COMMISSION RESUMES ON TUESDAY 6 DECEMBER 2011 AT 9.30 AM

NIGEL PRIESTLEY (AFFIRMED)

COMMISSIONER FENWICK:

Q. Professor Priestley, I went through a series of questions for Mr Jury. I don't know if you want me to go through those again, or perhaps you can remember them. Would you like to comment on any points you agree with or disagree with, the conclusions that were arrived at in those discussions?

A. I'm not sure that in many of the cases I think conclusions weren't really arrived at but there was agreement that these were possibilities. Perhaps I could make some general comments though. I think that the points that you were rightly raising were, if you like, a description of the second phase of failure. There seem to be general acceptance that there were significant problems with the reinforcement ratio being so low that it was likely to have concentrated cracking in one or a comparatively few number of cracks that the consequence of that could be a wide open crack at that particular level or at the critical level at the base of wall of floor at level one, and that the consequence could be fracture of the reinforcing steel. You raised some problems about the cross walls between the east and the west walls in terms of their shear capacity and there was some disagreement there in that Rob Jury came up with values extracted from the computer analysis which seemed less than you had calculated by hand methods and I wondered a little bit whether the reasons for that might have been that the end walls running in the east west direction were significantly stiffer than the central wall with the two wall openings in it and that as a consequence of that there was transfer of shear to the outer walls itself, reducing the shear in the central wall. I think that if there was a tendency for a failure you mentioned vertical shear at the openings, or between the openings, that any tendency for that to occur wouldn't quickly transfer force to the outer walls itself, so that may be part of the reason that there is a difference between those aspects. As far as the torsion that you've mentioned I think that that's definitely a possibility. It wasn't clear to me why you discounted both the east flange and the west flange in that. I would have thought that if it was rocking up with the west flange having fractured and the east flange would have been in significant compression that it would have participated in the shear transfer. I note also that it can be difficult to estimate the influence of this due to the fact that the rotational inertia of the floors itself will tend to control this to some extent, but I do think that that is again quite a possible contributor to the final failure of these walls. But I emphasise that these things to me seem to be secondary rather than the primary aspects and I think that the points that you have raised, which I think are very valid and are important, are an indication of how difficult it is to predict the final death throes if you like of the building. You can often get a pretty good idea of where things initiate but then saying what happens after that is very difficult to do either by hand methods or by the most sophisticated time history methods that we have.

Q. Professor Priestley, if the redistribution occurred to the, say the north transverse wall at the end?

A. Yes.

Q. What would happen then do you think, you're transferring the shear that way, it's now going to give you very high compression force –

A. Yes.

Q. – on the east side, where the wall is offset, where there's an offset in the wall at that level.

A. Yes.

Q. How do you think that compression force, do you think that might have created a problem if you got the redistribution you're referring to?

A. I think so, yes, I think it certainly would have contributed to the – so I think it's hard to determine whether that would be as significant as the same effect in terms of concentrated compression occurring from simultaneous north south and east west. Again it's one of these things that we can speculate about but I don't think we can come to a firm conclusion on – certainly I think that that's a possible contributor, certainly I think that biaxial attack, which we know occurred is a possibility. Certainly I think that the additional contribution of vertical accelerations is a possibility. Personally I think that that's maybe more significant than has come out of the non-linear time history analyses.

Q. You would see that offset in the wall as probably quite a critical key, critical structural weakness then in the performance?

A. Certainly, yes, I wouldn't because even in the most simple calculations you see that at least it doubles the compression stress in that wall because of the softness and about 50 percent of the wall underneath it.

Q. I think we've probably covered the main issues there.

A. Yes I think so.

Q. Thank you.

COMMISSIONER CARTER:

Q. Professor, my mind is turning to the practical outcomes of what we're learning this hearing, particularly in regard to examining older building stock for their weaknesses and we've heard about this particular building having been analysed on two occasions by very sophisticated methods, one of which we were told took 26 hours of computing time. I think when we examine the practice that's been prevalent in New Zealand by councils in looking at how to deal with their existing building stock we've got a variety of approaches from passive actions by councils to active participation in the upgrade considerations. Can you tell us your opinion of the practicality of simple procedures for actually finding out the weak spots, the Achilles' heel in our building stock because there could be a very big task ahead of New Zealand to examine its buildings to the level of finding where their weaknesses really are?

A. Yes I think that's a very good and important question. It's clearly not feasible to do non-linear time history analyses of all the buildings in New Zealand and nor is it necessarily going to come up with values that are significantly different from simpler approaches. As you're aware Dr Fenwick has been looking at simple approaches. I also looked at simple approaches to try and determine what I expected to happen. It's perhaps more what ifs associated with the simple analyses, but I think that they are certainly capable of determining and highlighting the weaknesses, the critical structure or weaknesses in the building and I think that very frequently you can come up with a pretty good approximation to the anticipated response under code level earthquake. The reason really that non-linear time history analyses were done in the Pyne Gould Guinness building in particular in the case of Beca Carter was that we had some pretty reasonable records and some pretty good idea of what happened in terms of the ground motions and the point then was to try and get the best possible estimate of the structural response of the Pyne Gould Guinness building to the rather well known seismicity. If you are assessing an existing building without the knowledge of what it might be subjected to and therefore looking at it in terms of the code requirements then I think it is less significant and less important to perhaps do a time history analysis. You are not trying to tie things down to the last, you know, dotting the last i and crossing of the last t, but I think that you are determining whether there is a critical response to this building and you do not need to go to that level of accuracy.

Q. From your knowledge and your teaching, the teaching part of your work, do you think there is a particular need for us to develop some workshops around developing engineering consistency in examining these, our building stock, to get a reasonably even-handed approach across the country?

