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 science thesis at Texas A & M University. Following the completion of his masters thesis in August 2010 Mr Urmson returned to New Zealand and joined ARCL in September 2010 as a graduate engineer. We have remained in contact as we continue to publish papers that were derived from his master's research. Mr Urmson was also previously employed at ARCL as an undergraduate intern during the summer of 2005 to 2006 and upon graduating and prior to his coming to Texas A & M University he was employed by ARCL as a graduate engineer.

During the summer of 2005 to 2006 my son Mr Thomas Mander was an undergraduate intern at ARCL. Upon graduating he undertook a master of science degree in civil engineering at Texas A & M University. I was not his thesis advisor but we did work together on a common research project and have published that work with other faculty and students. My son now resides in San Antonio, Texas. He assisted me with some aspects of analysing the behaviour of the columns in the CTV building and in particular carried out the computer programming.

During my own PhD studies that were conducted from 1979 to 1983 I was co-supervised by Dr M J Nigel Priestley who is a member of the expert panel. Apart from sitting on a NZSEE committee together in the early 2000s we have not worked together since 1983.

I do not consider that any of these relationships impact upon my impartiality or independence in this matter.

Qualifications and Experience. I hold a New Zealand Certificate in Engineering 1976 in the Christchurch Technical Institute, a Bachelor of Engineering Honours, first class honours, 1979 University of Canterbury and a PhD in Civil Engineering 1984 from the University of Canterbury. I am a fellow of the Institution of Professional Engineers of New Zealand and a member of the New Zealand Society for Earthquake Engineering. From 1973 to 1987 I was employed by New Zealand Railways (NZR). NZR also fully funded by education for the NZCE, BE and the PhD. I was initially employed as a draughtsman in Christchurch in 1973 through '78 then as an assistant engineer in Christchurch and Wanganui

1979-1985 and finally as the deputy group manager of the property business group in Wellington 1986.” Actually that should be 1987. I stand corrected on that. “In 1988 I left New Zealand to take up a position as a visiting assistant professor at the State University of New York at Buffalo. In 1988 I was appointed to a tenure track position in the Department of Civil Engineering as an associate professor and in 1995 was promoted with tenure to associate professor. I returned to New Zealand in July 2000 to take up a position as professor and chair of structural engineering in the Department of Civil Engineering at the University of Canterbury. In 2007 I returned to the United States and took up my current tenured position as the inaugural holder of a newly endowed professorship in the Zachry Department of Civil Engineering at Texas A & M University. I have written or co-written over 100 peer review journal papers, 40 peer reviewed research reports, 14 book chapters and 140 conference papers. Amongst several diverse topics the publications have a strong focus on structural concrete and earthquake engineering. I have been the advisor or co-advisor of 95 doctoral and masters graduate students including 19 PhD students, 10 of whom are currently in faculty positions at the level of associate or full professor in various universities around the world.

The research work on the behaviour of confined concrete conducted during my PhD studies is well known, highly cited and widely used internationally either directly or indirectly in codes and computer programmes”, and I might add such as SAP 2000. “My early work at SUNY was part of the first generation of research investigations on the seismic behaviour of non-seismically designed concrete structures such as buildings and bridges in zones of low to moderate seismicity such as Christchurch. I was also the developer of the theory and implementation of the fragility functions for highway bridges used in HAZUS, it’s a well known US FEMA initiative that is widely used in the United States and elsewhere for the risk assessment of the damage to the built environment caused by natural hazards. I have conducted numerous large scale laboratory experiments on structural components and subassemblages. In New York State I also performed field testing on several large bridge structures under ambient and forced vibration motion. My full resumé is annexed to this Statement.

I have been instructed by Buddle Findlay, on behalf of ARCL, to provide independent expert evidence on issues relevant to the collapse of the CTV building on 22nd of February 2011 following an earthquake of magnitude 6.3."

JUSTICE COOPER:

Q. Professor Mander can you just tell me when were you instructed?

A. It would be when I was here in April at the National Conference on Earthquake Engineering and it was either the first, second or third week. I don't remember the exact date. April this year.

WITNESS CONTINUES READING BRIEF OF EVIDENCE AT PARAGRAPH 13

A. "In particular I have been asked to undertake the following matters:

Review the key findings of the report of the Department of Building and Housing as reported by Dr Hyland and Mr Smith in January 2012;

Present, analyse and discuss the results of new investigations commissioned by ARCL into ground motions, concrete tests and column tests;

Perform an analysis of the column performance; and

Present and discuss any relevant issues, including in particular alternative collapse hypotheses.

I have prepared a submission which details my findings and opinions on these issues. My submission is annexed. Also annexed are a conference paper referred to, and this was actually presented at that conference I mentioned, by Mander and Huang "Damage, Death and Downtime Risk Attenuation in the 2011 Christchurch Earthquake", which is published in the Proceedings of the New Zealand Society of Earthquake Engineering 2012 Conference. The presentation that accompanied the paper, the slides for that are there too.

NTHA expert Panel

I am participating in the NTHA expert panel being facilitated by Professor Athol Carr. I reserve the right to modify or add to my evidence following the completion of this process.

(Professor Mander's resumé not required to be read)

EXAMINATION CONTINUES: MR RENNIE

Q. So Professor we now go to the first part of your first brief which is the document headed, "An Alternative Collapse Scenario for the CTV Building" and you'll recollect that where you've referred to a submission we are following the expression 'brief of evidence' in that respect and you start with the heading, "Introduction Scope of the Submission"?

A. Yes.

Q. And I think we need to go to the first power point.

WITNESS REFERS TO POWER POINT PRESENTATION

A. So that's me starting, Your Honour, with a mea culpa. I take blame for writing it as a submission. In fact I have a daughter who's a barrister and solicitor of the High Court and she advised me, maybe she should be ticked off for that, but she said that as a submission and I can walk away from it but as soon as I deliver it, it becomes evidence so I stand corrected on that. Thank you.

Q. I will take the matter up with her Sir but perhaps if we now start at the heading, "Introduction and Scope" and we can go to power point 2.

A. So I'm going to be talking about the three main headings here. These are quite lengthy so I thought I would just put these up first and we'll start off with the first one which is on a review of the findings of the DBH Report. Then we're going to present the new material that has been commissioned as part of this whole effort and then, thirdly, to look at the alternative collapse hypotheses.

So the introduction and scope of this submission. The first purpose of this submission is to review the key findings of the work commissioned by the Department of Building and Housing (DBH) on the Canterbury Television building, CTV building collapse investigation as reported by Dr Clark Hyland and Mr Ashley Smith and I will hereafter refer to this as the H-S Report. This submission will show that while the H-S Report has been comprehensively executed, much of the analysis has been based on several erroneous assumptions. The claim resulting from the CTV building collapse investigation are therefore faulty in their reasoning, leading to incorrect conclusions. The submission also considers the remarks made by Mr William Holmes who was the formally assigned external international peer reviewer of the H-S Report. Holmes is moderately critical of several technical points in the H-S Report, the foremost of which he considers the neglect of modelling the connections correctly. He goes on to point the way forward in seeking the truth to what really caused the final collapse of the CTV building but falls short of drawing firm conclusions. It should be noted that during the course of the CTV building collapse investigation the Hyland Smith team was advised by an external group appointed by the DBH. Professor Nigel Priestley was the Senior Engineering Advisor within the external group. Because the advisory group and the Hyland Smith team were not in agreement with the conclusions made in the H-S Report, Prof Priestley was invited by the Royal Commission of Enquiry (The Commission) to make a separate submission. Like Holmes, Professor

Priestley also points out several weaknesses in the report and also the way forward but without coming to definitive conclusions. This submission discusses the remarks by Prof Priestley who criticises some of the procedural analyses presented in the H-S Report and hence rebutts many of the conclusions made by the Hyland Smith team.
The second purpose of this submission is to present, analyse and discuss the results of new work done since the completion of the H-S Report. This new work includes a comparative analysis of the ground motions recorded recently at the CTV building site with the four other Geonet free-field recording stations within the vicinity of the Central Business District of Christchurch. A further analysis of concrete test results on test cylinders cored from the undamaged column remnants retrieved from the CTV building and tested by CTL Thompson, Material Engineers of Denver, Colorado and a review of the work completed by Dr Rajesh Dhakal, Associate Professor at the University of Canterbury who was commissioned to conduct full-scale tests on large intact column remnants retrieved from the CTV building. An analysis of column performance under double bending within each floor level of the CTV building to show the sensitivity of the concrete strength and confinement effects under different levels of axial load and some general conclusions from the above points.
The third and final purpose of this submission, or should I say evidence, is to provide an alternative collapse hypothesis to the original collapse hypothesis proposed in the H-S Report. It is shown that the columns independent of their degree of ductility capability should have collapsed over the lower four storeys from the classic type of buckling known as "oiler buckling".

Q. Professor, you read that as "should". Did you mean "should" or "could"? Could have collapsed?

A. Could have collapsed, that's better. Yes, could have collapsed over the lower four storeys from a classic type of buckling known as "oiler buckling" largely due to the overload effects arising from extremely high vertical ground motions and promoted from a deteriorated beam column joint condition. At the time of writing this submission full corroboration of the alternative collapse hypothesis for advanced computational analysis is still a work in progress. The additional analysis to be conducted by Compusoft, the original sub-contractors for the H-S Report, through the computational non-linear time history analysis expert panel process may provide useful insights that are expected to support or modify the alternative collapse hypothesis.

WITNESS REFERS TO POWER POINT 3

A. So what's on the screen here is taken directly from the H-S Report and in fact if you go to the very first page of the Executive Summary this information is there.

'The principal conclusion of the H-S Report states: Investigation has shown that the CTV building collapsed because earthquake shaking generated forces and displacements in a critical column or columns sufficient to cause failure. Once one column failed, other columns rapidly became overloaded and failed.'

I might say this is too vague to be meaningful and one did not need to spend millions of dollars to come with this conclusion. Somebody walking by on the street could have come up with this conclusion, so a casual observer could have conducted that from the sidewalk. So really we need to delve deeper and try and tease more out of this so if we continue on. The above conclusion is so generic that it could apply to virtually any type of building collapse. Moreover this conclusion is so vague it is neither helpful nor insightful. What is also not clear is what specific forces or displacements are being referred to. Are these north/south or east/west or torsional combination, up down or some unknown combination of all of these. On further reading of the H-S Report it becomes clear that an emphasis has been placed on the lateral displacements as the principal trigger mechanism that initiated collapse. Initial clues to this are found in the contributing factors listed in the commencement of the Executive Summary, with more detailed discussion in Section 8, which is on the collapse scenario evaluation of the H-S Report. This submission responds to and critiques the supporting conclusions made in the H-S Report, addressing each of the contributing factors in turn. So if we could have the next slide and those are listed there.

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Q. Power point 4, and the source of these?

A. This is – these are the bullet points on page, on the first page of the Executive Summary of the H-S report and basically I've taken these one by one and then tried to dissect them and unpack them and then give my own thoughts to these.

