it's not at variance with what Mr Holmes proposes. He believes that this would have initiated a collapse, presumably the disconnection at level 3 would also cause a gravity collapse at the face of the tower, but that collapse could've been arrested by level 2 slab. "After the complete collapse occurred at line 3 the slabs could have taken the configuration shown in figure 165. Alternatively the disconnection at level 3 could have allowed for large north-south lateral movements without a complete gravity collapse along the face of the tower also creating the configuration shown in figure 165." So I guess basically why this reminder was put in here is that this is not, my thinking is a parallel universe you might say, it's not at variance with what Mr Holmes says. It's a very subtly, slightly different point of view that more has to do with the nature of what happens after the trigger takes place.

Q. Now coming back to page 39 of your evidence, just after the reference reading, "Holmes also rightly points out…"?

A. Holmes also rightly points out the deficiency in the modelling of the joint strength and the dependence on side sway as an explanation of the failure mode. He then goes on to propose that a collapse mode over more than one storey – now he contended that he didn't say that but that was my reading of it and I kind of still think that that's the case, but even though he didn't, I respect the fact that he didn't mean that. I don't think it alters the outcome too much and we'll see why later. Holmes stopped somewhat short of completing the solution but it is considered that he was certainly heading in the correct direction.

Early in the Christchurch earthquake there was a substantial velocity pulse in the north-east south-west direction. The velocity pulse was about 0.7 metres per second, and due to its diagonal orientation with respect to the north-south facing building this pulse would excite the structure of the CTV building in both the east-west and north-south directions. The collapse mechanisms are considered by decomposing the overall ground motion effect into each of the two orthogonal directions, east-west and north-south.

An alternative collapse hypothesis is first examined by considering the motions in the east-west direction from which it is shown that previous damage along the west wall, as well as inadequate lock in details of the east-west beams into their seats, led to the unseating of those beams along line A. This eventually led to a subsequent overload of the neighbouring columns. Those neighbouring columns would've been overloaded in axial compression, especially when considering concurrent vertical vibrations arising from the exceptionally high vertical accelerations.

The second part of the collapse hypothesis considers motions in the north-south direction. The north-south direction has gathered much discussion by others which all refers back to the perceived inadequacy of the drag-bars. The lack of, or failed drag bars would be affected by a northward pulse, where the inertial forces are directed south causing the floor diaphragm to pull away from the North core. It will be hypothesised however, that the opposite action is also likely -– that under northward inertial forces the floors may “crumple up” in technical terms, buckle downwards. This leaves sufficient movement room so floor slabs from one floor to the next can move such that the columns to take up a buckled shape over four-floor levels; when coupled with excessive vertical overload, buckling of the columns ensues, along with a global collapse mechanism.

There is a corollary of the abovementioned northward motion induced collapse. A similar collapse mechanism occurs due to southward movement of the floors. Because of the absence of drag bars in the lower storeys, the floors are somewhat free to move away from the north core permitting a buckled shape to form.

It should be noted that in both cases the formation of the collapse mechanism is in three parts, and that's essentially on the slide here. First there must be an action that leads to a trigger, this leads to incipient failure or the first part of the failure, and finally there must be a statically admissible mechanism that can form that lead to the collapse mechanism proper.

So getting into the collapse mechanisms, 3.2.

Q. (overtalking 16:12:49) which is slide 3.

A. Collapse Mechanism in under East-West Shaking.

The trigger – 3.1 and that's the, it's really the top part up here, 3.1 presents the sequence of events that led to the trigger action. According to the Compusoft results, the effects of the large velocity pulse as recorded at the CCCC station would be felt from about 4.5 to 6 seconds. Although the veracity of the Compusoft results are questionable for various reasons already stated, it serves as an interim indicator of the displacement demands experienced. Here inter-storey drifts of about 3 percent are indicated for all floors, that's bigger G5 in Compusoft, or in other words a differential movement from one floor to the next, either above or below, of some 100 millimetres.