A. Yes I think there is an, and I believe that this is underway with the Society for Earthquake Engineering and other institutions but, and I would certainly draw attention to the fact that I believe that there may be significant differences at the moment if you take two different engineers and show them the same building as to what they will interpret the condition of that building to be. To quite a considerable extent that is inevitable just because of the difficulties and the fact that we do not get earthquakes every day so we do not look at buildings every day. This, as I mentioned yesterday, is a unique experiment, if we want to put it in a rather crass and harsh light which has provided a huge amount of information. The profession will undoubtedly learn a huge amount from this, not just in New Zealand but overseas as well, and it will certainly be taken by the profession as a means for upgrading its knowledge and its expertise in assessing the performance and the capacity of buildings, both before an earthquake and after an earthquake.

JUSTICE COOPER:

Q. Putting the various answers that you have given to Commissioner Carter together would I be right in inferring that in your view it would be possible to design a set of procedures under which it would be possible for suspect buildings to be examined to ascertain what their critical structural weaknesses might be?

A. Yes I think you could say that the Society for Earthquake Engineering's guidelines already provides such a means even if it is not a perfect means. I think that what I am saying is that that is likely to be significantly improved and upgraded as a consequence of knowledge coming out of the Christchurch earthquakes.

Q. But there would need to be some regulatory context in which those methodologies were required to be adopted rather than simply waiting for the next earthquake?

A. Yes, so you are saying that there needs to be something which will, to be taken by a territorial authority and saying, "We must look at these buildings of a certain age"?

Q. Yes. Yes.

A. Well certainly it is possible to do that and I think that Wellington has had a system which has had a timeline associated with buildings which are of a certain age or unreinforced masonry to have a look at them and upgrade them. This could certainly be extended and I think would be a good idea.

Q. Do you think that it is likely to be any bright line though in terms of the age of construction of a building?

A. I think there is a fairly obvious break in 1976 with the introduction of modern techniques in design so that would be a fairly obvious cut off point in my view.

Q. There is some indication though, is there not, from the experience of the February earthquake that there are more modern buildings that have critical structural weaknesses?

A. Yes there are but again it should be recognised that the earthquake that, the February earthquake was of an intensity that was very substantially larger than we designed for, so one could argue that the problem was not so much in the design of the buildings but in the provision of the, or the estimates of the seismic intensity and the probability. There are a lot of faults in the Christchurch and Canterbury area which were not recognised before that, before this sequence of earthquakes itself.

Q. Yes, I assume to the extent that this very powerful earthquake has thrown up critical structural weaknesse. The provident thing to do would be to consider the implications of those weaknesses as well for design purposes or am I being too...?

A. I am sorry I did not quite catch the sense of that?

Q. I am saying that to the extent that this very strong earthquake in February has highlighted other aspects of design that might be described as critical structural weaknesses, would it not be reasonable to learn those lessons as well rather than assuming that there will be no repetition of such a strong event?

A. Well I think that that is also inherent in the work that is going on in New Zealand at the moment. I think that there are a number of modern buildings that have not performed as well as we would like and a number of them have had structural weaknesses which we would not have expected them to be there but there I guess will always be some buildings which are not designed perfectly which have problems to them and Professor Tom Paulay, anyone who is an engineer in New Zealand reveres as one of the great earthquake engineers, used to say buildings do not read codes, that is one of the things, and also in earthquakes the earthquake is going to find the critical structural weakness and I am not sure that all designers in the most recent times have perhaps thought that, about that as much as they might have.

Q. Can I just ask you, I think I heard you say yesterday, I may be wrong, that in the case of the PGC building, compression failure of the west wall was the most likely cause of failure?

A. No, of the east wall.

Q. It is the east wall?

A. If I said the west wall that was incorrect. It was the fracture of the west wall, fracture of the reinforcement in the east wall leading to a series of possibilities which included, which ended in compression failure of the east wall.

Q. Well that was the Beca Carter conclusion?

A. Yes.

Q. I wondered about that. Thank you very much. I understand you will be staying to participate in the panel discussion later in the morning?

A. Yes.

Q. Thank you very much.

WITNESS EXCUSED

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MR MILLS CALLS

WILLIAM THOMAS HOLMES (SWORN)

Q. Your full name is William Thomas Holmes?
A. Yes.
Q. And your residence is Oakland, California, United States of America?
A. Yes.
Q. Now I'm just going to touch on a few issues about your CV. Again your full CV has gone on to our website but just to touch some of the high points for the purposes of this hearing. You hold a BS and MS degree from Stanford University?
A. Yes.
Q. You're a registered structural engineer in California?
A. Yes.
Q. And other states in the United States?
A. Yes.
Q. You've been working in your field for around 45 years?
A. Yes.
Q. And you've had a particular interest in designing for protection from earthquake effects?
A. I have.
Q. You've been involved as well in post-earthquake reconnaissance?
A. Yes.
Q. And you've been active in developing the United States codes and standards for the design of new buildings and evaluation and retrofit of existing buildings?
A. I have.
Q. And for the purposes of the evidence you're now giving you've been retained by the Royal Commission to review the Department of Building and Housing Consultants' reports and the expert panel review, is that correct?
A. Yes.

Q. And in the course of carrying out that function you've monitored the development of the reports and the process that was being followed?
A. I did.
Q. And in addition you've reviewed the final reports?
A. Yes.
Q. And you've prepared your own report for the Royal Commission?
A. I have.
Q. I'll just go to that report just to confirm the identity of it. My reference at least, one starts to be hesitant about references, but it's ENG.RUT 0003.1.

WITNESS REFERRED TO REPORT

Q. That's your report?
A. Yes.
Q. And I understand you are now going to deal with your report by speaking to a series of power points?
A. I am.
Q. All right well I'll leave you to do that thank you.