So let's start with the first one, higher than expected horizontal ground motions. The CTV Building was designed and constructed in compliance with the applicable design and building codes and the developer was granted a building permit from the Christchurch City Council. The CTV Building was also designed in accordance within accepted industry practice of the 1980s for a structure to withstand much smaller elastic forces than a full design-level earthquake. Then when the full force of the design-level earthquake is applied, the structure is expected to be damaged, but without collapse. Even though the damage to the structure can be tolerated, life-safety via collapse-prevention must be ensured. To illustrate the design process, and how the structure of the CTV Building measured up to the design expectation, a comparison of the as-designed seismic capacity with the two major earthquakes it was exposed to will now be given. The CTV Building had a first mode natural period of 1.0 seconds, for which the loadings code NZS4203 specified a spectral acceleration of 0.095, 9.5 percent of g. Implicit in this prescribed value is that structures, if appropriately detailed, should survive an earthquake some four times the value of 0.095. Thus, at T = 1.0 seconds, a 5% elastic response spectral acceleration ordinate of 0.38 g is implied by the loadings code for the CTV Building. Note that g is the gravitational acceleration constant of 9.81 metres per second squared. Recent work by Dr Brendon Bradley has shown that for the February earthquake, the Christchurch earthquake, at the CTV building site a median conditional spectral amplitude for T = 1.0 seconds is 0.75 g. However, there is considerable spread and the plus/minus one standard deviation results range from as low as .55 to 1.0 g. Now I was asked to put in an explanatory comment here so I'll read that too. Whenever an earthquake strikes, vibration waves propagate through the rocks and soils. Soils are particularly problematic because their properties vary so much, in a random type of fashion. Therefore, the manner in which the seismic waves propagate is affected by this randomness in the soil's properties – the velocity and severity of the seismic waves are altered by the soil variability. For example, at two relatively nearby sites, seismic sensors could record potentially quite different outcomes, particularly in the high frequency band. Any two earthquakes have quite different properties. In general, this randomness or variability called aleatory uncertainty, a type of uncertainty that can be quantified and thereby mathematically modelled in a probabilistic sense. When such known uncertainty is applied to the CTV Building site, it leads to a relatively broad band of possible outcomes. In statistical terms, this is quantified by the standard deviation. From any statistical tables this band of spread ranges from the sixteenth percentile (meaning 16 out of 100 similar events would have a smaller result) to the eighty-fourth percentile, or 16 out of 100 similar events would have a larger result). Or in other words, roughly two-thirds of all possible events or earthquakes like the Christchurch earthquakes would expect to produce vibration signatures that would fall within this plus/minus one-standard deviation range. Compared to code-base design motions, the CTV Building site withstood much higher expected horizontal ground motions. Now I put that in italics and underline it b those are not my words, those came directly from the H-S report but I do agree with them, naturally. For any structure to survive such a high level of shaking is a bonus. It was certainly not a requirement at the time the CTV building was designed and constructed in the late 1980s. So the supporting conclusion in the H-S Report, that higher than expected horizontal ground motions were observed, is correct. However, the H-S Report essentially neglects the effect of earlier earthquakes on the structure of the CTV building. While much higher than expected ground motions were observed during the Christchurch earthquake, focusing the discussion on this disregards the fact that the CTV building, and indeed all structures in the Christchurch area, suffered varying degrees of damage in previous earthquakes, commencing with the magnitude 7.1 earthquake in Darfield on the 4th of September, the Darfield earthquake. Based on the results again from Brendon Bradley in 2012 at the CTV building site the Darfield Earthquake produced a median conditional spectral amplitude for 1.0 seconds of 0.33 g, with a plus or minus one standard deviation spread between 0.24 and 0.44. Hence it can be inferred that there is a 40% chance that the –

JUSTICE COOPER:

Q. (inaudible 10:27:51).

A. Okay, .44 at 84 percent and .24 with a 16 percent.

Q. Yes.

EXAMINATION CONTINUES: MR RENNIE

A. Hence it can be inferred that there is a 40 percent chance that the Darfield Earthquake ground motion at the CTV Building site was larger than the level of ground motion.

Q. Professor, in making that statement have you referred to Ashley Smith's evidence?

A. Yes I have, he took exception that 40 percent for him wasn't good enough Sir, but that kind of surprises me given that he probably would have studied statistics at Auckland University, I hope so, but basically if the answer was 50 percent then that's like a bulls-eye. So we're not trying to get 100 percent bigger than, we're trying to get sort of parity or at 50/50 and if that was the case then one would say, well it was more or less exactly the same but we can't say anything with such definitiveness but what we can say is that is essentially the same so I'm claiming that this it is essentially the same, and by the way that is actually a technical term that civil engineers and academics such as myself would commonly use.

Q. Thank you. From the third line, "That is the initial ..."

A. That is, the initial Darfield Earthquake alone produced essentially the same forces and acceleration that the Christchurch City Council permitted CTV building was designed to resist. By design, significant damage would be expected from such a level of ground shaking. The fact that the CTV Building survived the design-level Darfield earthquake, with only minor visually observable damage, is a testament to the sufficiency of the design – it met the aim and objective of the design codes. However, as will be discussed in section 2 below, it is evident that the structure of the CTV building must have also sustained hidden, unobserved and/or unobservable damage. It can be argued that with the level of observed as well as hidden damage, the CTV building should have been red stickered following the Darfield earthquake.

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Q. Now Professor I think you discussed that point somewhat later in your brief of evidence. There's reference at 4.3 on page 89 of the Commission's reference numbers but briefly can you at this point just indicate what you mean by the expression "red stickered"?

A. Red stickered is I guess my own term. One could use that or paraphrase it in many different ways. I could say that it's a red zone at one building if that's all it was. If it was a particularly critical building you would put a ring fence and a red zone around that one building but actually I think it's bigger than that. Like this class of structure was quite typical of other structures that may have or may exist in this and other jurisdictions and it would behove structural engineers to be a little more cautious immediately after an earthquake. I think we all now understand that this to be the case and as a consequence whenever there is a major aftershock essentially everybody is extremely cautious but they weren't at the time and I think that's largely because there were no fatalities and people, engineers and regulators were lulled into a false sense of security which really was a pity it wasn't there. Now I haven't, I don't believe the plan is to present and discuss my paper that I presented that's been submitted as part of this package.

Q. That's the 2012 New Zealand Society of Earthquake Engineering paper?

A. Yes but it might be worth looking at one slide in particular if you would please.

Q. The paper is annexed to your brief. It's .93 and it's a joint paper by yourself and Mr Huang I think.

A. Miss Huang.

Q. Miss Huang. And I'm not certain which slide you wanted to refer to.

A. Well it's the one with the red, yellow and green stripes in it.

Q. We will find that for you in one moment but just while we're finding that does the Society for Earthquake Engineering have a concept known as a red placard?

A. I guess they did.

Q. Are you referring to that or the Christchurch City Council's white, yellow, red system or what are you referring to as the red sticker?

JUSTICE COOPER:

Well it is green, yellow and red.

EXAMINATION CONTINUES: MR RENNIE

Q. Sorry Sir green?

A. I'm referring to this in a very generic sense in fact because this is a common parlance used in other jurisdictions particularly in the United States. If I'm not incorrect I believe it actually started more in the earthquake territory on the west coast but it's very close to home and it's on my mind a lot because we deal with this all the time in Texas where we have hurricanes and where we try to give advanced warning by evacuations and so you might say an evacuation zone is a red zone. This is really essentially what I'm saying is that if people are in buildings that are a danger to them they should be evacuated from those just like every New Zealander and myself included have fire drills and people are not let back into the building until the firemen clear the building and it's safe to go back.

Q. I think we now have the slide which you wanted. The reference is WIT.MANDER.0001.105.

A. Okay I must admit this is relatively new thinking but it's not entirely new it's been around for a while but if one looks at these four graphs here, on the top right-hand corner we have the earthquake shaking intensity and the vertical axis and the horizontal axis we have the radial distance so that's a classical accentuated relationship and then if we go to the left we have a, this is essentially an IBA curved result.

Q. To be clear you're referring to the top left-hand graph?

A. The top left-hand graph and so this is a characterisation of the building. That index there B is typically about one for tallish buildings because that just conforms to the equal displacement principle and then below this we have a damage relationship that relates losses and it can be in terms of death, it can be in terms of downtime or it can be in terms of actual damage and so the engineering demand parameter that that's defined with respect to most engineers would be familiar with it. This is just a plain old raw drift that we often refer to. That particular parameter is used as a major measure of damage. Now when you put all these four together you can then transform this round onto the fourth graph and then if you look at these different damage states as given by these vertical lines in here then it's possible to figure out exactly where one would safely put down a red zone. Now this is not to say that all buildings should be classified thus but what it is saying is that if a preponderance of buildings were of a similar ilk to this particular building that's under analysis here then we can ring fence this and say inside that we need to take, at least take a second look. We should not go in there again in a very cavalier fashion expecting there to be no damage, and then the next zone of course is the yellow zone which means that you probably shouldn't sleep in that building or work in it but you can go in more or less freely to get out goods and valuables, and then the third region which is green is actually where there may be still damage but that damage is of such a superficial nature it probably doesn't matter. So this is kind of well correlated back and I, and one can do these sorts of analyses as a very simple formulae and that's why in the conclusions I make the remark that this could be done by fiat in that somebody as the regulator would have to make the call. Now in Texas a very important person does it and he's called the Judge. The county Judge and that's all the power's put in the Judge's hand to make this call. They are seen to be apolitical and it's a very effective tool as to how this works.

Q. Going back to –

JUSTICE COOPER:

Q. Well can I just clarify if I am understanding you you have described a system which would result in all the buildings in a particular area being treated as within a red zone and not able to be occupied, is that right?

A. It may mean that if there were a lot of buildings of that type because we all know in Christchurch one of the things after, particularly after the February earthquake, that I suppose you might say we were all scared about was the drop zone, the fall zone and that you might have a perfectly robust building next door to one that's offending and then if it was to collapse then of course it can, there's a lot of collateral damage.

Q. But construct here depends upon distance from the epicentre doesn't it?

A. That's correct.

Q. So in such a system how does one allow for the fact that a building might have been designed to the latest code and its neighbour may have been designed to a code that was 20 years old.

A. Well the persons making the call would be fully aware of that. Like I'm not sure how you would want to apply this in New Zealand but as I've said I know in Texas the county Judge would make that call but as is fit and when that decision was being made he would have the probably somebody like the chief of Texas taskforce one who is like the equivalent of the USAR here and also a county engineer so those are all, those decisions are made collaboratively but the responsibility is left with the Judge.

Q. So is the system analogous to the red zone that was established here after the February earthquake is it?

A. I think this is very, very similar.

Q. Yes.

A. And the point, I guess the lesson to be learned is that we learned the lesson very quickly after February but it hadn't occurred to us all and I'd say we're all guilty as engineers in the sense that we didn't realise the severity of what was lurking.

Q. Correct me if I'm wrong but are you suggesting then that there should have been a red zone similar to what was imposed on the central business district in February in September?

A. Well I, I would hesitate to go that far Your Honour, but what I would say is that there needed to be more caution in my view because how it turned out from what I can tell is that people went ahead in good faith and made inspections, very quickly, and if they saw nothing then nothing was wrong. Now this'll come up further down when we can discuss it further but my view is that –

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Q. But one could say all that without having coloured graphs?

A. No because the coloured graph, as you know, like I think when others have been asked about this they've been kind of aghast at the thought that this is kind of draconian and dramatic and I believe that it may seem like that but my feeling is that how the whole approach has panned out from the Darfield event and the thinking at the time was that, well buildings are not damaged unless you can see the damage, and in fact Mr Keho may, he still stands by that. Now, I don't believe that that is the case. I think like buildings are not people. Buildings are inanimate objects. We don't have to be worried about offending them and I believe we should work and operate under the adage that these buildings are guilty until proven innocent.

EXAMINATION CONTINUES: MR RENNIE

Q. Going back to your brief and your statement as to red stickering, are you identifying the CTV building as one of a type which on the characteristics known after Darfield would have fallen in your red segment?

A. Yes I believe so. Like take for example I knew Brian Bluck from many years ago and I –

Q. To be clear, he was the former Christchurch City Council chief engineer?

A. Yes. And I was unaware until recently that he had passed away, but I knew him from in the days when I was a student and he was a man to be revered in the city. He knew everything, and he is the sort of person that would've had all this knowledge in his head. He would've known the types of buildings that he would've permitted and had somebody said to him, "Hey, these buildings could've been in trouble," he would've been the sort of person that would've advised the mayor that there may be some problems here, we should be extra cautious. That's basically what I'm saying.

Q. And on your principle of a building being guilty until proven innocent, what to be clear is the element of proof needed to achieve an acquittal?