1611

Q. Just pause there for a moment professor.

MR RENNIE ADDRESSES JUSTICE COOPER

For the record Compusoft figure G5 is ENG.CRM0001.122, I'm not going to go through it but that's the reference.

EXAMINATION CONTINUES: MR RENNIE

Q. Thank you, now if you keep reading please professor.

A. I'm going to read directly from the figure because I think the comments relate directly to the little cartoon directly to the right.

So stage 1, the building sways to the west with a large velocity pulse. The east/west beams on column lines 2 and 3 at the west wall are required to form large negative moments that cause the joint core concrete and the beam soffit cover concrete to crush. So that's down in here in the purple colour, this crushing.

Stage 2, during the next half pulse the building lurches eastward. The beams along line 2 and 3 pull away from the west wall and their line A column seats to form the alternating positive moment. The crushing cover concrete from the previous reversal spalls off and the beam slumps down a little, with a partial or full loss of seating. Due to a loss of seating at the support line A there is a transfer of the previous gravity load from the tributary area of the beam onto the neighbouring columns on line B. This action an axial force increase of up to 40 percent on the columns on line B results. I will point out that this type of failure that we 30 years ago when I was at the University of Canterbury we tested a lot of hollow core floor slab systems and this is the class of failure we saw. The pre-cast concrete elements were seated on relatively narrow sills and here they're only about 20 or so millimetres and what they do, because that concrete can be quite strong, is they more or less act like a bottle opener under opening up and you get this break out of concrete and you lose the seat and you'll notice the red piece of the bar here of the middle diagram, is not anchored in very deeply and this is because the wall itself is 100 millimetres narrower than all other locations and so the seat here is quite poor in terms of these large displacements that one might expect to see.

So that leaves, once this reaction force R vanishes towards 0 and eventually will become zero and as the building attempts to return to an upright condition by moving west, the unseated beams are inhibited from fully returning due to the presence of the west wall so they bump into something and then, essentially makes everything unstable in that vicinity.

Then if we move to stage 4. What we have discussed is the first two parts, the trigger and then the incipient failure. This is the incipient failure at the end of stage 3 and then stage 4 this sets up the way for a mechanism to perform. Permanent differential deformations remain, that inhibit the columns along line be from remaining straight. This sets the columns up for a classic Euler buckling type failure, especially under further axial load derived from vertical accelerations and their consequent vibrations.

Now you'll notice that there's kind of a dark red in the figure and then there's also a lighter red, it actually comes across as pink. The dark red is where you get buckling between essentially the two joints and the other point that I need to emphasise here is that the joint up in the floor above, at the fourth floor, can be quite intact, we can get a moment in here, because this becomes a pin essentially through here, simply because the concrete has deteriorated in the joint. Through here although the concrete may have deteriorated the chances are that it still can take some moment because the shear in this vicinity is zero, the shear demand is zero, so it doesn't matter if the capacity has more or less vanished to zero, but the moment capacity by virtue of the bars remaining intention plus some concrete on the compression side will permit this to perform or provide some positive moment and then, but this is the weak zone in here, this whole thing will bulge out and then collapse downwards.

Now it could happen here at the second storey but it doesn't have to, it's not a requirement. This could happen only at the second storey where the second storey bulges out and becomes dislodged in against the end on line A and so you can get a buckling where the length that you would use in your buckling formula is down in this phantom node, down below the ground level so this is another possibility. So these are two complementary mechanisms. My guess is that the higher one may happen but the bottom one is also a high possibility and it's interesting in looking at the latest second round of non-linear time history analysis the results tend to point to a lot of the bad things happening actually on the ground storey down here. So maybe this one here is more likely to occur, but I think again this is debatable because of how the inputs have been put into the programme. It predisposes this column crushing or shortening to take place and it doesn't really permit this geometric non-linearity to take place so easily.