WITNESS REFERS TO POWER POINT PRESENTATION

A. Okay I didn't review the entire report although this hearing is primarily on one of the buildings. Upon agreement with all my comments on chapter 4 I'm going to give now, brief comments. Chapter 4 was called "Context" and I think it's a very important chapter because the context of the series of earthquakes is extremely important for all to try to get a grasp on, so I thought the contents of this chapter was, was good but there were some issues I had that my understanding is that the Department of Building and Housing has agreed to edit some of these issues, not all of them perhaps. My first comment had to do with several statements in chapter 4 in article 4.3.1 and I've paraphrased them here. In one place it says, "Buildings are designed to survive ground motion with return periods of 500 years", and in another place it says, "Total collapse – while it can never be ruled out, is not expected at ground motion corresponding to the design level, which is also presumed to be the 500 year return shaking". I certainly don't agree – disagree with either of these statements, however it leaves some things unsaid I believe. The commentary to New Zealand Standard 1170.5 suggests that although buildings are primarily designed for this 500 year return period ground motion because of the factors of safety and ductility and other aspects of the design requirements there is a margin against collapse at that design level of at least 1.5 to 1.8. This is in the commentary discussing what the code expectations are. So it follows up by saying structures should have only a low probability of collapse for shaking considerably larger than a 500 year return period shaking up to intensities with return periods of 2500 years. Now this is new buildings or buildings built very recently, this is the code expectation. It, parenthetically this is not unlike the US procedures. We have a design event also. It's not defined by the 500 year return but it is a primary design event and we also think that we have some margin for rare events that might go above tha. So I want, I just want, thought that was important to point out what the code expectation is although it's in the commentary not in the primary code. The other issue I had with the context chapter is you've seen these pictures many times. They were placed in the chapter to describe in a simple way what a response spectrum was and that's been explained several times as well as try to put the context of these motions in with respect to design expectations. So there was one for the September event and there was also one for the February event. Now if you look at those two pictures they look a lot the same but it's been pointed out that the scale of the, the vertical scale is completely different. In September this number was 1.0 and in, and in this picture it's 1.8. So for a lay person or someone not very familiar with response spectra I thought it was not real clear the difference and has been said many, many times – Professor Priestley just said it a minute ago – the extraordinary size of the February event is a big part of the context. So I just digitally reduced the figure 4.4(a) of the September event to about the same scale and this is what it looks like, so, so the scale here is the same as the scale here so you can visually see how much smaller or how much bigger the February event was than the, than the September event. So these are important parts of the context. As I said I think the Department of Building and Housing has agreed with most of these comments and are unclear whether they going to have another report or how they're going to edit this report but they've at least agreed in principle. So that's the extent of my comments on the context chapter. So shall I proceed with…?

JUSTICE COOPER:

Yes certainly thank you.

A. Okay I also reviewed the expert panel chapter 5 as well as the Beca report that we've been discussing for several days and many of my comments are, have been covered repeatedly but I am going to go over them as well. We've all seen these plans several times. The significance is that the large number of walls on the ground floor compared to the tower that goes up from there and it's not surprising to an engineer that the first level of the smaller tower would probably be the critical location. A couple of other things that I noted upon looking at the drawings very quickly is that as also pointed out by Professor Priestley, it's fairly large span girders coming across from these very small exterior columns and landing on the wall which has been characterised and I'm glad other people use inches so I feel free to use inches, eight inch wall with very little amount of reinforcing and I was very surprised to see that there was no special reinforcing under any of those girders. You would expect some sort of a column to be embedded perhaps in the wall or maybe what we call a pilaster where the wall gets a little thicker underneath that, those, those beams or girders. You probably could make the wall figure technically from compressor stresses but certainly I have to say if I was the engineer doing this I probably would have put some extra strength under those locations. So I think we all agree that the building collapse due to the failure of the central tower at this floor one and two. I certainly don't disagree with that.

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This failure caused a large movement at level 2 downward and to the east and I found a picture actually which I haven't seen anybody else show directly from the top and by looking at a bunch of other photos I identified a exact same location on level 2 and scale from the drawings, the corner here which has not been move. So this exact location that I found on other photos was originally here so that's a known distance from the plan and I then found it again her. So I was able to major the horizontal movement of the second floor at three metres. I was doing this because I earlier had a theory that perhaps the tower failed and shear at level 1, 2 rather than flexure and I kinda wanted to know how far the second floor had moved because if it was flexure it might have one tendency to tip over and if it was shear it might want to move horizontally. It is very interesting but it doesn't conclusively show, you know, anything more than we've seen before but it is a pretty remarkable movement and it's not surprising that other elements started to fail under that large horizontal movement. At that point, as been told by others, some of the girders that were supported by the tower probably pulled away from the tower and collapsed in some unknown sequence and at the same time the props placed behind the perimeter columns as a retrofit to provide supplemental support for the very small columns under excessive drifts which I think were thought of by engineers in a range of maybe 5cm or so – certainly not metres. So the props were obviously ineffective because no-one ever dreamed that they would be dealing with drifts of this size. So the exterior columns also collapsed in an unknown sequence. It is somewhat interesting to speculate, however, that if there was an earthquake shaking of a size between September and February whether or not there would have been some damage in the tower that would have allowed a drift large enough to fail the exterior columns so the props actually would have been effective. It's possible. So it's just was that you went from no damage and no drift in the exterior in September to this unrealistic drift in February for which the props were ineffective.