A. Well not a jury, first of all. I think there needs to be some wise people judging what the building is likely to, how it's likely to perform, and first and foremost in all of this, a lot of this could be done from a desk study. You don't need to go into the field necessarily. But I think in the first, like everybody knows in emergency situations most people if they're going to sadly pass away it will be in the first 72 hours, and it's in that first, certainly 24 but up to 72 hours one needs to be extra cautious. And then during that time you have a sufficient time to regroup and, you know, it seems, it may seem to New Zealanders that's rather harsh but you know I live all the time with, in Texas where at any, every season, it's coming up right now, this time of the year through to November when at any time you need to have a bag packed ready to leave because you might have to evacuate because of a hurricane. Now you're going to be gone for two days, 72 hours or a little bit more. That is, you know, worth it if it's going to save your life.

Q. Now going back to the brief, the second paragraph on the page, the fourth line, "Inspecting engineers…"?

A. Inspecting engineers would have been well aware that the level of ground motions sustained was similar to the level that the design code NZS4203 of 1984 called for. This should have served as a signal that substantial inelastic response would have occurred, whether it was seen or unseen. It is therefore concerning that the inspectors did not immediately red sticker the CTV building. What is more concerning is that following the Darfield Earthquake, eyewitnesses reported on numerous occasions that the CTV building was uncomfortably lively. Again, this should have served as a signal and as further confirmation to inspecting engineers that the CTV building had sustained some hidden damage, and that they should take a second look to determine the source of damage. Some insights into the cause of this structural liveliness are discussed at the close of section 1.2 below.

So 1.2 is the next one on our slide, exceptionally high vertical ground motions. The seismic design environment of the 1980s. During the 1980s the structural engineers operated using a heuristic rule that vertical ground motions would be about two-thirds of the horizontal motions. Stiff and heavy horizontal elements such as floor slabs tend to have high vibration frequencies greater than 2 Hz, that is the vibration frequencies for those elements are normally less than about 0.5 seconds. The 1980s design spectrum for Christchurch for short periods (that would affect higher modes, such as floors) was 12.5% g. This implies elastic spectral response accelerations for T is less than 0.7 seconds is equal to 0.5 g and 0.33 g, for horizontal and vertical motions, respectively. Floor slabs tend to be excited by these vertical motions, and at this modest level of shaking the normal safety factors inherent in gravity load design make the floors capable of sustaining vertical ground motion induced vibrations. By inspecting the vertical response results in the Compusoft report, it says H-S, it should be Compusoft report, (figures 51 and 56), it is evident that the CTV building had a floor system with a vibration period of T equals 0.25 seconds.

Q. Just pause there, that's ENG.COM.0001.75. Do you have that in front of you?

WITNESS REFERRED TO SLIDE

A. Yes.

Q. That's the first of the two that you refer to. Do you particularly wish to point out on that?

A. Well actually the two graphs more or less line up. The upper graph in 50 basically shows the side sway effect. So those are the horizontal motions and the simple way to read a graph like this is to look at the peaks on one side and so you can see they're occurring about every one and a half to two seconds. Now I just want to point out here that the natural period of the structure is thought to be about 1.0 seconds, but under inelastic response, that means it's got damage, the period is going to lengthen to somewhere about one and a half in one direction and two in the other direction. Now superimposed on that of course you have vertical motion vibration, and the lower graph shows the liveliness of the floor slabs during the earthquake. Now these are very, very significant ground motions, and if anybody had lived through that and felt it they would be, that would be certainly something to remember. And so that's the main point to make, whether the floors themselves were extremely lively. Now those vertical forces that are generated by basically the floor is going to be more or less fluttering up and down like this. And so in the process of doing this the reactions that go into the columns are going to dramatically increase as well, they're also going reduce a lot, and so with that very large variation in axial force the internal resistance of the elements have to re-orientate themselves very rapidly and the stresses are going to amplify dramatically as a consequence. There will be a lot of internal damage if the structural elements are not appropriately detailed.

1050

Q. And now it's figure 51 which you referred to, if we go to .78 which is figure 56. If we can enlarge the 56 please.

A. So this is a similar graph only basically everything is on, is plotted on the one graph. The purple line or the – actually the blue coloured line is probably the better one to look at. That's again the horizontal motion and the red line is not a displacement now, it's the right-hand axis which reads axial load so that has an average, but this is in the lower storey floor slab. This has an average of about 1700 kilonewtons but the columns were shaking backwards and forwards horizontally and meanwhile the vertical reactions are dramatically changing through the floor system and the unreinforced joints and the columns have to – will have particular difficulty coping with that.

Q. Going back to your brief, the fourth line, "During the Christchurch earthquake ..."

A. So during the Christchurch earthquake the observed vertical spectral response accelerations results are, for a short periods at T = 2.5 seconds, .55 –

Q. T = .25 seconds?

A. Yes, T = .25 seconds, .55, 0.65, 0.85 and 0.95 for the Botanical Gardens, the Christchurch Hospital, the Catholic Cathedral and the Resthaven recording stations respectively. Clearly, these seismic demands were much higher, and again that's emphasised using Hyland's words, than the expected level of 0.33 g implied by design. Thus, the supporting conclusion of the H-S Report that exceptionally high vertical ground motions helped lead to the demise of the CTV building is correct. But again, the H-S Report essentially neglects the effect earlier earthquakes had on the structure of the CTV building. During the Darfield Earthquake the observed vertical spectral response accelerations were point, at the same period, at .3, .3, .27 and .37 for those same four stations. These demands are about the same as the 0.33 g that would be expected for a design-level ground motion. Again, inspecting engineers would have been aware that this design expectation for the CTV building either was already met or had been exceeded. Therefore, engineers should not have been surprised by reports from occupants that the CTV building was considerably more lively after the Darfield earthquake. The effects of the damage felt by the CTV building occupants should have served as sufficient evidence that the CTV building should have been red stickered. I might add add this point that both Mr Holmes and Professor Priestley were questioned on this point, asking what they thought of my stuff, and one of the things that they seemed to say was that while it wasn't all that bad. I would contend that what they were saying wasn't that bad, was probably in a different part of the spectrum. Clearly in the section of the spectrum where it really and truly matters, it was bad, no doubt about it in my mind, and we will cover that some more later. The unexpected consequences of exceptionally high vertical ground motions –

JUSTICE COOPER:

Q. Is that word heading there –

A. I'm sorry I don't know that got there.

MR RENNIE ADDRESSES JUSTICE COOPER

EXAMINATION CONTINUES: MR RENNIE

A. Not referred to in the H-S Report is the fact that the CTV building had a continuous three-span one-way slab system in the north/south direction. The relatively high vertical vibrations from earthquakes prior to the Christchurch earthquake essentially would have broken the fixed end condition to make the slabs function more like three individual simply supported units. Therefore, vibration-induced deflections would be amplified up to, and I stand corrected by Professor Priestly here, this is actually 400 percent. Although these vibrations made the CTV building occupants feel uncomfortable, they would not have seriously impaired, it would have not seriously impaired the safety of the vertical load-carrying system. However, the floor-seat connection damage made the CTV building ill-prepared to survive subsequent large earthquakes, in particular the Christchurch earthquake, which produced force demands and accelerations that were some three times larger than those implicitly accommodated for in the code-based designs of the 1980s. During the Christchurch earthquake, out-of-plane shaking induced damage would have impaired the ability of the floors to adequately transmit concurrent in-plane forces and may have led to a buckled folded-plate failure mode. In light of these exceptional demands on the CTV building, it is quite surprising that this point has not received more attention. For example, if we were to run this analysis as a graduate student project, the students would be instructed to analyse the effect of the vertical ground motions alone, without the horizontal components of ground shaking and to investigate the extent of damage caused to the structure. Naturally, all motions combined would also be analysed as well and the results compared. Instead of the effect of the horizontal actions being the primary cause of the CTV building failure, it is contended that the exceptionally high vertical ground motions were a primary contributor to triggering the CTV building's failure and subsequent collapse.

1.3 – Lack of ductile detailing in critical columns.

Although confined concrete columns have been the hallmark of building and bridge structures constructed in New Zealand since the 1970s, a liberal interpretation of the 1980s building design codes allowed for designers to choose other strategies to provide earthquake resistance. It appears that the deviance from ductile detailing in the concrete columns was contentious at the time that the CTV building designer sought the building permit from the Christchurch City Council. This deviance from customary ductile detailing remains a contentious issue in the H-S Report. Good ductile detailing, including confinement of columns, is highly desirable in the delivery of a robust structure.

However, during the 1980s era of building construction there began a time when developers and contractors put immense pressure on structural designers to deliver buildings at low cost, coupled with rapid construction details. The former mold was broken, moving from cast-in-place moment-frame systems that were the hallmark of a mini building boom in the 1970s, to the entirely modular precast structures of today. This modern era started around the time of the design of the CTV building when the time-cost-of-money, interest rates are approximately 26 percent, was dictating shorter project delivery times. The CTV building was in fact quite revolutionary at that time, as the details of the design are clearly contractor-friendly. It appears to be for these reasons that the structural designer evidently sought a simpler form of construction that avoided the use of copious quantities of transverse reinforcing steel to provide a ductility capability.

Transverse reinforcement in columns provides three primary functions: (1) it confines and strengthens the core concrete so that when the cover concrete in the end regions spalls off at high bending strains, the strength is restored due to the substitution of the strong confined core. Confined concrete also permits very large strains to exist (from sway effects) that will still providing a substantial amount of stress resistance.

(2) It provides additional shear resistance that is not possible once the concrete cracks. Because the concrete is highly strained and highly cracked in the end regions, the shear strength is sustained through tightly wound spirals or closely spaced hoops.

(3) Under high bending rotations, once the cover spalls off, the longitudinal reinforcing steel is prone to buckle. Closely spaced transverse steel inhibits this buckling and allows the reinforcing bars to maintain their high compression strains and loads.

Although it is true that the columns were not provided with substantial transverse reinforcement, this was neither a problem nor a cause of failure within the CTV building. If it were a cause of the collapse, then there would be substantial forensic evidence indicating that most columns had significant lengths of cover concrete spalled off, substantial buckling of the longitudinal reinforcing bars due to the high axial loads, and diagonal shear cracks. There is little evidence of such damage to the columns of the CTV building. There is only some minor evidence that short circumferential rings of concrete were missing, but it is contended that this was mostly an outcome of the building collapse not the cause.

There is an analogous problem to low transverse reinforcing steel in the columns: no transverse reinforcing steel in the beam column joint regions. Transverse reinforcing steel in the beam column joint regions in the form of horizontally oriented closely spaced spirals or hoops is normally provided to help resist high joint shear forces. Such horizontal joint steel is also called “confining steel” in the United States, whereas in New Zealand it is merely called joint shear reinforcement. Such reinforcement is not intended for reasons of ductility per se, rather it provides the two functions listed in paragraphs (ii) and (iii) above, and specifically that’s shear resistance and longitudinal bar buckling of strain.

The shear force demands in beam column joints are several times greater than in the surrounding columns and beams. For robust performance, it can be shown from the fundamentals of mechanics that there should be roughly the same area of transverse reinforcing steel in the beam column joint as there is in the top and bottom beams – and I mean by that beam steel – combined. Such highly reinforced joints are very hard to construct, and contractors certainly prefer not to place steel in the beam column joints as it is a very slow, awkward and therefore costly within the construction process.

The fact that a beam column joint has no transverse steel does not mean that it does not have a shear transfer mechanism. Instead, concrete arch action occurs and a diagonal concrete strut provides the shear transfer mechanism. The concrete strut also serves as part of the axial (gravity) load path. However, the joint concrete's ability to transmit this heightened level of load due to higher stress intensities is impaired under significant transverse tension strains (ε_1), cyclic loading effects, or both. Earthquakes are of course highly cyclic in nature and the alternating loads eventually wear the concrete's resistance down to a point where it becomes rubble.

It is contended that the lack of joint shear reinforcement is one of the principal contributing factors to the CTV building's collapse. In fact, prior to the Christchurch Earthquake, substantial fatigue-like damage would have already existed in the joints in the CTV building. Damage to the concrete beam-column joints resulting from the Darfield Earthquake also provide additional evidence of the lively nature of the CTV building, and I should add there, although this is not so easily discernible, this sort of thing can be fairly easily calculated from a subassemblage and we'll be going onto this later.