Now when you get this type of failure, you can see this strung out here from left to right, the left is the trigger, beam seat fails, you get this incipient buckling set up and then between the second and the fourth storeys and centred around this floor level here. Basically everything collapses down and in many respects this is quite consistent with the eye witness reports where the bottom storeys pancaked and then the upper two is where most of the survivors came from and that's because these two here, or these four here probably crushed first.

1621

JUSTICE COOPER:

Q. Just record shall we if this BUI.MAD249.05B1C.5?

A. The effect of such movement, we're on the top of page 43. The effect of such movement are led to the initiating trigger action are shown on stage one of figure 3.1 where the east west beams move eastward causing a large negative moment (which means tension in the top steel compression at the beam soffit) to form. The concrete at the beam soffit would be expected to crush, as well as the weaker concrete in the joints. On motion reversal towards the east, any crushed/spalled concrete is expected to break away as shown in stage two of figure 3.1. The reaction on the soffit in turn vanishes.

In stage three of the sequence, the reaction is instantly transmitted to the neighbouring column on line B. It is estimated that when including vertical motion effects there is a 400 kiloNewton increase in the axial load on the second storey level of columns. This effect leads to the formation of the incipient collapse mechanism as shown in figure 3.2.

The incipient failure as shown in figure 3.2 presents the formation of the incipient failure mechanism. For this to occur this needs to be a relatively small perturbation or inherent fault. In this case a small differential displacement over the floor height suffices. The mechanism consists of a column under double bending over all the four floors. This concurs with eyewitness statements and there are some examples.

It should be noted that for this mechanism to form, the demands on the beam column joints are relatively modest. For example, there is no moment within the joints at level two and four, while level three has high moments through the joint has essentially no shear. Calculations show that incipient collapse will occur once a differential storey movement of 37 and 42 millimetres equivalent to a single bending drift in this case of 1.15% to 1.3% for the cases of columns with concrete strength as used by in phase one and phase two of the NTHA records for analyses.

The collapse mechanism if the collapse is initiated at the west wall then it follows there is an eastward failure moving from columns line B to F. This explains why more debris fell near the Madras Street corner of the building. The collapse mechanism is presented in figures 3.3 and should be noted that the mechanism once fully formed will push the walls out first at level three at the east end along line F then secondly at level two the lower column will blow out due to the now very large displacements in the columns. This is consistent with eyewitness observations.

If we go to slide six this is talking about the, there are two complimentary collapse mechanisms here. We're calling these north south shaking. One is really a northward mechanism and one is the southward mechanism. The northward mechanism is first in the upper part of the diagram and the southward mechanism is in the lower part of the diagram. Now it's a bit hard to read on the screen so if we go to slide seven we'll see the northward mechanism.

Here the trigger figure 3.4 presents the sequence of events that led to the trigger action. From figure G3 of the Compusoft results the effects of the large velocity pulse as recorded at CCCC station would be felt from about 4.5 to six seconds with the interstorey drifts range from 2.3 to 2.5%. Coupled with this are substantial vertical vibrations in the slab arising from the vertical ground motion. Given the pre-existing damage that was evidently observed by eyewitnesses due to the liveliness of the CTV Building, it is possible that much of the metal tray-deck had de-bonded, with the floor slab going into catenary action. The vertical vibrations in the Christchurch earthquake would have caused further damage. And along with inertia forces in the northerly direction the combined effect led to the downward shape buckle (or folded plate), as depicted by the red curves in Figure 3.4.