JUSTICE COOPER:

Q. I'm not following that point. I'm sorry.

A. Pardon me.

Q. I said I'm not following that comment. Could you just make it again.

A. The props were the steel columns put behind the exterior columns as a retrofit because there was a concern that if the building were to drift laterally – drift means floor moving horizontally differentially from the next floor – that those columns were not tolerant to such drift and that they could collapse. So there is a point of disagreement between the previous Holmes report and the Beca report perhaps that the tower could never create or at least did not create drifts that would have collapsed those exterior columns until it failed. Okay, well it was a very large event that failed it and caused it to move three metres but what if it was slightly larger than September but less than February so that it would have damaged the tower but not caused it to move three metres. That also, considering the fact that the building has some torsion in it and torsion would cause the exterior to drift more than the rest of the building, just a possibility that there could have been an event which didn't occur but there could have been an event where the props actually would have been effective. I just wanted to point that out. We use props once in a while, not a lot, in the US but we use them for the same reason when we have drift intolerant columns and for some reason it's not feasible to reinforce them or whatever but we might put a prop in there. It was not necessarily a bad retrofit in my opinion.

Q. Had you finished the previous slide?

A. I did. As has been pointed out by many others, level 1 to 2 had many seismic deficiencies and as has been pointed out by many others the importance of identifying these other seismic deficiencies is in the future in trying to identify critical structural weaknesses to identify other buildings and I thought that perhaps the Beca report was too narrow in discussing this flexural compression failure and not pointing out these other potential failure modes. The end of the report has a recommendation that we learn from the critical structural weaknesses but the fact of the matter is they were never pointed out. Professor Priestley has pointed out some of these. I may have the same ones. I may have some slightly different ones. We know that because of the light centre reinforcement in these walls the tower was weak and what we might call global flexure overturning compression failure. It was also, however, weak in east-west shear as I think Commissioner Fenwick has pointed out. It had many openings, low reinforcement ratio and small trim bars. The peers in the far north wall which had been previously discussed perhaps as a torsional element. If we look at that wall I don't think it would be very good to prevent torsion. That's this wall right here. It would appear to me as if these three piers are what we call 'shear critical'. They would fail and shear before flexure. Shear is a very brittle failure particularly where there is such small amount of reinforcing so as an engineer without doing any calculations if I looked at this building I would say the critical structural weakness was east-west shear. I would think the building would fail in shear. The exact same thing would have happened. It would have failed in shear at this level. The second level would have moved as it did and the ensuing collapse would have occurred so it's a matter of detail and I'm not disagreeing with your non-linear time history analysis although many people have pointed out that the actual exact deformations of a building in a non-linear time history analysis are very sensitive to the actual motion and many other things. The balance between a flexural failure and a shear failure and a tower has to do with where the effective mass is being pushed. If it's being pushed higher you have a tendency to fail in flexure. If it's being pushed lower you'll have a tendency to fail in shear. So I did a few very simple calcs as well and I had the same questions I think that Professor Fenwick had. Are these walls sufficient

to tie the east and west walls together for tower action? Are these cross walls sufficient for just pure shear strength?
Q. It would help me if I knew what aspect of the building I was looking at?
A. I'm sorry. These are three of the four east-west oriented cross walls, tying the north wall and the south walls together. I should have pointed that out. Sorry about that. I'll have to go back too far to show you. Do you want me to show you?
Q. Yes.
A. Okay I can do that. I'll go all the way back to the first. There. There are these three walls. I didn't draw this end one here but the one that I thought might fail in shear is this end wall.
Q. That's marked, that's on the line little b.
A. The sign line little b.
Q. On the structural plan, ground level 1.
A. The next wall with the two doors, there it is, this one, near C and then the other one is between –
Q. D and E.
A. It has two doors as well and then there's one more wall but it was in a different drawing and it's convenient not to copy.
Q. Thank you.

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A. So again that is what these look like and the far north wall is the one which has these very brittle piers in my opinion. So the, if you try to figure out which of these failure modes would occur the answers you will get is that they are relatively close together. They are relatively the same strength although the flexural weakness is the biggest depending again upon on other assumptions, but there are ground motions that could occur that would probably cause a shear failure. So the fact of the matter is that the shear strength should also be considered a potential critical structural weakness, that is my point.
There are additional seismic deficiencies that have been pointed out. There is a discontinuity at the north end of the east wall. It has been

pointed out that this wall right here is not supported along this length because this wall steps over it, it has been discussed at some length.

Q. So this is the wall on line capital E?

A. Yes. Capital E I am sorry. This would cause other things to occur so a discontinuous wall in the US is immediately considered a potential critical structural weakness, not necessarily, you would have to look and see what it does.

There was no confined column elements under the floor girders I pointed that out that that causes additional compression to occur.

There was a poor connection of the girders to the towers at all levels. I will talk about that in a minute.

And we know there were displacement critical gravity columns at the perimeter that had retrofit props put under them for that reason. So these girders, as I pointed out, had no significant column element underneath them. Now it is interesting to speculate, if there had been a pilaster with a tied column under each of those girders, one here, one here, one here and one here, interestingly these two would be at either side of the discontinuous wall. It would be interesting to know whether that would have been enough compression capacity to have prevented the flexural compression failure which in my opinion would have caused a shear failure. So the same thing would have happened but it again points out the sensitivity. You cannot ignore all the critical structural weaknesses. Another thing that was somewhat surprising to me was that these girders where they were supported on the wall had a lot of reinforcing in them for the gravity loads that they span across and there were very few of these bars that were tied into either the wall or the slab on the other side. It was, I could use the word ‘shocking’, because you look at the picture and there is this very large girder supporting all of this floor load and it was not connected to the wall very well at all.

Q. So does that, the member was sort of strong in itself but not given means of support at either end. Is that what you are saying?

A. Well it was fine for gravity load. It stood there for 45 years but the minute this tower obviously started these large displacements some of these obviously tore away.