The simple beam column cruciform subassemblage, as depicted in figure B.5 of the Compusoft report, is not complete. When properly completed, it tells the story of very high joint shear intensity, where the beams are stronger than the columns and the columns stronger than the joints. For the CTV building, because the joints were the weak link in the chain, they were part of a primary trigger mechanism of the collapse mode.

Finally, on this point of ductility, it can be shown that if the NZS3101 code prescribed amount of transverse reinforcement was provided in the columns, this would not necessarily have prevented the collapse of the structure of the CTV building via the columns. This is because the 400 millimetre diameter columns were small with a relatively high degree of concrete cover, sized for a Darfield Earthquake type of event — a test which the CTV building demonstrably passed. Had 500 millimetre diameter columns been used, along with closely spaced spiral (confinement) reinforcing in the columns, and even more closely spaced spiral in the beam column joints, then the CTV building would have still been damaged in the Christchurch Earthquake, but the vertical load path would have been maintained thereby giving the CTV building a greater chance of surviving a collapse. But in the 1980s at the time of design, such columns and joints would have been considered an expensive and unnecessary luxury that would minimize the developer's profit margin.

1.4 Low concrete strength in the critical columns.

When an engineered structure is being built, the contractors order materials based on a specified strength. Once purchased, there is a design-based implication that there should be a 95% chance that the observed strength of any materials sampled either meets or exceeds the specified strength. Thus the as built strength is generally somewhat greater than the specified strength. The in situ strength of concrete is formally defined as the average crushing stress (in megapascal units) of three standard 100 millimetre diameter by 200 millimetre long test cylinders when tested at 28 days after pouring the concrete. To achieve a concrete strength to ensure a 95% exceedance probability that the provided in situ strength will be greater than the specified strength, the ready-mix concrete batching plant uses a target mean strength that is greater than the specified strength by some 25%. Once placed, the in situ concrete further hardens over time, such that after a few years the strength is typically 20% greater than the 28 day test results. Therefore, when assessing the strength of an as built structure in the absence of any material test data, it is customary professional practice to use a probable strength of 1.5 times the specified strength. However, in order to conduct a full forensic analysis of any collapsed structure, it is wise to obtain and test samples of the materials used in the structure to ascertain the actual as-built strength.

When analyzing the concrete strength of the critical columns of the CTV building, Opus International Consultants Limited (the subcontractors) carried out formal testing of the concrete using several core samples extracted from the CTV building. These small diameter test cores were obtained by drilling into the sides of the CTV building columns. To supplement only a few destructive crush test results on these smaller than standard cylinder specimens, an alternative rapid non-destructive test method was applied – the Schmidt-Hammer test. Once calibrated and collated, the results were averaged for all concrete and presented graphically in the form of a normal distribution, as shown in figure 5 of the H-S Report.

1100

Q. It is best if we just bring that up, that's BUI.MAD249.0189.39.

WITNESS REFERRED TO SLIDE

Q. Do you have that?

A. Yes, perhaps we could enlarge the first one? So in particular I'm going to be talking about this again, but I guess it doesn't hurt to have some positive reinforcement, but one is particularly alarmed when you see that, like the area in here is, well let me start by saying any, under any one of these graphs the area should add to one. As one calculates the area. So this area in here is about one third or maybe –

Q. You're referring to the column concrete strength from tests between the base and 25 MPas?

A. Yes. And so that would, according to their test, would say there is a one third chance that the concrete was under spec and cert – that immediately raises alarm bells in light of what I've just read because one would expect the strength to be stronger than that. But to me a more alarming thing is not so much this, is that people do not plot the actual data points here, and that's particularly concerning and worrying because it leaves an implication that if you look down on the tail of the curve, like if you were to take the five percentile which you would normally expect it to be greater than, and then put the five percentile on there, that's going to be about 12 and a half megapascals. Now I can assure you, it's not rocket science but even a layperson can look at concrete and tell that the quality is not good if it's that weak. And I find that extraordinarily hard to believe. There was absolutely no evidence from the consultant who went round afterwards in his photographs that the concrete was bony. Now bony concrete is something that we refer to where the sand is missing. When the sand is missing, some of the aggregate hasn't been vibrated correctly. You can tell by observation you're in trouble. Now the other thing that engineers do is they use various tools at their disposal, just like geologists do and one of the formal tools is actually a pocket knife so if you put a pocket knife into the stony bony concrete then you can prise out the stones and again this will be a signal that it's weak. There's no evidence there was anything like that so this kind of bothered me when I saw this. There's something very, very fishy so that caused us to think some more and do some more.

1110

Q. Very, very fishy with the concrete or very, very fishy with the tests?

A. Well I suppose you could put fissures in the holes if it was really bad concrete. I remember seeing a lot of this in the days when I worked for the Railways in the North Island. I did a lot of tunnel work and the linings on the Mangaweka Tunnel and the others that we worked on they were all hand-compacted concrete. You could basically read the day clock by looking at that concrete and look at the bony concrete and say, "Yep, that's the end of the afternoon" and because about two hours later you could see where all their lunch wrappers were wrapped up when we pulled the lining off so it's very easy like to a schooled eye one can tell what goes on in concrete and there's no evidence in my view that this was bad concrete.

Q. Thank you, next paragraph – Examination of the Plotted Test Results.

A. Examination of the Plotted Test Results imply that the as built concrete only possess a characteristic strength. That means a 95% exceedance probability of about 15 MPa. Clearly this is an unrealistic result. It is evident from a cursory inspection of the rubble at the Burwood dump site, that the quality of the concrete is mixed. Much of it is damaged partly from the collapse and also partly from the fire. However, a substantial amount of the concrete appears to be undamaged. A visual inspection reveals the quality to be quite sound and not likely to be as weak as 15 MPa.

Dr James MacKechnie was engaged by the Commission to review the concrete tests and the associated conclusions in the H-S Report. His review casts serious doubt on the process and procedures.

Dr Brendon Bradley was engaged by Buddle Findlay on behalf of ARCL to conduct further analyses on the concrete tests presented in the H-S

Report. His rigorous and formal probabilistic analysis shows that there is *no statistical significance* in the claim that the columns had lower concrete strength than specified.

CTL in the United States tested eight large diameter core samples extracted from the central region of column remnants from the CTV building collapse. All test results showed the concrete to be above the specified strength, with an average value of 40 MPa. Even without further analysis, it is immediately apparent that the CTL results are more reasonably in keeping with what one would expect to observe with concrete aged some 25 years.

In concert with the physical strength tests conducted by CTL, Mr Douglas Haavik was engaged to conduct a detailed materials study on the quality of the concrete in the columns of the CTV building. Haavik concludes:

"... there is no reason to believe that there was a systematic reduction in concrete strength supplied to the project and that any such reduction is likely attributable only to gross error for a specified load of concrete which itself is extremely unlikely."

In spite of the so-called "low concrete strength of the critical columns", Compusoft wisely chose to ignore this advice from Opus, and used the "specified strength + 2.5 MPA." However, in light of the more recent evidence from the CTL labs, even this assumption was on the low side of the probable strength.

Further analysis presented in Section 2 of this evidence on the concrete tests conducted by the CTL labs. This analysis demonstrates that had the customary practice been used of having a probable strength of 1.5 times the specified strength, then more realistic results would have been obtained in the non-linear time history analysis work conducted by Compusoft. In summary, the claim in the H-S Report that the concrete had low concrete strength in the critical columns is erroneous.

1.5 Interaction of perimeter columns with the spandrel panels.

Historically there have been many instances of collapsed structures where the so-called "soft-storey" effect has been caused by the presence of "short" shear-critical columns. Often the short column effect is due to the presence of up to half-storey high infilled frames. The presence of the spandrel panels in the CTV building alludes to this class of failure, with the resulting collapse mechanism developed as shown in Figures 17 and 18 of the H-S Report.

While the interaction of the perimeter columns with the spandrel panels in the CTV building may have been a contributing factor in the final demise of the structure, this was neither the trigger nor the cause of the collapse. The exterior frames where the spandrels were present were more lightly loaded than their interior cousins. This lighter value of axial load reduces the P-delta instability effect. It will be shown later that the more heavily loaded interior columns were more critical.

When future analyses on the CTV building are conducted, it is essential that he spandrel effects are modelled directly with the use of gapping elements that mimic the opening and closure of the clearance gap between the columns and spandrel panels. Unless this feature is modelled accordingly, it is not possible to know whether the interaction of the exterior columns on Line F in particular (the basis for the main collapse hypothesis proposed in the H-S Report) was instrumental in initiating the structural collapse of the CTV building.

1.6 Separation of the floor slabs from the North Core.

It is agreed that the separation of the floor slabs from the North Core is problematic.

JUSTICE COOPER:

Q. Just tell me what that sentence means? I don't really grasp it.

A. Well I'm basically agreeing with the H-S Report contention that it is a problem.

Q. It's a problem that is shown to have existed?

A. Yes, yes and I'm kind of agreeing here that this is a problem.

Q. I wasn't sure who you were agreeing with or what you were agreeing but you've explained that now. Thank you.

A. I think everybody agrees that it's a problem, well maybe some people don't but I think all the other naysayers are in agreement on this actually.

WITNESS CONTINUES READING BRIEF OF EVIDENCE

A. This separation permitted differential displacements to occur between floors. In the alternative collapse hypothesis developed in Section 3 of this submission, it will be shown that in many respects, this detail in the CTV building would perhaps be a necessary feature of several different failure modes, including that proposed in the H-S Report, and so I guess that's what I'm agreeing with Sir.

Notwithstanding this detailing feature due to the absent drag-bars, the history of the CTV building design should be recalled. The original Christchurch City Council permit for the CTV building construction did not require drag bars as part of the design. Just a few years after the construction was complete, the CTV building was up for sale and as part of the sale process, Holmes Consulting Engineers pointed out that drag bars should be installed on all floors.

However, on the design review by ARCL, based on the required resistance for the design-level earthquake, ARCL recommended that drag-bars need not be placed on all floors. On the one hand ARCL have been criticised for not providing sufficient redundancy in their detailing. But on the other hand, ARCL can feel vindicated because the structure survived the design-level Darfield earthquake without collapse which was the main aim of the design.

COMMISSIONER CARTER:

Q. Is it your comment that the construction did not require – that's more in the nature of an omission rather than an exemption from need I presume?

A. I'm specifically saying it was not, okay, I understand your question and I think I would tend to, my own personal view if that obviously I would recommend that they be required but I can understand why others came to the conclusion that they aren't needed because they have taken a minimalist approach to the design and on the one hand Holmes said, basically were saying well let's be cautious put them on all floors and went back to ARCL, they looked internally and said well we can see that it should be on the upper four storeys but calculations show that you don't need it on the lower two. And I think much has been made of these hearings about the drag bars but I do believe that the ones that were installed on the upper storeys probably helped significantly.

1120

Q. I think your point as you describe it now is clear to me. The point that I was trying to draw out. It was not that the City Council did not require these.

A. Well I guess they were –

Q. That they were not on the drawings.

A. Yes then I guess there were errors and omissions all round.

EXAMINATION CONTINUES: MR RENNIE

Q. You were in the third paragraph under that heading, "On the one hand ARCL had been criticised –

JUSTICE COOPER ADDRESSES MR RENNIE

EXAMINATION CONTINUES: MR RENNIE

A. Let me start on 1.7 shall I?

Q. I think that's the place to go.

A. Okay thank you.

WITNESS CONTINUES READING BRIEF OF EVIDENCE

A. "Accentuated lateral displacements of columns due to the asymmetry of the shear wall layout.

Similar to the issue in section 1.5 above while the accentuated lateral displacement of the CTV building columns may have contributed to the collapse of the CTV building this factor has been overstated. This overstatement has also been noted by Professor Priestley.

Although pushover analyses have been conducted by Compusoft on the CTV building the results were not put to good use to give further insights into the performance of the CTV building. The NLTHA also conducted by Contrasoft was in 3D. Consequently all torsional and eccentricity features are automatically captured in a more rigorous advanced method of analysis.