Now some commentators have said that this is not a particularly likely scenario and I will concede that. It's not farfetched. It's definitely possible in my view but –

JUSTICE COOPER:

Q. Who are the commentators that you –

A. Well I think Dr Priestley made some points and remarks and probably in his words and I'm paraphrasing here that they were probably naively simplistic but I would maintain that with this violent shaking the bond between the tray deck and the concrete was essentially severed, most likely even in the Darfield earthquake and because that was not there and also –

EXAMINATION CONTINUES: JUSTICE COOPER

Q. Sorry what was not there?

A. It would have either literally fallen down onto the floor or maybe even into some ceiling space it's possible, but I believe that it's conceivable that it would have at least disengaged and then given the fact and we've talked about this at quite length, Frost and Heyward point to the evidence that the connections were broken at the supports of the slabs in the east-west direction then this is very easy to sort of flap up and down quite freely so the problem then becomes that this is got to take gravity. This is not ordinary gravity. It's gravity up to 2G downwards so it's doubling the downward load on a very thin slab where you've only got this mesh that's holding things up and we know that the connection at supports has been weakened anyway from this repeated loading so to me it's beyond the pale to suggest that this is going to flap up and down like this and then crush. It doesn't have to go very far. It's really a matter of making sufficient clearance so that the columns relative from one side to the other can move closer as shown in the green line here and of course the green line on the figure is highly, highly exaggerated but in reality that only has to be in the order of 25 to 50 millimetres, one or two inches is sufficient for this to happen. This is the small firm (inaudible 16:29:07) I talked about that all is necessary for some sort of buckling failure to occur.

WITNESS CONTINUES READING BRIEF OF EVIDENCE

Q. "The Incipient Failure.

The folded plate action would provide sufficient movement for the columns at levels 2, 3 and 4 to also translate northward, permitting a double bending buckled shape to be set up over the lower four storeys" That's the green line. "This is shown by the green curve in Figure 3.4. The calculations are similar to those in the E-W mechanism described above, except the extra beam weight is not added. Calculations show that incipient collapse would occur once the differential story movement of about 1.2% interstorey drift occurs.

The Northward Collapse Mechanism and it Southward Corollary.

If the collapse mechanism is initiated, it would be most likely along column lines three at possibly rows C or D." And I single out rows C or D simply because the northward bound diaphragm forces have to concentrate and get into the walls so because the in-plane forces are going to be higher in that location it's more likely that this will be the precipitator of the failure. It won't be general so like some have claimed that you have to have a whole row of columns go together. I dispute that. I think one or two is sufficient to get things moving. If the collapse mechanism has initiated it would most likely along column lines three at possibly rows C or D, so that's also in the vicinity of the stairwell. "Once these columns collapse downwards they release load which in turn must be carried by their neighbours. Consequently, the surrounding columns are also overloaded, bringing the entire structure down. It should be emphasized that the main reason this mechanism can occur is because the building possessed only one-way slabs that were beamless in the north-south direction. The in-plane diaphragm stiffness was consequently low, thus the slabs had a high propensity to out-of-plane buckling due to in-plane seismic loads. Again this downward out-of-plane slab buckling was exacerbated by the exceptionally high vertical accelerations.

1631

Q. Slide 8 please.

A. So section 3.3.2 - "The southward mechanism.

Firstly the trigger.

There is a corollary to the above-described northward motion-induced buckled plate/column collapse mechanism. Suppose a large pulse acts in a northerly direction, inertia forces act southward and the floor slabs are dragged away from the wall. Irrespective of the merits of whether the drag bars had sufficient capacity to restrain these inertia forces, the fact remains that there were no drag bars in the lower storeys. Such a lack of restrain permits the lower level floors to move relatively freely southward, especially at the eastern side of the building where there was a frame but no wall (as on the west side) that would otherwise provide some additional restraint.

The Incipient Failure

As the columns on lines 2 and 3 are free to move, they will form a buckled shape, as shown in the green line in the lower diagram of Figure 3.4.