Q. Yes.

A. Now you can again speculate if there had been a very strong connection at those points or if it had been the columns that I was speaking of perhaps these girders would not have torn away from the tower and they would have formed some sort of a tepee kind of a shape and it would not have collapsed pancake-style. It is all speculation but again it, there is more stories to this building than simply the flexural compression failure in my opinion. And then lastly as I pointed out the drift sensitive exterior columns we have talked about that so all of these things would be what I would call seismic deficiencies. The real problem is identifying what seismic deficiency is actually a critical structural weakness that will cause collapse. That is the real problem and what combination of critical seismic deficiencies will cause collapse. It is a very difficult engineering evaluation problem you were discussing a little earlier. It is something we are struggling with in the United States right now and we have not really come up with a brilliant solution that is efficient.

I also in my review had a comment on the use of the capacity spectrum method in the Beca report and fortunately Professor Priestley has already gone over that so I do not have to go over it. We use this method to estimate what a displacement of a building would be in a given ground motion and as the pushover curve is in the damage range there is increased damping and there is ways to estimate what that increased damping is which lowers the spectra. I did not put a sketch of this in the presentation but there is a sketch in Professor Priestley's so I am just echoing his comments and if you made that reduction in demand due to the damping it would actually change the ratios that were deduced to come up with a percent of NBS. It does not change them significantly but it does change them.

So what are the lessons for older concrete buildings? In my opinion the first one is what conditions can we take from this building that should be considered critical structural weaknesses and how would engineers in other countries or in New Zealand use them to figure out whether a building was actually a collapse hazard. I am asking that question. I do not have an answer unfortunately because there were five or six here – all buildings do not have five or six. Does it take five or six to cause a collapse? Unlikely but it only takes one in some cases.

The other issue that I wanted to mention that has been discussed at length in New Zealand for a decade is the use of the %NBS as a rating you might say or as a way to describe the goodness or badness of a building. We do not have a better system in the United States I will preface but I think the %NBS has some issues. There were some disagreement about what this building should have been rated prior to the earthquake ranging from 35 up to 50 or perhaps even 60% NBS. However you could argue that in September the shaking was not exactly but the same order of magnitude as 100% NBS and there was almost no damage so there is something perhaps very conservative with the calculations of NBS or it has to be looked at in a little bit more detail. You could also have a brittle building at 100% NBS and it may be dangerous with only a small increase in shaking intensity, and this has to do with a much bigger question of how large an earthquake does the society want to think about, they want to deal with. Every country has made that decision one way or the other and none that I am aware of actually specifically designs for something as large as a 2500 year event or as large as this particular event was in February. They are almost all in the 500 year return range or smaller so the fact that we had failures in this event when it was so large. It is going to take a lot of thinking, I think, in New Zealand to decide whether more conservative criteria should be considered in the future at a cost. It does not come for free. If you have a more conservative criteria then more buildings will turn out to be earthquake-prone. That mean more buildings will have to be retrofit, that is more cost so it is a cost benefit trade-off. A very difficult one for society to deal with and it should not be only engineers who make these decisions. It should be a combination of policymakers and the technical people working together. I think that's my, end of my formal comments.

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CROSS-EXAMINATION: MR ELLIOTT

Q. Mr Holmes, firstly there's been evidence that there was cracking in the shear core at level 1 after the September and Boxing Day earthquakes and then of course the shear core failed on February 22. Do you think that the failure of the shear core is likely to have taken place at the same places as the damage that was in existence before that day?

A. Well I think that the cracks that occurred in the shaking in September would be an indication of the tower's higher stressed points. As has been pointed out that that earthquake was maybe more north south than east and west but there certainly was an east west component, so I would say that the further shaking would probably have attacked same places, the same weak locations.

Q. The document which you may not have seen entitled "Answers to critical questions about buildings" produced by various entities including the Royal Society of New Zealand and SESOC and IPENZ and the NZ Geotechnical Society. Just going to ask you about an expression that's used in the document and that is an expression of earthquake life for a building, but that I think the meaning is the building can withstand a certain amount of shaking before structural elements begin to fail. Would that be an acceptable way of defining…?

A. I would probably use the word degrade first and then fail.

Q. Degrade, all right.

A. Unless it's very brittle.

Q. All right, so the earthquake life for brittle buildings is generally much less than for ductile buildings?

A. Well depending upon their strength it's a strength and ductility issue you could have extremely strong brittle building that would withstand strong shaking before it got to the point of its brittlenes. So it's, it has to include both the strength and the ductility of the building.
Q. Where does the ductility of a building come from?
A. From its detailing in the way the reinforcement is placed in the concrete or if it's a steel structure it's, it's slightly different the way the things are welded together and it, it comes from ability of the elements to go through inelastic or non-linear deformations.
Q. So in the PGC building would it have been the reinforcing steel in the shear core that provided ductility, if anything?
A. It would have been if, if it was properly put in it would. There was very little ductility there because of the reinforcing patterns and extent.
Q. There's been evidence that diminished capacity was assessed by looking at visible damage, but in assessing ductility would one need to know about the state of the reinforcing steel within the shear core?
A. To know about ductility one would have to know something about that yes.
Q. And that isn't evident from a visual inspection?
A. I don't think it would be possible to see that from visual inspection.

UNIDENTIFIED COMMISSIONER:
Q. I'm sorry I didn't catch the answer, you don't…?
A. I do not think that's possible to see inside a wall, I mean you would have to see inside the wall or you would have to have looked at the drawings.

CROSS-EXAMINATION CONTINUES: MR ELLIOTT
Q. Is it possible that the reinforcing steel in the shear core even if not fractured had suffered some degradation before February 22 thereby using up some of what we might call the earthquake life of the building?
A. Well it's possible. I mean, that is the judgments that engineers make when they, when they look at the crack or look at the crack pattern. Yeah, there clearly were cracks. They were noted in the reports but it

was the judgment of the engineers that they were insignificant and that is a judgment that has to be made, certainly at a level 2 assessment.