One simple method to check whether the lateral-torsional coupling effects are significant is to apply the results of the pushover analysis. In theory it is possible to take the pushover curves of a structure and normalise them then superimpose to normalise pushover curves on the Acceleration Displacement Response Spectrum (ADRS). By performing this analysis a Single-degree-of-freedom (SDOF) simplification can be made of the structure and then used along with an appropriate damping factor for modelling the overall system hysteretic behaviour in order to infer the maximum earthquake response of the structure." If I might add that Mr Kehoe who talked about this and I think most people would have been lost on this point but he did point out that his firm in California were and particularly there's a chap there called Sig Freeman who invented this whole method was a very handy method where you can tease out a lot of information essentially a simple hand method of analysis for getting non-linear behaviour and it's far superior to ERSA but it's essentially using ERSA type principles but in a non-linear domain.

WITNESS CONTINUES READING BRIEF OF EVIDENCE

A. "When performing this simplified 'capacity spectrum' (SDOF-ADRS) method of analysis on the CTV building the results agree remarkably well with a non-linear time history analysis. They do not indicate that the translational displacements are significantly amplified by lateral torsional or eccentricity effects.

1.8 Accentuated lateral displacements due to the influence of masonry walls on the west face.

The west wall of the CTV building was damaged due to two factors. First the Darfield earthquake left the west part of the CTV building badly cracked. This damage and the additional aftershock damage along with more damage that was most likely sustained as a consequence of the demolition of the adjoining building would have left the integrity of the west wall of the building impaired. Eyewitnesses even reported seeing daylight through portions of the west wall. Consequently subsequent to the Darfield earthquake the west wall and the frame was essentially unfit for purpose of providing a substantial degree of seismic resistance." Now I realise that this wasn't intended to provide seismic resistance but in other words it upset the integrity of the whole wall system due to disturbances there. "Evidently the deteriorated condition of the west wall was so serious that it was necessary to implement repairs which were still being undertaken at the time of the Christchurch earthquake.

Second, the lack of integrity of the west wall may have contributed in promoting the unseating of the east/west beams along column lines two and three of the CTV building. It is hypothesised that the pull out of those beam connections into the west wall region led to the unseating of the beam. As a consequence the gravity load normally carried down the west wall frame would need to be resisted elsewhere. This load would be mostly transferred to the neighbouring interior columns along line B of the CTV building. Additional axial load demand along with the exceptionally high vertical ground motions would then cause a P delta type of instability of the columns under sized weight. This may well have been a principal cause for the damage of the CTV building in the Christchurch earthquake. This collapse concept is analysed further in section three of this submission.

And 1.9 which is not on this slide but it's the last point, "Limited robustness and redundancy.

The CTV building did have a limited degree of robustness and redundancy. This conclusion is based on different reasons than those set out in the H-S report. The CTV building was a one way slab system which is relatively unusual in the modern era of buildings. In two way slab systems the floor deflections are significantly smaller as the slabs are stiffer. But the integral more robust and redundant two way slab-on-beam floor systems are also more expensive and slower to build. Cast in place concrete is normally used for the two way systems, a slower more labour intensive construction method. In contrast the CTV building was in a different class of structural systems that consisted mostly of one way frames and floor systems. The modern building era has adopted a variety of precast modular building systems", and these I might say kind of evolved out of that era in the mid eighties.

"As explained in section 1.1 above the CTV building survived its design event, the Darfield earthquake, but for a structure of the CTV building to survive a substantially larger earthquake such as the Christchurch earthquake more robustness was necessary. One key item missing in the CTV building was a series of north/south support beams between the columns. Such support beams although not a requirement of the design and building codes of the day would have improved the diaphragm transfer mechanism and inhibited the possibility of out-of-plane buckling of the slabs along east/west yield lines. This is discussed further in section three of this submission.

HEARING ADJOURNS: 11.27 AM
HEARING RESUMES: 11.45 AM

EXAMINATION CONTINUES: MR RENNIE

Q. Now Professor, we're now at section 2, page 15 of your evidence, "Supplementary investigation work conducted on the CTV building collapse" is the heading.
A. Thank you Mr Rennie.
Q. Can we go to slide 6 please and you'll discuss ground motions?

A. So, well maybe we could go back to the previous slide please. So the second major section is basically divided into five different topics:
firstly the ground motions,
then we're going to talk about the concrete testing,
and then some additional concrete testing on the CTV building columns themselves,
and then there's going to be a discussion on performance of the columns based on some analysis,
and then some problems as identified with the beam column joints.

So first of all 2.1 ground motions and slide 6.
So I've just put what I'm going to read out, I've put on the slide just for your reference because it's like alphabet soup getting your head around all these letters. So surrounding the Christchurch CBD are four ground motion recording stations as part of the Geonet monitoring platform. These stations are :
CCCC at the Christchurch Catholic Cathedral College
CBGS at the Botanical Gardens
CHHC at Christchurch Hospital and
REHS at Resthaven Home.
In the computational modelling using NLTHA, Compusoft used only the first three of this suite of stations; the REHS station located on Bealey Avenue was not used, ostensibly because the soft soil deposits, possibly peat, was near the surface. However, much of the CBD had pockets of such soils, and it seems premature to discount the REHS station for this reason alone.
It remains unknown as to what the ground motion was exactly like at the CTV building site on 22 February 2011 when the Christchurch earthquake struck. The best practice is to analyse the CTV building structure at the other recording stations, as if the CTV building was located at those sites, then infer from the results any trends and likely outcomes that may have occurred at the CTV building site on the corner of Madras and Cashel Streets. Naturally the more results one can

produce, the more robust the inferred outcome; greater confidence can then be placed in the resulting conclusions. It was therefore inappropriate to remove the REHS recording station *a priori;* the REHS station should remain as part of the four-station suite of earthquakes until such time that sufficient evidence is compiled to remove it.

Dr Brendon Bradley was commissioned by Buddle Findlay to conduct an investigation on the statistical significance of either keeping or removing the REHS site from the suite of ground motions. In his report, Bradley presents a thorough analysis of the four-station suite of ground motions. As part of his analysis, Bradley employed empirical ground motion equations showing a range of results that may be expected at the CTV building site. Bradley's empirically predicted range was then compared with the actual results from the four Geonet recording stations. With some minor exceptions, the range of results, within the probabilistically defined range for the CTV building site, is well captured by all four ground motion stations, as shown in figure 2.1 for the Christchurch earthquake and the Darfield earthquake.

Q. So we now go first to slide 7.

A. Now in some ways I can understand why the Compusoft were advised to remove this, because if you look at this kind of spill-over here at one second. But two things should be noted regarding this, first of all the REHS site, this is really one peak in the ground motion response that causes this maximum response and then the subsequent cycles are more normal and the reality is that the building doesn't vibrate at this period anyway, like it will yield at lower amplitudes of excitation, and in the process of yielding the period lengthens so it's more going to be round here where actually the CCCC is bigger than the REHS and then in the north/south direction it's more likely to be around here that's providing the excitation demands on the structure and you will notice in this region in here all of the four stations are more or less within this conditional bounds that come from stand their ground motion modelling for the whole of New Zealand.

Q. And go to slide 8.

A. And then certainly one would be remiss in not analysing how the CTV building was operating in earthquakes prior to the fatal February earthquake and one will note that even at one second this is no exacerbation here of the ground motions of the REHS. All four are more or less within these plus or minus one standard deviation bounds, as was what was pretty much what would you expect. Dr Bradley himself is going to be talking following me but he will explain why these points amplify out here. This is due to the basin effects in Canterbury.

JUSTICE COOPER:

Q. So that's about two and a half seconds?

A. Yes, and then at about one and a half seconds you do sometimes tend to get some local amplification. This is thought to be somewhat to do with the soft soil deposits in the upper 20 metres of the earth crust.

EXAMINATION CONTINUES: MR RENNIE

Q. Reading from "although the CTV building."

A. Although the CTV Building had a natural period of vibration in the order of 1.0 seconds, due to inelastic response the Compusoft results show that the CTV building vibrated at about 1.5 seconds and 2.0 seconds for the east/west and north/south directions, respectively. Over this particular range, the four ground motions fall mostly within the sixteenth to eighty-fourth percentile fractals of probable behaviour at the CTV building site for both the Christchurch earthquake and the Darfield earthquake. Based on this analysis, there appears little reason to remove any one ground motion station from the suite.

Finally, Bradley went on to conclude that the ground motions observed at the Christchurch Police Station, Westpac building, and Pages Road Pumping Station are not considered appropriate for application at the CTV building site.

Notwithstanding the above, ARCL installed a strong motion CUSP accelerograph that is compatible with the Geonet network at the CTV building site. Several substantial ground motions of earthquakes up to magnitude 5.2 have been recorded.

From these records the ground motions were analysed by Dr Geoffrey Rodgers. I might add that Geoff Rodgers is a former PhD student of mine and he now works partly at University of Canterbury and partly at the Christchurch Hospital doing bone mechanics. His work was commissioned by the author and Buddle Findlay on behalf of ARCL. Dr Rodgers analysed the elastic response spectra for the earthquakes with magnitudes greater than 4.5. In figure 2.2, the results of the largest three earthquakes are compared with their companion Geonet records observed at the four CBD recording stations. It should be noted that for two of the three events presented there was one Geonet sensor inoperative – these results were not selectively removed.

Q. Just pause and we'll have slide 9 please.

A. There are actually six graphs that you will see. They come in pairs. The left-hand side is the horizontal ground shaking and the right-hand side graph is the vertical motion and I'll make some comments now.

This is the – the one that's come up first, this is the largest that was measured on 26th of May, magnitude 5.2. Now admittedly this is somewhat smaller than some of the earlier damaging earthquakes but nonetheless it's indicative that the ground motions are more or less pretty much all in harmony. They're not the same and you wouldn't expect them to be the same, but they're similar so let me read.

As shown in figure 2.2, the CTV building motion generally lies in between the CCCC, okay so the CCC is the green line on the graph, up here, you can look at the legend, and the CTV measured one is in black, so the black CTV, generally lying between the green and the yellow lines. So the yellow line is the disputable REHS site. Well I should say it "was" disputable, like I think there's been a change in view or a change in heart on this, and it's now accepted that it's appropriate to have REHS. So perhaps on reflection this similarity should not be surprising as the CTV building site lies midway between the CCCC and the REHS stations, forming almost a straight line of sight back to the epicentral region of the damaging earthquakes.

1155

Q. Just pause there and we'll have slide 10. These are the other four that you were referring to?

WITNESS REFERRED TO SLIDE

A. Yes and so these are not, these were not in chronological order. They were put in order of biggest first, and then this is the medium one and then the smaller of the three major earthquakes. Now we got many earthquakes with similar results, but what we've done is just chosen the biggest three for sake of illustrating the point, and in spite of that Professor Priestley kind of took contention with extrapolating from these results out to the, what would've happened, and all I can say is this is the best we can do. I think anybody would've done the same. It is true that we still don't know what happened at the site on the day but at least we've got confidence now that these are not that markedly dissimilar to discount anything, and if I was to choose any two, and I know the more the better, but if I had to only choose two stations I would choose CCCC and REHS as the prime indicators. They basically do similar things and one sometimes is higher and the other one is sometimes lower. But there's, and it's truly random from that point of view. So they're quite appropriate to use.

Q. Read on?

A. From the results in Figure 2.2, the following two key conclusions can be drawn:

First, the three records that were used by Compusoft to analyze the likely response of the CTV building should continue to be used. Further, the REHS station ground motion that was not used by Compusoft displays similar responses to the other three recording stations (CHHC, CBGS and CCCC) those ground motions that were used. This confirms Bradley's assertion that there was no valid reason to exclude the REHS station from the Christchurch CBD station suite; the REHS site should be included in the four-record suite for any future analyses. And I can just make the comment that as part of the NTHA exercise we have indeed done that.

Point 2, the CTV building records are all evidently bounded by one or more of the other earthquakes in the four-station suite of CBD ground motions. This independently validates Bradley's class of conditional seismic demand envelop that also encompasses the CTV building site.