The Southward Collapse Mechanism

The structural columns were the most heavily loaded along lines 2 and 3. Once one or more of these columns become overloaded and tend to collapse downward, the loads they previously carried needed to be transmitted to the neighbouring columns which in turn become overloaded. Once several columns are overloaded, a general buckling of all columns along a line develops, bringing the down the entire structure. The relative lateral movement, initiated by the pullout of the wall anchorage led to the general buckling mode of failure." So I just want to specifically talk about the diagram here. So you'll notice that in here the red line doesn't connect back to the wall. There's some sort of small gap in here. Again I want to emphasise this is highly exaggerated for illustrated purposes. This could be quite small and –

JUSTICE COOPER:

Q. Is that the level 3 floor?
A. Yes.
Q. Just try and use some words to explain.
A. Yes it could be on level 3, level 2 or level 3 it's most likely to be there.

Q. What I was going to say was please use words to describe where you're pointing the mouse for the record.

EXAMINATION CONTINUES: MR RENNIE

A. So this is level 2 on the top here so there could be a gap formed here. There were no drag-bars which are strong elements to help restrain and as Commissioner Fenwick has pointed out with his cross-examination on the shear effects, the slab steel in particular is going to be in peril due to east-west action. So, and as has also been pointed out by Professor Priestley, when one gets a crack perhaps as small as 2mm again that slab steel is going to be imperilled and that's really the only thing that's tying the floor slab on level 2 and level 3 back to the frame and with this gone it sets up this particular failure shape. Now again I want to emphasise that ideally you would want to have these to break free here but –

Q. The break through being –

A. At two and three but at level 2 alone it can still work because if this single floor comes across then you'll get this buckled shape down in the first storey and it could be sufficient to permit the bottom and that's the one of course with the highest axial load in the ground floor.
Okay, so the columns were most heavily loaded along column lines 2 and 3. "Once one or more of these columns become overload and tend to collapse downwards. The loads they previously carried needed to be transmitted to the neighbouring columns in which they become overloaded. Once several columns are overloaded, a general buckling of all columns along a line develops, bringing down the entire structure. The relative movement initiated by the pullout of the wall anchorage led to a general buckling mode of failure, this would be exacerbated by the very high horizontal accelerations.

So a summary of the failure modes. There have been three general failure modes postulated – a four-storey double bending buckling failure staring at the column Line B leading to the east-west collapse failure mode; a northerly motion induces collapse failure mode; and a southerly motion induced collapse failure mode.

Now what is common amongst all these three failure modes is that they require the same class of buckled columns over the lower four storeys and in fact it is conceivable that a combination of these modes could co-exist under torsional motion. So what I've talked about, and it's quite hard to draw this, but if you have sort of two floors twisting and kind of dancing the twist then it's quite possible for any one column to be out of line with its neighbour below and then, as a consequence, that will lead to instability at a given storey so the failure modes that lead to the general collapse of the structure are consistent with eye witness statements and because it was the lower four storeys that collapsed, and I will say here that it could have been the bottom storey alone but it seems like it may have been over four storeys, the people that generally survived the collapse were those in the upper two storeys on level 5 and 6 of the CTV building.

Q. That now takes you Professor to Section 4 – Conclusions.

A. "Conclusions

Based on the points raised and the analysis presented in this evidence, the following conclusions are drawn:

4.1 The CTV building was designed and constructed in an innovative fashion. This structure was one of the first in a new generation of multi-storey buildings in the 1980s that used precast components. Instead of using a ductile moment frame as had been the custom for cast-in-place structures of the day, the CTV building was designed with a "strong" wall system coupled with an "elastic" frame of columns and beams to support a proprietary type of floor system composed of a lightly reinforced slab cast on galvanised steel metal-rib decking.

Q. Can you just pause there please Professor. This is Slide 9.

A. Most of these conclusions are coming up on the slides. "The building was designed to the NZS 4203 Loadings Code and a deflection check was made to ensure the displacements under the code-specified seismic loading were not excessive and that the columns remained within the elastic range."

Q. Slide 10.

A. "4.2 When the Darfield earthquake struck, it imposed ground acceleration that were essentially similar to the design limits for which the structure of the CTV building had been designed. As a consequence, the structure was damaged; such damage would be expected, by design. The structure did not collapse and met its design objective of ensuring life-safety."