Q. Would the cracking have given any indication to them about the state of the reinforcing steel in terms of its ductility though?

A. Well it, it could have if the, if cracks are hairline it – most engineers would think that it probably is okay. If the cracks get bigger and they were measured by the engineers or if there's a lot of spalling around the cracks indicating a lot of working either in shear or tension at the crack that would tell you that probably something else was going on inside.

Q. Degradation to the reinforcing steel would be more likely to be important where there was only light reinforcement in a wall?

A. No you can have degradation of very heavily reinforced steel it just takes more, it takes more flexure or more load to, to degrade that. We've seen evidences of shear walls in buildings here that were very heavily reinforced that buckled and the whole wall degraded in strength.

Q. Would there be more earthquake life, if I can put it that way, in a building with more reinforcing steel?

A. Well earthquake life hasn't to do with the number of cycles beyond the elastic range that a building might have I would say yes. I mean we've seen these pushover curves of, of an elastic straight line in stress and then it bends over. That's the non-linear range of reinforcing and a ductile concrete element would have far better capacity in the non-linear range than a brittle one.

Q. You made the comment about strengthening around the shear core and correct me if I'm wrong but did you say that you were surprised that there wasn't strengthening around the shear core?

A. I don't think I said that.

Q. Did you – when looking at the plans would you have expected to see more strengthening there?

A. No –

Q. What were you meaning?

A. I was surprised that the original design did not have some additional reinforcing under the pilasters or under the girder, where the girders came in. It was a very small wall with, with reinforcement spaced out and there was not even a note to say "please put one bar under the girders". It just, they just marched along at whatever the spacing was and the girder came into the wall. It just surprised me. I had not seen that kind of detailing before.

Q. This area that you're talking about just to clarify what you're talking about, Mr Hare wrote this letter back in 1997 where he proposed the possibility of damage reduction, "Consideration could be given to strengthening the transverse shear walls. This could include infilling of windows in the walls on line B and strengthening the lintels of the walls on line C and E", are you referring to a same or –

A. Same walls that I show there yes.

Q. If this type of strengthening that's contemplated there had been installed might there have been a different outcome in terms of the collapse of the building?

A. Possibly. It would depend upon how much that strengthening affected the flexural strength of the tower in general because I think everyone's agreeing that the flexural crushing is probably what caused the collapse. The strengthening of the shear by itself would have some effect because some of that concrete would come over and attach itself to the, to the east wall. So there would be some strengthening but it's hard to, to know whether it actually would have been able to withstand this level of shaking.

Q. Am I right in saying that you identified the shear strength as being a critical structural weakness. So if you'd been asked to advise on earthquake strengthening by an owner willing to spend money on addressing life safety issues would you have advised that the shear core presented life safety risks?

A. That's an extremely difficult problem. I probably would have entered into a conversation that started discussing what damage there actually, what damage we're talking about?

Q. I'm talking pre any earthquake?

A. No I know but I mean I – in order to decide whether an owner would want to do that additional strengthening I would enter into descriptions of what, what could happen and I would probably be talking about somewhere in the parlance of the New Zealand assessment in the 70 to 100% NBS shaking level. What would happen to those walls at that level of shaking because that is the evaluation, that is the design event that was decided by others for Christchurch. So I probably would not have been thinking about the level of shaking that happened in February when I was discussing this with him. So there would be cracking, an additional cracking at 80 percent NBS or 90, whatever number you want to use. If I felt that the north wall might be a brittle shear failure I would have explained that that, you know, could have caused a progressive torsion in the tower and could have led to a bad result but I have not run any calculations on that, I'm not sure. It's all a matter of communication when these issues come up of partial strengthening and how much does an owner want to do, it's a matter of communication.
Q. You made a comment towards the end about the building being only slightly damaged in September which arguably had shaking of the same order of magnitude as 100% NBS. No two earthquakes are the same are they?
A. No.
Q. Isn’t it potentially simplistic and even likely to create complacency to consider whether the building might have got through the September quake in a particular way given that even a similar earthquake, a similar magnitude might have created different impacts on the building?
A. Well currently engineers have not considered the possible different response from different earthquakes on their buildings. They more or less have used their response spectra which is a smooth average of many earthquakes. So you probably would be looking at demands, either displacement demands or forced demands from a smoother response spectra and you wouldn't be saying, well I know there's one earthquake that is similar to that response spectra that wouldn't do nothing, and there's another one that would do bad things. We are

simplifying all of those possible events in a response spectra design. That's the – we can't consider all the variabilities, it's almost impossible, so that's why we use a response spectra.

Q. Is the response spectra a tool –

A. It is a tool.

Q. – which should be used in assessing the overall picture rather than as a definitive answer?

A. It is a tool that is used in the design of new buildings and evaluation of existing buildings all over the world and when we do non-linear time history analysis we typically, because of the variability, there is some reason why we are doing that advanced analysis, so that means that you're probably more concerned than you would be if you were using a response spectra, so we normally require the use of three or maybe seven different non-linear time history analyses to get some of this variability that is between different events.

Q. And finally, I asked Professor Priestley yesterday about whether there may have been some existing training packages or courses available on the issue of diagnosing what cracks might mean so as to educate engineers, and you've been able to produce at least one and that is, is it FEMA 306 Evaluation of Earthquake Damage to Concrete and Masonry Wall buildings, prepared by the applied technology at council?

A. Yes.

Q. And are there any others that you're aware of?

A. No, that particular study was done specifically for this purpose, not for a tagging or level two, but for more detailed evaluations.