Q. Just pause Professor and we'll go to slide 11?

WITNESS REFERRED TO SLIDE 11

A. Okay so, that's right, this is a new one. But basically this is showing some matters while I don't want to talk about some matters arising from last week and the point I'm heading here is that some have claimed that the Darfield earthquake vertical motions were not exceptionally high. Is this true or false? So one can really make a view that they're not all that exceptional and that would be true if we had structures that were not prone to vertical excitations for some of the various developments. Like you'll notice here that if you compare the right-hand graph with the left-hand graph, and these are both drawn on the same vertical scale, that when the periods are smaller than about 0.4 or 0.3 seconds the ground motions are quite high, and they are indeed more than two thirds of the vertical motions in the horizontal direction. If you go, we'll be talking some more about this later on.

JUSTICE COOPER:

Can I just read into the record that this is BUI.MAD249.0531A.11

MR RENNIE:

0.11 and then there's 12.

JUSTICE COOPER:

Q. There's another one.

A. Oh, thank you, I almost forgot about this one but, now this is the upper two graphs are again the left is always horizontal and the right is vertical, and then the upper two graphs are the Darfield earthquake and then the lower two graphs are the Christchurch earthquake and again Professor Priestley and also Mr Holmes weren't all that excited about the Darfield earthquake being all that fantastic. Well it actually as you can see there on the left, it pretty much met right across the spectrum the NZS4203. On the right it is true that it's less than the two thirds, which is the horizontal dash black line, or the Z shaped black line, but at the very left-hand end you'll notice that the accelerations are up there, in particular in the REHS, they're somewhat above the two thirds, and so I think to me at least this is ample evidence that I guess there were no people in the building at the time this earthquake happened, but had they been there sitting at their desk they would've got a really good fright and they would've got a big shake up. They wouldn't have quite gone airborne but in the second earthquake I think anybody that was in the building literally would've gone airborne. They would've been thrown off the floor because the ground motions in the vertical direction exceeded 1 g and in fact these are some of the highest response accelerations ever recorded by man. So they're quite exceptional. It is very true what is in the DBH report, that these were exceptionally high and nobody can dispute that. Interestingly though, it settled down, it settles down fairly quickly to the right of that, and that's partly because the, in the left-hand end everything is highly vertical because the earthquake, so we're virtually directly underneath the city of Christchurch, and so that is essentially why it's a major, it's been a major problem in the response of the building, and I believe that it's those vertical motions that led to quite serious signs of damage in the building prior to February 22nd and those signs were not getting any better. Some have claimed well, you know, people are going to be more alert to earthquakes. I think that's true, but it should be pointed out that the human body is an absolute fantastic vibration sensor. Our bodies can detect accelerations in excess of one or 2% g quite easily and if we get enough of these types of excitations we calibrate our own body in a relative sense of what is stronger and what is weaker. And so people knew after Boxing Day for example, that when things did vibrate they were vibrating more strongly after Boxing Day than before Boxing Day. And when you think about it again, this is not surprising. It should be no surprise, because that particular earthquake, the Boxing Day earthquake, was virtually beneath the CBD, right underneath Cashel Street I believe, and so there's clear evidence that the waves would've come straight up from, from beneath and shook very violently in the vertical direction and not so much in the horizontal.

Q. Can I just read into the record, this has the suffix 18.12. Can I just confirm, the top two on the left and the right, they are readings from the September 2010 earthquake are they? Or have I got that wrong?

A. Yes, yes that's correct. That's correct.

Q. What, is that a date code after the letter T?

A. Greenwich Mean Time Sir.

Q. Sorry?

A. It's Greenwich Mean Time.

Q. Oh, it's Greenwich, so that explains why it ends with 03?

A. It's basically a day out, you have to add a day onto it, convert it to local time.

Q. So it's the, when you were referring to the high degree, the very high degree of vertical acceleration, that is

A. I bumped the mouse –

1205

Q. That was about – you were speaking of the February earthquake were you?

A. Yes, yes, let me be very clear on this, because I think it's quite important.

Q. Now we've gone back, is this where you wanted to be?

A. Well – here we are. So this one here, this is the first earthquake and it's –

Q. Well let's just work out –

A. It's not exceptionally high.

Q. Just pause for a minute, because when we're reading this later we will need to know what this one here is.

A. Okay, sorry, I'll try to remember.

Q. So you were talking, we're back on suffix 18.12 and you're addressing the top right-hand graph?

A. Yes, this is the September vertical motions.

Q. Yes.

A. They are high, they are very high but I wouldn't use an adjective to say they are exceptionally high. These are extraordinarily high –

Q. You're now looking at the –

A. And this is the (inaudible 12:06:26) February earthquake and these are extraordinary –

Q. Well I follow all that. I just wasn't clear from your earlier words, thank you.

EXAMINATION CONTINUES: MR RENNIE

Q. Thank you, now we're next moving professor to section 2.2, concrete testing and I think you've, I was going to say I think you've twice given us a glimpse of this slide by accident. Could you now explain this, this is slide 13.

A. Yes, thank you Mr Rennie, we've kind of already started talking about this before, as the fact that when one, I might add that I – my job as a professor, one of the things you get a lot to do as part of your professional service is get sent papers to review from all over the world and so I do a lot of that and you get very familiar with looking for things that tell telltale signs about whether somebody has done something correctly or perhaps not so correctly, and –

Q. Just for the record this is from the DBH report and the reference point is 0189.39 with some material added to it by you.

A. That's right. So what I've pointed out is in that normal distribution curve, the bell curve, on the left-hand side I've shown that there is a 33 percent of the concrete is weaker than specified. That's the area inside the red zone there. I've red stickered it, and then with the two question marks showing 15 megapascals, that's basically showing that as I mentioned before, that if the concrete was indeed that weak one could tell that by observation. One does not really need to do a test to get information out there, and yet they've plotted this as though it's truth, whereas in fact they have no test results to substantiate this and that's particularly concerning, like as a reviewer if I was doing this say for ASCE or another highly reputable journal, I would be tossing this back to the editors to send it back to the author for correction.

Q. Thank you, now reading from 2.2, "As discussed in section 1.4."

A. So concrete testing.

As discussed in section 1.4 of this submission or its evidence, CTL tested eight 145 mm diameter core specimens, where the coring was performed parallel with the longitudinal axis of the column segments retrieved from the CTV building site after its collapse. The cores were sent to an independent concrete testing laboratory in the United States for compression strength testing by CTL, and the testing was conducted in accordance with international best practice.

The 145 mm specimens were further cored down to 99 millimetres to obtain standard test proportions, and then I list in ascending order, the eight results. The above results were normalised and then plotted in the form of a cumulative distribution as shown in figure 2.3. The process of the normalisation is explained in the following.

First, it should be noted that the columns of the CTV building had higher strength concrete in the lower storeys. Specifically, levels 1 and 2 specified 35 megapascals, level 3 specified 30 megapascals concrete, while all other concrete in the CTV building was specified to be 25 megapascals, including the column concrete at levels 4, 5 and 6. It was not known where in the CTV building most of the reclaimed columns were originally located.

It is reasonable and logical to assign the highest specified concrete strength, 35 megapascals to the two highest test results, the intermediate strength to 30 megapascals to the third ranked test result, and the most common concrete in the columns are 25 megapascals to the five remaining test results. These results were then normalised using the following relationship.

The cumulative distribution of these normalised results is plotted with the staircase line, and if we can go to the next slide please.

Q. Slide 14.

A. In figure 3.

Q. That should read figure 2.3.

A. Thank you. A two-parameter lognormal distribution has been fitted to these results using median of f ’c = 1.5 times the specified strength at 28 days, and the lognormal standard deviation which is the standard second parameter for a log normal distribution of Beta is equal to 0.23. The lognormal standard deviation is approximately equal to the coefficient of variation of a normal distribution. Good agreement between the observed test results, the staircase line shown in red, and the empirically fitted distribution via a standard equation that gives the smooth curve as shown in blue is evident.

And just a comment on that, the staircase-type line in red exists because there are only eight samples. If there were many more test observations, and those results conformed to the above calculated modelling parameters, there would be many small steps in the staircase and the red line would tend toward the smooth mathematically modelled blue curve.

The general result that f ’c is equal to 1.5 times f ’c specified should be no surprise for two reasons. First, a well known and common recommendation for evaluating the strength of existing aged concrete is to take the assumed strength at 50 percent above the specified strength. Second, the dispersion of the results, that is Beta is equal to 0.23, is quite similar to that for concrete, where the co-efficient of variation can vary from 0.15 to 0.25.

Based on the evidence of the CTL physical test, the comprehensive forensic tests of the concrete material from the above analysis, the following may be concluded for any future analyses, including NLTHA: that is the in-situ strength for the CTV building should be assumed to be f’c = 1.5 f’c specified.

When conducting – if we could have the next slide please.

Q. Slide 15 please.
A. This is actually written here, when conducting an advanced analysis such as NLTHA, it is always prudent to perform a few swing analyses to examine the sensitivity of the overall outcomes to values adopted for certain key parameters. In the case of the CTV building, the concrete strength is a very important parameter, largely because the columns are compression-critical. It is for this reason that the lower values previously used by Compusoft should be retained to model the extreme possibility of weaker concrete. The Compusoft analyses used concrete strengths amplified some 10 percent above the specified strength. With respect to the median concrete strength observed in the CTL tests, the Compusoft assumed concrete strengths fall approximately on the tenth percentile of the distribution.
Q. And it should be the blue curve in figure 2.3?
A. Yes, which we've already looked at, so let's – shall we go back to that? That's the previous slide number 14, thank you. So if you were to look down here, this is the red, the initial red staircase, the fifth percentile comes in about here and that's fairly close to the specified strength and what, where they've – these have been analysed at the moment for way down in here so really what the initial analysis done by DBH in many respects one can argue that it is giving a lower bound solution which is going to give perhaps an unduly harsh prediction of what was expected to happen in the response of the building. That might be fine if you're doing a design of a structure but when one does forensic analysis the aim is not to be conservative by design. The aim is to search for the truth and therefore one should give the most probable result and using a median or expected value is really what one can do. So slide 16 please.

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WITNESS CONTINUES READING BRIEF OF EVIDENCE

A. "2.3 the additional concrete testing on CTV building columns.
An inspection of the Burwood dump site revealed that there were several columns remaining from the CTV building that were in relatively good condition. The columns were evidently from the sixth floor level

and thus would have a specified strength of 25 MPa. As part of the more comprehensive forensic analysis on the CTV building collapse it was considered essential that these columns be tested in a full scale condition. Three specimens were retrieved by ARCL and taken to the University of Canterbury structural laboratory for testing under concentric axial compression as seismic strain rates in the 10MN Dartec universal testing machine.

One purpose of this part of the investigation was to compare the results obtained from the CTV building columns with similar well known test results on unconfined and confined concrete column of the 1980s. The work has been reported in Mander 1983 and Mander et al in 1988 a and b. In those early University of Canterbury tests Christchurch-sourced ready-mixed concrete and steel reinforcing materials were used, similar to the materials that were later used in the construction of the CTV Building. In the comparative test evaluation, the aim was to investigate whether any unusual surprises in performance existed—especially when tested under dynamic loading rates.

A second purpose of the full-scale testing was to investigate any size effect that may have been present. The so-called "size-effect" in concrete structures is based on the fact that when the size is increased by a factor of 4 (from the 100 mm diameter test cylinders, to the 400 mm diameter prototype column), the failure stresses are not the same. Empirical evidence shows that the larger scale leads to a smaller failure stress; in simple terms this reduction can be thought of as being akin to the weakest-link-in-the-chain theory. In the case of the University of Canterbury tests performed in the 1980s, the size effect was found to be a 15% reduction in capacity.

A third purpose of this testing was to examine the performance of concrete column elements that exhibited a poor post-collapse condition. The third specimen yet to be tested" and this is at the time of writing, "Visually appears to be in poor condition; the concrete may have been damaged, either from the collapse or the fire."

Q. Slide 17 please.

A. So there you see the three specimens as retrieved from the steel. They are sitting in the lab at the University of Canterbury. The two outer ones that are coloured purple clearly those are the specimens that are in relatively good condition. The ends have been cut square. The central one C13 you'll see with paint squirted on it was not in such good condition. Now it's not clear as to why that was not in good condition but my supposition is that some of the damage that we see on the end, one end was near one of the joints and the other end was I think was cut near some damage but it may well be that that had suffered quite a lot of damage in the fall, in the collapse itself.