Q. Slide 11.

A. "4.3 In light of the possibility of a large aftershock, and given the fact that he engineers knew many structures around Christchurch had either met or exceeded their design expectations, they strictly should have been immediately Red Stickered by fiat; a site inspection was not even necessary to make this decision. Following this period, such buildings should have been both inspected and analyzed for collapse potential in subsequent earthquakes. If necessary, gravity critical structures (such as the CTV building) should have been shored up to ensure collapse prevention while valuables could have been retrieved and repairs or retrofits implemented."

Q. Now Professor we discussed this section earlier in your evidence.

A. Yes we did.

Q. Is there anything you want to add to what you've said there?

A. I don't think so. I'm bound to get lots of questions on this I'm sure.

Q. We'll go to Slide 12.

A. Okay, so 4.4. "The CTV building was inspected after the Darfield earthquake and damage noted and the building deemed safe to reoccupy. However, the owners/engineers evidently did not pay heed to the many reports from the CTV building occupants that the building felt uncomfortably lively. Further questions should have been raised regarding the soundness of the structure by the owners and thoroughly investigated by the assigned inspecting engineers.

1641

Q. Slide 13.

A. 4.5 The CTV building tragically collapsed in the Christchurch earthquake with a significant loss of life. An investigation into the collapse by the DBH led to the H-S Report. This report has been discussed and critiqued in this evidence and there are several assumptions and various aspects of the H-S Report that bring into question the veracity of the claims and conclusions. In fact the peer reviewer Holmes, as well as the DBH expert advisor Priestley, are not in agreement with key aspects of the report. It is for this reason further work is essential.

Q. Slide 14.

A. 4.6 One of the key areas leading to faulty conclusions in the H-S Report concerns the concrete strength. Testing and analysis commissioned by ACRL, and undertaken by, this should be ARCL sorry, undertaken by independent experts, demonstrated that the concrete was not deficient as claimed in the H-S Report. In fact the concrete strength is likely to be in the range of 1.5 times the specified design strength.

4.7 Another key area of deficiency in the analysis is the correct modelling of the columns coupled with the degrading strength of the beam-column joints. Axial load-moment interaction was not currently considered –

Q. Not correctly considered?

A. – not correctly considered in the non-linear time history analysis. Also, the beam-column joints that had no transverse reinforcement were modelled as rigid end blocks. As such the strength deterioration that occurs when the joint core concrete cracks was not modelled.

Q. Slide 15.

A. I would just like to note here that this comment was based on the initial NTHA that was done by the DBH and for the second current NTHA the simplifications still exist with respect to the joints but I think a lot of the issues related to the columns have indeed been corrected.

Q. Or would you prefer to pass on from that to slide 16?
A. Well I think we can read slide, number 4.8.
Q. Yes.
A. 4.8 Further non-linear time history analysis is needed and in fact it has been done, to fully understand the nature and causes of the collapse of the CTV building. In those analyses it will be essential that all four Geonet motions recorded during the Christchurch earthquake are included in order to correctly gauge the spread of results that might have conceivably happened at the CTV building site on February 22, 2011. Moreover, it is essential that the effect of the weakened structure following the Darfield earthquake be captured. This is most easily done via an end-on-end analysis, where the damage done in the Darfield earthquake is captured. In previous analyses detailed in the H-S Report on the work performed by Compusoft, the program was stopped at the completion of the Darfield earthquake and then restarted as if the structure was undamaged at the commencement of the Christchurch earthquake.
Q. Slide 16.
A. And again a supplementary comment is that the revised analysis does partly consider this but it's only for two earthquakes. That's the end on end analysis, it has redressed that comment.