COMMISSIONER FENWICK:

Q. There's quite a lot of comment been made about this crack, one and a half metres up the north west and east walls. I just wonder if you'd like to comment. Do you think if there had been a similar crack at ground level where, you know a lot of us have thought a critical section would be, do you think that would have been actually spotted by anyone walking around the building?

A. At the ground level as opposed to level one?
Q. Sorry, at level one, at floor level.
A. You mean at the floor level – I would think that an evaluating engineer would just as well notice a crack that's one and a half metres up the wall as they would one low, although it's logical to look low but there certainly were diagonal cracks, there were pictures of diagonal cracks that were higher.
Q. Would this require then removing any flooring to spot the crack, any carpets or any lino or (overtalking 10:35:42).
A. Certainly not if it's a metre and a half high.
Q. I know, no, I'm saying, I'm really questioning whether anyone would have spotted the crack at the actual floor level itself. I just wondered, I mean it would take a –
A. Well it depends upon what the finish was. If there was a carpet –
Q. Plaster, plaster finish on the wall.
A. Well that's a different – I mean it's two different issues, you could have a carpet or some sort of furred flooring that is several centimetres high, that the crack, horizontal crack, could have been behind, that's possible. I think that those, you might call them destructive investigations are not really destructive, but you have to start removing finishes, you know, are triggered by engineers who see evidence of a building moving a lot. I mean that's why you would do such a thing. That evidence could be other cracks in the wall or it could be distress at the exterior columns or windows breaking, or doors jamming, or you know anything. It's a whole body of evidence that makes you think that the walls moved around a lot and as far as I can tell that other evidence, corroborating evidence so to speak was, there was none.
Q. I agree with you, but the crack in the wall at one and a half metres up. You can see it, you can get very close to it. If it was a crack between a wall and the floor, you can't get close to it can you? So the chances are it was there but not seen do you think?
A. It's possible, I mean who can say.

Q. One other point you might like to comment on and that is the shear performance of the walls. Now we heard Professor Priestley suggesting the reason that the apparent low shear stresses or what from my crude analysis appear to me very low shear stresses that Mr Jury was getting in his time history analysis, was possibly due to the flexibility of that diaphragm allowing the shear force to track round to the north and the south walls. Would you like to comment on that?

A. Well as I pointed out if it tracked around to the north wall, the north wall was maybe the least capable of all four walls, so it could have tracked there but it wouldn't have found much strength in that particular wall. So does that answer your question?

Q. Yes, yes it does. There's one other issue I'd quite like to bring up, so I just wonder if we can have the BUICAM233, I think then it's 0156. It's the sketches you put up for me yesterday. If we could just start off with the sketch 1, the first one, that's page 1 and then after that I'd quite like to go to – right. So we're looking at the transverse walls. There you've got the wall in the north which I've labelled wall 1, wall 2 is the central wall, wall 3 and then you've got wall 4 at the end. Now I mean when you're looking at the centre of mass and vibration of course, the torsion and shear actions will always add for the walls three and four, and be of opposite sine or subtract from the wall, wall 1. Can we go now to sketch number 5 please? Yes there it is there. Now when you look at the structure this shear core is in the middle of the structure. It's surrounded by a floor with beams. If you were, I'm sorry I haven't got this on wall, the north wall for you but it's on the central wall. That structure is surrounded by floors which goes straight through the core so it's tying the two sets together. If you were to start to get flexural cracks in the coupling beams above the doors, the flexural cracks would actually open out a bit due to elongation and you would then have this floor slab actually tending to clamp the whole thing together. Do you think that might have had an influence on the shear performance of the different walls.

A. The vertical shear or the coupling shear?

Q. The coupling shear whether it's vertical or horizontal.
A. It could have. It's a secondary effect that is not normally considered but it could have provided additional tie there.
Q. It's not easy to put a figure on is it but it's something which seems to be arising in a number of structures –
A. I think it was mentioned before that the behaviour of buildings when they get to a level of high damage it brings in a bunch of secondary effects that engineers don't usually deal with which makes the prediction of what starts to happen to these structures extremely difficult and I think what you're describing is that it would normally be considered as a secondary effect.

COMMISSIONER CARTER:
Q. Mr Holmes I'm sure you're aware that the terms of reference of this Royal Commission expects us to determine whether the standards that we work to this country are appropriate and I think your appointment to help us coming from a country where we often compare our techniques etcetera with dealing with earthquake is one that's rather similar to New Zealand and its exposure to risk. In your evaluation of the way the importance of earthquake is treated in California versus the way it's been dealt with in New Zealand, how do you feel about the standards that New Zealand in general is applying to its approach to this matter in comparison to the approach that you're used to in North America?
A. I think they're very similar and it's not accidental. There's a lot of interchange between the earthquake engineers in the two countries. As far as policy goes we have done perhaps some things to a larger extent than you. I think we have dealt in California with the unreinforced masonry problem a little bit more completely than you have. On the other hand you have the earthquake prone programme where at least nationally you're trying to identify buildings that might be earthquake prone. We have nothing like that. We have been attempting, we being the engineers, have been attempting to get the attention of policy makers particularly for older concrete buildings because we know that

not all older concrete buildings will collapse but one collapse could cause a lot of loss of life and we have been trying to explain that to policymakers but we have not been successful in any significant way. So there's some things we have gone farther than you and some things you have gone farther than us but I think the two sets of policies and engineering design criteria are very similar.

JUSTICE COOPER:

Q. There's been quite a bit of discussion of this concept of critical structural weaknesses and you've used the term 'seismic deficiencies' and I infer that with this particular building there were a number of weaknesses, not all of which contributed equally to the collapse of the building perhaps but it had a number of design aspects to its design which would have been better avoided. Is that right?