WITNESS CONTINUES READING BRIEF OF EVIDENCE

A. A photograph of the three specimens prior to testing is presented in figure 2.4. The damaged central column shown in the photograph will be used to develop the third test specimen." And what we did there I might add is we actually, this is quite common practice to do this, we casted special ends of strong concrete, rings around the bottom so that that wouldn't perpetrate a premature failure in the end zone so that we could actually get failure in the mid region of the column.

WITNESS CONTINUES READING BRIEF OF EVIDENCE

A. It is expected that the performance may be inferior, but as to what degree it is not clear. The results of the third specimen test may give some insight into the low concrete strengths inferred as a result of the Opus testing. Dr. Rajesh Dhakal was commissioned to conduct the testing in the Dartec universal testing machine at the structures laboratory at the University of Canterbury. At the time of writing this submission, the results of the final specimen test along with further analysis are not yet complete. Provisional results from the first two tests reported to date show the following:

Q. Slide 18 please.

A. The concrete strength is above the specified value of $f'_c = 25$ MPa. There is a size-effect present. This may be in the order of $f'co = 0.85 f'c$, where: o $f'co$ = the in-situ strength of the full scale structural concrete;

and o $f'c$ = the standard 100 mm x 200 mm test cylinder strength for the concrete taken from the same pour. Finally, once the testing and analysis is complete, more definitive recommendations can be made on the precise concrete strengths to be used in any future NLTHA on the CTV Building.

Q. And Professor is it your understanding that the further work that has been done that information will come forward in the concrete section of this enquiry?

A. Yes that's a little bit of homework I have to do when I go back to the United States but I believe that was in the next couple of weeks and it will be done by then for sure.

Q. Thank you.

COMMISSIONER FENWICK:

Q. What were the earthquake speeds and does the .85 apply to when through a normal concrete test at the standard rate compared with a large specimen test at a dynamic rate?

A. Okay so what is meant by earthquake speed here is that where basically the originally test that we did some 30 years ago were tested where the buff pattern on the Dartec machine comes upwards essentially at maximum speed which happens to be about 16 millimetres a second. The strain rate that that will give is in the order of .01 or somewhere between .01 or .015 depending on how the actual strain is measured in the specimen which is significantly faster than quasi-static test rates that one would test standard cylinders and so the rough rule of thumb when you increase by that amount you're going to get about a 15 percent increase in concrete strength due to the dynamic strain rates. Now that will be somewhat offset by the fact that when you test a large full scale specimen it will also reduce so in many respects the results come out to be awash and that what you gain through dynamic strength enhancement is also lost by the size effect and the purpose of doing these tests really is to more compare them with what we did 30 years ago and my point is that very simple is that this is

Canterbury concrete. It was probably made from, some of it was probably made from the same concrete pit that we used as I recall we used Ashby's which is down Wairakei Road back in the eighties when these specimens were made at Canterbury. I don't know if that was used for the manufacture of the concrete in the CTV building but all those Canterbury gravels are mostly greywacke. They're very similar so what I'm trying to argue here is that we're trying to create a level playing field of competition between what was then and what is now.

Q. I understand that but you're saying that the increase size offsets the rate of loading and so we can come to a conclusion that if tests are 25 megapascals you're saying the likely strength would have been 25 megapascals times 1.5?

A. Yes in fact if you go back and read my earlier work that's generally always the case that there's a 15% loss between prototype and cylinder tests. That's quite normal for this sort of size that we're talking about.

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Q. We're talking at cross-purposes. You said elsewhere due to the desire to make sure not more than one in 20 specimen fails the concrete supplier aims at a higher strength –

A. Yes, yes.

Q. 20% up or thereabouts –

A. Yes.

Q. – and then you have an ageing effect –

A. Yes.

Q. – which you said now it should assume that the actual concrete strength is 1.5 x the nominal strength. What I'm putting out is what you found here is the actual strength you've got is the design strength not 1.5 times that? It may not be significant. I'm just pointing out.

A. I do understand this Commissioner Fenwick. It's very complex I must say and how you look at this is quite complicated and what I hope to do is fully clarify this when we talk about the concrete more specifically but for analysis in spite of what in situ concrete strength is for analysis it's been shown that you should generally revert back to using just the standard *f* prime c. Probably another way of explaining this is that you will well recall that when you analyse a column and draw a column, a moment interaction diagram under concentric axial compression you always modify the axial compression load by .85 and a lot of people misconstrue that as being something to do with, you know, either a Beta factor but it's really there for a size effect so the size effect it appears more pronounced admittedly if you do concentric axial compression on full-scale specimens but then again if you then go to use that and say bending the same concrete then if you use the reduced strength that you use from a large-scale test, it wouldn't give you the same results as if you use the cylinder setting. Standard cylinder strength would end up giving you more correct results when you kind of back reconcile what the results are saying.

Q. Is that an answer to your question whereby you would have expected the concrete strength of the cylinder to be 1.5 times the value of the design value and yet here the tests show that that concrete strength taken from the cylinder under compression came out 25 MPa but when you allow for the size effect and the rate of loading. I don't think you answered my question.

A. I'm sorry. Okay, so the cylinders that we tested were actually in situ concrete. So they weren't cylinders that were cast that were set aside. Now admittedly they weren't the same columns that we tested.

Q. Can we just refer, not to your tests of 20 years ago –

A. No, no, I'm talking to you about

Q. – but the tests on the CTV.

A. No, I'm talking about these cylinders, the ones that we just read about.

Q. Which ones did you just cast that you took from the CTV building?

A. Sorry.

Q. Which cylinders have you just cast that you actually extracted from the CTV building?

A. We didn't cast any cylinders.

Q. Well you've just mentioned the cylinders –

A. No, I'm saying, sorry, the ones that we tested.

Q. - that you've recently cast.
A. The ones that Haavik tested in Colorado were bored down the axis of columns right and it is admittedly it's a mystery to me that it could well be that some of these full-scale tests are going to show that the concrete is weaker in its full-scale sense and that is a mystery. I haven't reconciled that and I have to think about that but, as I say, I am about to analyse the results.
Q. I look forward to seeing the analysis.

EXAMINATION CONTINUES: MR RENNIE
Q. I think we've reached 2.4 – Column Performance Analysis

WITNESS READS BRIEF OF EVIDENCE FROM LAST PARAGRAPH OF 2.3
A. Finally, once the testing and analysis is complete, more definitive recommendations can be made on the precise concrete strengths to be used in any future NLTHA on the CTV building.

2.4 Column Performance Analysis

Figure 2.5 presents the CTV building under the sidesway motion effects of an earthquake

WITNESS REFERS TO SLIDE 19

For the east-west components of ground motion, the CTV building really exists in two parts: the frame (i); and (ii) the shear wall systems which consist of the south shear wall and the North Core. To view more simply how this dual system functions, Figure 2.5(a) shows an elevation of a typical interior frame connected through rigid links to the wall system. The links provide the in-plane connection that represents the floor diaphragm. The aim of the present analysis is to examine how half-high column components would function under the same type of lateral displacements as shown in Figure 2.5(b).

Earthquake structural engineers commonly refer to the inter-storey displacements as the “drift” or, more strictly, the total storey drift, Theta T is equal to Delta divided by H_s where:

Delta = to the displacement of one floor with respect to the floor below; and

H_s is = to the storey height or the distance from a given floor to the floor level below.

As shown in Figure 2.5(c), during an earthquake the drift on any one storey in the frame (Theta T) is imposed by the displacement compatibility with the North Core (shear wall) and the South shear wall of the CTV building – both the frames and the walls are constrained to have the same drift angles.

WITNESS REFERS TO SLIDE 20

Q. That’s what you’ve just referred to?

A. Yes, so basically that picture there I might add that with respect to the previous one which was the undeformed state of this, this is essentially how we would have had to analyse the CTV building less than 10 years ago before we had these non-linear time history analyses and non-linear capability in 3D. We have had 3D capability for many years elastically but only comparatively recently in 3D and I think that's one of the reasons why there’s been a lot of teething problems getting the modelling right but in the good old days this was how it was done and it’s still convenient to look at how the building would operate if you had to string it out and analyse it in this fashion so what you do is you separate the walls from the frame and then join them together with rigid links. Now those are shown in green. Those rigid links are essentially representative of the in-plane floor slab forces that one would measure and what I’ve show is the red dots are hinge locations in the columns. Those, as per the CTV building’s sidesway mode and as given somewhat initially, well in the initial analysis of the DBH would infer that this is what you would expect pretty much to see. You would expect to see some hinging down in the bottom of the wall. You would expect to see some hinging at the ground floor level. You would expect to see some probably positive hinge pull-out type hinges here at the ends of the columns at the roof level or 5th floor level, most probably at the head of the columns and then each column. Now this is really all that could be done in the previous analysis because the joints weren't modelled so they're really either wittingly or unwittingly using column hinges as a surrogate for what's going on in the beam column joints.

The NLTHA results presented in the Compusoft report (and if we refer to levels 2 to 5 in Figure G6) they show that the walls swayed laterally essentially at a constant drift angle with respect to the ground. It is for this reason that the performance of the critical elements can be determined by seeking the critical columns in the structure through modelling the components as in Figure 2.5(b) – each column is constrained to have the same displacement field, only the axial load will change as the storey changes.

WITNESS REFERS TO SLIDE 21

The total drift angle (Theta T) on the system is composed of three components as follows; and the equation is there and also it's on the bottom of the slide and the figure shows the deformed shape which was taken as part of Figure 2.5(e).

The interstorey drift, well let me just define these components. So theta T is the drift, it's this displacement at the top with respect to the overall height, and there is an angle between this tangent that goes through here. This angle is theta B, the middle one, and that's to do with the deformation in the beam which I've drawn of course in a very exaggerated fashion, and then there's the angle that the column is actually introducing through bending as shown, and then it's very hard to see and it's very hard to draw but inside the joint here there's going to be a rotation on the joint, gamma J which is due to the stresses that come in from the beams and the columns that distort the shape of the joint. So if the joint as shown here was square, it's going to be distorted into more of a rhombus shape. The interstorey drift for each level is essentially constant, thereby putting the columns into double bending with inflection points at mid-storey height. Thus over the height of the walls, the subassemblage representations shown in figure 2.5(b) and also figure 2.5(d) and 2.5(e) are a reasonable approximation of general seismic performance.

For a subassemblage over the lower four stories of the CTV building, an analysis shows that for the interior connections, the columns are weaker than the beams. Therefore, "pushover" analyses on a half-high column components in figure 2.5(b) have been conducted for the interior columns as single column components. The results are presented in figure 2.6 and figure 2.7.

Q. Can we have slide 22 please?

WITNESS REFERS TO SLIDE 22

A. So slide 22 here, figure 2.6 presents the results for the case where only normal gravity load exists on the columns; that is, there is no concurrent amplification from vertical earthquake motion effects. An analysis was conducted for each storey using the two concrete strengths. In the left-hand column, results are presented assuming the specified concrete strength + 2.5 megapascals. Recall that this is also the strength Compusoft assigned in their analyses. According to the more recent test results described in section 1 above, the Compusoft assumed concrete strengths would fall on approximately the 10th percentile of the probable range, so there is a 90% chance that the concrete is stronger than Compusoft assumed.

Just while we're on this figure I would just like to add a few comments. Firstly, I guess Professor Priestley was mildly critical of this, so let me defend this. So one of the points he made was in this upper left graph he said, 23% ultimate drift, that can't be. Well actually I'm not using anybody's arbitrary number of 80% from peak as defining ultimate drift. I'm using the definition that Mr Holmes recommended which is now customary in the United States, is to use a definition whereby the drift capacity more has to do with its ability to carry axial load. Now admittedly 23% is excessive but that's just the computer analysis is pushing it out to as far as it goes and then it stops. And I would admit that that's unrealistically large, but of course the axial load, and this is in the fifth storey, the axial load up in the fifth storey is very modest and so that's why it can go a long way. Let's continue on to the next.