4.9 Analyses as part of this evidence show that a sway failure is unlikely, and that a classic elastic Euler buckling failure over the lower four storeys is possible in either the east/west or the north/south directions. Such a failure does not rely on significant, if any, post-elastic performance. The lower four storeys were able to buckle due to the relative movement of the floors with respect to the shear wall system, and the relative movement necessary to achieve this need only be small, in the order of 30 millimetres. The collapse is primarily caused by the substantial increase in axial loads in the columns due to the exceptionally high vertical accelerations.
Q. Slide 17.

A. Now this, these are some supplementary thoughts that I had that go beyond the conclusions and I'm sure a lot of people are wondering why the CTV building collapsed or could it have been avoided, so first question here is, how could have the CTV building collapse be avoided? Well if by design larger columns had simply been used, that would have made quite a bit of difference, I've already said that if one had, the designers had chosen to use 500 millimetre columns instead of 400 millimetre columns, then the stresses in the columns would be considerably less and the columns themselves would be more capable of sustaining the high vertical accelerations and axial loads. They would also be markedly more buckling resistant. In addition to providing larger columns of course if the next thing that one should do after that would be to provide some more transverse reinforcement. Now we know that from the hearing so far this is contentious because some would claim that it had to be there, others would claim that it should at least be there based on best practice so that's why I've put that comment that from a best practice point of view certainly, and I would recommend that it should be there, but evidently according to some people’s view, it is not essential back in 1986 as is evidenced by the fact that quite a few buildings were built with that style of construction where essentially the main frames, gravity load bearing frames were considered as remaining elastic, not needing all the extra transverse reinforcement. Another feature I think that would have made quite a bit of difference in the performance is if bottom slab steel through the beam supports, now you wouldn't need this by design, the design calculations would show that the moments would be negative at the supports which means that you don't need bottom reinforcement, but the problem arises that because you have this tray deck, it’s discontinuous as it seats on the beam on one side to the other and if you had some steel that passed through the joint like a second layer of mesh on the bottom, that would certainly help tie things together. Again though the only difficulty is that from a construction point of view you would actually physically need to feed some bars in under the beam steel but in essence this would lock the slab into the beam and make that connection more integral at least for a lot longer than it would have been in the case of the CTV building. So those are just some supplementary comments that relate to what we've discussed.

Q. Now professor finally there's the matter of the questions that the Commission have put to expert witnesses such as yourself. You're familiar with those?

A. I am.

Q. Do you have them to hand?

A. I – it's all in my head. We were told to answer them orally so I'm prepared to have a crack at that.

Q. The first set of questions relates to the south wall, line 1, the opening observation is this wall appears to have been designed as a coupled shear wall. The first question is, would this wall have behaved as a coupled shear wall in the Canterbury earthquakes? In particular would the coupling beams have yielded with plastic hinges forming in each of the walls?

A. So maybe we could have that picture up please.

Q. Yes, the reference to the picture is BUI.MAD249.0493.2.

A. Okay, so a couple of comments here, one of the major issues with coupled shear walls like this is that they are seated on pad footings and they are not seated on pile foundations and this is a major problem for the left-hand wall. The left-hand wall is prone to go under tension and in order for the moments that you would see in the wall it has to be well anchored down. Now if that column is not well anchored down and particularly given the fact that we have high vertical accelerations in the order of, right, these are response accelerations in the order of 2G then that left-hand wall, my supposition without doing the calculations it is highly likely to go into tension and so instead of working as a coupled wall the coupling beams themselves are going to more or less hold the wall together and it's going to be more like a rocking bridge pair, such like as the south Rangatikei rail bridge which I am quite familiar with. It used to be on my patch on for the railways. That particular structure has two massive columns and it kind of steps from side to side under earthquakes okay and then you have this coupling beams that go across and they are deliberately by design designed to be quite rigid. Now it may well that this has been like this by accident but based on the questions that Professor Fenwick has put to others it is true that there is a problem here with it being slab steel being present because if you look at the diagram and I'm pointing now to the slab there's going to be some additional steel going through here so if you take a cut down this joint here and you take moments about this point here then the bending moment that one can squeeze out of this particular piece of coupling beam is perhaps quite markedly enhanced by virtue of the presence of the slab steel. Of course this is on the proviso that the slab steel itself hasn't fractured. But at least in the early parts of the earthquake it's highly likely that this would have been, would have remained together and the slab steel of course extends out into the beam quite a lot. So that's not going to be your classical x type of bending moment diagram that almost looks like the steel going through here. This is going to be an additional triangle on the one side and you'll get a markedly higher moment on this side than you will on that but that could be sufficient over strength and you typically call these sources of additional strength that is not included by design over strength, and if the over strength is maybe in the order of a factor of 1.5 which is not uncommon for this class of structure then it's highly likely that the mechanism as drawn there would not actually happen as expected.

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Q. Do you want to comment on the question whether the coupling beam had yielded the plastic hinges for me?

A. Well we talked about that before. I think it's clearly evident and again this just goes back to my supposition that one side becomes in tension and the other side rocks in compression then on the compression side at the toe of the wall you have very high compression, and I believe that at least in compression it has yielded at the toe of the wall. Now the associated question with that was well where's the tie down coming from and to provide a substantial amount of resistance it perhaps doesn't need a big tie down because it's basically compression that's providing the overturning moment resistance of the wall just on the compression toe and the other wall, the tension wall on the left here essentially can be, have no load in it at all and most of the action is taking place on the right-hand wall.

Q. Second question on the south wall: what influence would the floors in the building have had on the behaviour of the south wall?

A. Well I think I may have answered but however I'm quite sure that by design it was – I don't remember scrutinising the calculations for that particular aspect but it's customary to neglect the slab and pretend that it's not there whereas in reality it is and so the design is typically just considered the big purple line which is the tension tie provided across the right diagonal that would provide tension into both sides of the wall. Now having the, an additional tension into the through the slab is only going to make things worse.

Q. The purple line you refer to is the purple line shown on the upper diagram on 493.2?

Third question: was there an adequate load path to transmit the inertial forces from the floors into the south wall?

A. Well I think it's very precarious relying on HRC mesh. This has become more well known based on the more recent work that we did at Canterbury back in the 2000s where we did a large scale, a full scale test of hollow core floor slab systems which is not that different from this class of system here where you have a relatively thin topping and then you rely more or less entirely on mesh. It inevitably fails and so once that has failed it's very difficult to get loads into the wall via classical shear. Now that's not to say that you can't get loads into the walls because what the diaphragm will do is it will seek an alternative load path and that alternative load path will go via the columns and then the beams themselves can drag, provide drag forces onto the walls. And so what you would then need to rely upon are those large D28 bars in the top of the beams to literally drag the walls backwards and forwards while the wall is oscillating back and forth so instead of relying shear coming in from the outside which is possibly what was conceived of it's going to rely on these drag forces so that puts a lot of distress on the beam column joints facing the framing into the wall and they themselves will end up eventually becoming distressed as well.

COMMISSIONER FENWICK:

Q. Was that (inaudible 16:57:46)?

A. I don't believe so Sir no.

Q. In what way would it fail?

A. Well again the bottom bar problem is problematic as you know. The fact that those bars wrap up and they don't anchor in very deeply is a significant problem. I think of course under repeated cyclic loading if this was really truly working as intended by design with the Christchurch earthquake there are so many cycles of loading one can expect that this class of steel would have possibly fractured from fatigue.

Q. You're talking about the bars in the columns.

A. I'm talking about the what's framing into the walls from the beams nearby.

Q. 24 millimetre bars.

A. 24 sorry.

EXAMINATION CONTINUES: MR RENNIE

Q. And the fourth question on the south wall: how did the design inertial forces between the wall and the floors compare with the corresponding design actions calculated from NZS4203 and NZS117O.5?

A. I'm sorry I haven't done that calculation here.

Q. Moving to the north wall complex between –

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DISCUSSION

HEARING ADJOURNS: 5.05 PM

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