Q. Top of page 27, "The Compusoft analysis case…"

A. If I, if we can bring up the next figure too please at this time?

Q. Yes, slide 23 please?

WITNESS REFERS TO SLIDE 23

A. So the other comment that Professor Priestley made is he said, "Well, you know, this doesn't look right. How can you have this coming out to a peak and then coming –"

JUSTICE COOPER:

Q. Let's just note that for the record that you're referring to bottom right-hand –

A. Bottom right graph thank you.

Q. – graph on WIT.MANDER.0001.70

EXAMINATION CONTINUES: MR RENNIE

A. So he said, "Oh this doesn't look right, coming back like this," and on first glance I thought the same, but I'm thinking is there are two reasons for this, one, with presenting it like this, like it's tempting to stop the analysis here where the cursor is at the moment and then drop a vertical line down which would be true if the column existed in isolation. But the reality is the column doesn't exist in an isolation, there is another half of it not presented here, and in reality all that would mean is that the inflection point would change and that half of a column would, the deflection would actually effectively become smaller and the other half of the column it may become larger, depending on what was on the other end or the details on the other end of the column. So it's not an inadmissible result, it's just a quirk of how it was analysed.

So the Compusoft analysis case is therefore considered to be representative of a lower-bound strength condition. The right column , that's the graphs on the right column, present the results for the median value concrete strength based on the CTL tests, that's 50% above specified strength, and is considered to be more representative of the probable in situ condition of the concrete.

From a general inspection of the results in figure 2.6, it is evident that both the lateral load resistance (which is the same as the shear force in the column for the particular storey) and the deformability of the frame (that is the drift capacity) improve as the concrete strength increases. And again Dr Priestley made kind of remarks that these results were all over the map, well of course they are because the concrete strength varies, and also the axial load increases as one goes down, so what we would normally expect to see is as the axial load increases the column is going to be somewhat embrittled, but as the concrete strength increases also that's going to somewhat offset that.

Where did I get to sir? Second paragraph?

Q. Yes you were at, "From a general inspection…"

A. From a general inspection of the results in figure 2.6, it is evident that both the lateral load resistance (which is the same as the shear force in the column for the particular storey) and the deformability of the frame (that is the drift capacity) improve as the concrete strength increases. An accurate definition of "drift capacity" is difficult to determine, but is generally considered to be when the structure has lost some ability to carry substantial lateral load. In lieu of a more precise definition, "drift capacity" is often taken as the drift when the post-peak lateral load or column shear falls by 20%, or 80% of the peak value.

Figure 2.6 also shows that in the lower stories the deformability of the structure becomes more restricted as the axial load increases with respect to the concrete strength. More specifically, the third floor appears the most critical in terms of strength and drift capacity. This will be examined below for the more conservative of the two cases (f'_c + 2.5 MPa).

Figure 2.7 presents in a similar fashion for those shown in figure 2.6, but with one key difference. The axial loads used here are used in each analysis were taken as the minimum axial loads registered for the CCCC ground station motion –

Q. You read "minimum" but I think you mean "maximum"?

A. The maximum axial loads, sorry, registered for the CCCC ground station, ground motion NLTHA results for the Christchurch earthquake, as depicted in figures 52 to 55 of the Compusoft report. During the Christchurch earthquake, there were large amplifications of vertical axial load due to the extremely high vertical ground accelerations.

Because the frequency of the vertical components is some five times greater than the horizontal response, it is inevitable that the two displacement and force maxima will coincide momentarily, producing extremely high loading and stress demands on the materials. If the materials are overloaded, this means at least some partial damage or breakage of the weakest of those vulnerable components.

When compared to figure 2.6, the results in figure 2.7 do indicate that the structure of the CTV building was vulnerable to the vertical motions as a consequence of the extremely high dynamic axial load effects. It should be noted that this aspect of vertical horizontal load coupling was not correctly modelled in the Compusoft analyses. In its modelling, Compusoft reported the level of axial load and moment, but did not adjust the moment-axial load failure surface accordingly.

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Q. Just pause there, can we have BUI.MAD249.0189.54.

A. You could bring up that graph please and while that's there I'll comment on it but while it's there I'll read the next couple of sentences.

Q. Yes.

A. It is for this reason that figure 15, which we now have on the screen, of the H-S report, displays inadmissible results. Numerous data points are plotted outside the failure surface – such performance is theoretically physically not possible. When the load path reaches the failure surface, and you think of this as a fence, something has to fail, either:

(i) the steel yields; this happens for low levels of axial load in the upper stories under normal gravity load, or

(ii) the concrete crushes, this occurs in the lower storeys under gravity load, and also in mid-height storeys under high level of axial load caused by vertical acceleration effects, that is for axial loads more than 1700 kilonewtons. So this was actually, this graph here is really taken for the lowest storey and the horizontal line that's going across here, I'm pointing to the blue line, that's the average without vertical ground motion effects that would be in the lowest storey column, like C2 for example.

JUSTICE COOPER:

Q. Let's just make it plain, you're talking about the solid blue line?
A. Yes, the solid blue line. That's correct, 1700 kilonewtons.

EXAMINATION CONTINUES: MR RENNIE

A. Now Compusoft analysed, they would have set the strength of that column to be right where I've got the cursor pointing which is at the intersection of the 27.5 megapascal concrete and also the 1700 kilonewtons, and then they would have set the parameters in the hysteriatic model for that. Then they would have run the programme and then plotted results, now it's okay to have these results to the left of that green line and I would well expect there to be some results slightly to the right due to some effects of strain hardening that one would expect to see in the reinforcing steel, but I would not expect to see the results this far to the right, so these results in my view are inadmissible, and then the second point is that if during the analysis the column gets to this fence as it were, it can't get, it should not be able to get through it which means that something else has to give and what's generally happening is that the deformations will increase, the forces won't increase but the – or the moments, the deformations will increase and also the damage will increase, so in some respects this, doing this is kind of unconservative but on the other hand it could be kind of conservative, the point being though is that it cannot trace the failure through to fruition as part of the analysis. Okay, I've lost my place, where am I up to?

JUSTICE COOPER:

Q. You're at, "while the outcome," third paragraph, page 28.

EXAMINATION CONTINUES: MR RENNIE

A. While the outcome in paragraph (i) is desirable, the outcome in paragraph (ii) in contrast may be catastrophic. This is especially true if the column is in an unconfined condition, as was the case for the CTV building. Had the column axial load-moment interaction been modelled correctly, then many yield or failure points would be plotted on the outer curves, not beyond them. What this means is that the post-peak performance of the CTV building, and the consequent redistribution of forces, has not been tracked correctly.
Now I might add as a pre-cursor, the NTHA has been re-run and this time round that has been modelled correctly but I fear to say that the joint hasn't been modelled correctly and in fact there's some respects there's no choice about that. If you were to plot what's going on inside the joint perhaps with respect to its strength, you may get a diagram that's similar to what we just saw, but at least the columns have been done correctly and to my satisfaction I note now that you actually see the results turn round as they should.
Therefore the clearly demonstrated modelling inaccuracies of the failure criteria puts a large cloud of doubt over most of the results in the H-S Report, and in particular the interpretation of the results. This is particularly true for the lower two storeys of the CTV building, where the failure trigger may have initiated. Take for example the column at level 2, by considering the left-hand column graphs in both figures 2.6 and 2.7, maybe we should go back to that.
Q. Slide 22 and 23.
A. Okay, so if we go back to the previous figure.

JUSTICE COOPER:

Q. You want the first of those now?

A. Well they're here, that's fine.

EXAMINATION CONTINUES: MR RENNIE

A. So the left-hand column of graphs in figure 2.6. In both cases, the effect of the higher axial load was to reduce both the resistance, roughly a 10 percent reduction in strength, and embrittle the column. So embrittlement here means that the column has less ductility or deformability capability, specifically a 50 percent reduction in drift capacity.

If there was a sway failure, such as that modelled in figures 2.6 and 2.7, the structure of the CTV building would have attained only modest drift and then collapsed; indeed a collapse following the Darfield earthquake even would have been conceivable. Moreover, there would be supporting forensic evidence of observable damage amongst the rubble. One would expect to observe many columns, at least all of the interior columns of one storey along lines 2 and 3, to fail in this way. The damage would propagate out from the joints and cover much of the length of the columns – yet this was not the failure model for the CTV building collapse, and by that I mean the DBH failure prediction.

So what was the actual cause or trigger for the failure and eventual collapse of the CTV building? Some clues, but not the complete answer, are given in the section 2.5 below on beam-column joints, and a full solution is postulated in section 3 of this submission.

So the next slide –

Q. 24.

A. 24 has got the headings here. Now if you look at page 30 of my write-up here, what I've done is put that side by side for comparative purposes on the slide. So just talking through the slide, on the left-hand column we have normal capacity design, the design strength hierarchy from weakest to strongest. This is what contemporary designers these days, and I might say for the most part back in the 1980s would have strived to do. You would come up first of all by making the beams the weakest link in the chain, that's called beam bending or flexure. Columns would then be the next strongest element but stronger than the beams. You cannot afford to have the joints to fail so they are made stronger again, and then the last thing you want to see go and fail is the foundation. So the CTV building on the other hand in my view, number 1 is the joint shear was critical, followed by column flexure was next critical, beam flexure, they're almost in the reverse order and then finally wall capacity or foundation rocking, this is likely to have been the sequence of strength hierarchy. Now reading through that on page 30, and –

JUSTICE COOPER:

Q. You can take it as read because you've covered it down to the bottom, there are several reasons.

A. Okay.

EXAMINATION CONTINUES: MR RENNIE

Q. Yes, three lines from the bottom professor.

A. Thank you. There are several reasons the beam column joints in the CTV building were the weakest and thus the most vulnerable elements. First the joints have small cross-sectional area, note that the joint shear strength is proportional to the concrete area.
Second there was no transverse spiral reinforcement within the joints surrounding the longitudinal column bars; if present and close spaced such spirals can add substantially to the joint strength.
And third, the shear force demands are significantly higher in the joint regions compared with the adjoining beams and columns.
The problematic high joint shear stress intensity is illustrated via the analysis presented in figure 2.8. First, a subassemblage is extracted, as shown previously shown in Figure 2.5(e). The force actions at the

inflection points (which are at the end of the members shown) are shown in Figure 2.8(a). Next, if the beams are removed, their effect must be replaced by applying equivalent beam-end forces arising from the loads carried by the reinforcing bars going into and out from the joint region, as depicted by Figure 2.8(b). The left-hand side of figure 2.8(b) shows the bending moment diagram (BMD) for the column, including the effects through the joint region. By differentiating the column BMD over its member length, the shear force diagram (**SFD**) is derived as shown on the right hand side. Vjh is the (horizontal) shear force intensity through the joint; calculations show that this will be some 5.3 times greater than the column shear force, ($Vcol$) for the CTV Building.

As the joints do not possess transverse reinforcement, the joint shear resistance is provided by a corner-to-corner arch (or strut) within the joint, as shown in the drawing of the beam column joint in figure 2.9. The magnitude of this joint-strut force cannot be resisted without the concrete within the joint becoming overstressed. Furthermore, reinforcing bars and concrete on the (opposite) off-diagonal of the joint are in tension, and this tensile action causes a progressive weakening of the compression resistance of the concrete within the strut.

On the first pulse of the Christchurch Earthquake, if the inertia forces pushed the CTV building from left to right (as shown in Figure 2.9), the forces in the joint may have been resisted without too much damage to the concrete. But if the axial load in the columns is high, as it was in the lower stories of 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."

Now if I can use the pointer on this slide here so you get compression coming down this side and a compression load path following the cursor down here now like so. On this side there's kind of a companion, tension load path and so you have tension in these bars which will try to get into the concrete through here and it will literally tear the joint apart on this diagonal so you'll get cracks running parallel to the principal 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.

1255

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."

1425

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.

1435

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.

1445

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.

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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.

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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 –

1700

DISCUSSION

HEARING ADJOURNS: 5.05 PM

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