A. That's true.

Q. And if we're to learn lessons as a society from this event I understand what you say about the severity of the February earthquake but is it a prudent course or a realistic course to follow that one might isolate these seismic weaknesses in buildings and take steps to ensure that they are not repeated in ongoing building design?

A. Absolutely. Every earthquake has a learning from earthquakes phase. Engineers try to see what improvements they can make in their code for new buildings. That has been the traditional learning from earthquakes but I think many engineers are now becoming more interested in what we can learn and apply to our older buildings because the fact of the matter is most engineers think that our new buildings are actually pretty good. They could be improved but they are pretty good but the real hazard, seismic hazard and risk to life in most countries, is from older buildings.

Q. Yes but I suppose that's a relative term isn't it because we have in the Christchurch earthquake this building is constructed in the mid 1960s, some time ago, but is that what you mean by an older building?

A. It is relative. We have concrete buildings in California starting in the 1930s, large buildings, running all the way up to current. We do use some dates even though you pointed out that there might be exceptional buildings that are post, we call them 'bench mark years'. There may be some buildings that slip through that process but we have so many buildings that we have to start separating them into bins somewhere. We have used four or five different Building Codes in the United States over the years so there's a fairly complicated chart in one of our evaluation documents that indicates what year for every code that's been used is considered acceptable. So if a building was designed to the 1976 UBC or the National Building Code some other year if it's beyond that theoretically if the building was designed in accordance with the code it would be acceptable. So that's the first separation. Then we have two bins – one that's considered okay – and one that's considered suspect and the suspect bin is the problem. How do we sort that out and find the maybe ten percent buildings that really are collapse hazards without having to fix everyone or spend $50,000 of engineering to evaluate everyone. That's where we are right now and we don't have a method of finding those buildings quickly and efficiently.

Q. And is part of the problem when you say you don't have the method is part of the problem the regulatory environment?

A. Well that's the policy problem, but there's an engineering problems and that is that we don't really have a method of, whether you want to call it critical structural weaknesses or whatever it is by looking at a building in a reasonable amount of time perhaps for an engineering cost of $5–10,000 of saying that building is a collapse hazard and that one isn't.

Q. Well how far can you go by looking at plans?

Well I'm assuming you're looking at plans and of course sometimes plans are not available but plans are typically available in California anyway so we're assuming we're looking at plans. We just not looking at the building. I don't think you can even get a start at this problem without looking at the drawings and maybe doing some calculations, but as Professor Priestley said there is a distinct possibility that you're gonna get different answers from different engineers and you would very much like a process or a procedure that engineers could follow to assure better consistency of the judgements as to which buildings need to be fixed because there are huge economic consequences. In New Zealand right now because of the freshness of these tragic events I think owners will be much more tolerant to spending money to fix their buildings but I can tell you that will last not very long. It's been shown earthquake after earthquake is a very short memory so if you have not had a recent earthquake and the City Council tells building owners they have to spend thirty percent of the value of their building for a retrofit there will be arguments about, "No my building isn't that bad" and so on so you really need a procedure which can be applied consistently to identify the killer buildings.

COMMISSIONER CARTER:

Q. How specialised do you think the skills need to be to have that consistent approach?

A. Well if you have a procedure it requires less skills than if you don't have a procedure. If you don't have a procedure then it really takes experts to, you know, to quickly identify these buildings but if you want the procedure to be done by every structural engineer practising and have some reasonable confidence that they are all getting the same answers, you do need a procedure.

JUSTICE COOPER:

Q. Can I just ask you something else from your understanding because you've studied the consequences of earthquakes with structures all around the world. Is it always the case that a significant earthquake is followed by aftershocks?

A. Yes.

Q. So that apart from the idea of having an understanding of the state of the building stock and what might be done to improve its seismic performance, there would be a real value in having that information at hand in the post earthquake situation where you were wondering what buildings might be safe to occupy and what might not?

A. In my opinion that would only be practical and feasible in a second level of evaluation. The first emergency stickering –

Q. Yes, that's my proposition.

A. – No way you could do that. The second one there's a lot of policy issues. I mean if you're gonna have buildings identified previous to the earthquake that may be earthquake prone or that you're concerned about and you're gonna say because of the aftershocks regardless of what the stickering said I'm gonna close those buildings until I can evaluate them. It has huge economic consequences to the owners of those buildings and the people who work would lose their jobs or would not be able to work for a month or whatever it is. It takes a huge engineering effort to make those detailed evaluations at a time when most structural engineers are very busy dealing with damaged buildings and so it's a practical consideration. The stickering procedures of ATC20 which is what most people have similar policies, you know, was done with all of this in mind and the most efficient philosophy was one that has been mentioned here many times and that is if a building is as safe as it was before the earthquake and the community is letting people occupy it before the earthquake then the aftershock shaking generally almost always is less so it is a reasonable and efficient policy to quickly make these occupancy decisions using that criteria and that's a whole premise of the evaluation policies. If you have an event that is twice as big as the first event it's really not within the eyesight of the developers of that procedure that that is a possibility.

Q. But there are various issues that need to be considered but I suppose the proposition I was putting to you is that that decision about what was the state of the building after the earthquake must be a better decision if it is better informed by some knowledge that has been gained in less difficult circumstances about the state of that particular building and that that would be one of the benefits of having a survey of buildings and an understanding of which had identified critical structural weaknesses?

A. Yes there is actually a programme in San Francisco that I would have to say is a positive version of what you just described because owners of large buildings are very concerned that they want to get occupancy as soon as possible and the City of San Francisco who would control stickering, so to speak, allows an owner of a building to retain an engineer to evaluate the building in great detail and understand all of its characteristics so that they can quickly go look at those areas which they thought might be weak or have damage and could inform the City of San Francisco quickly that it has not been damaged or it has. So it's a formal programme where you go sign up with the City of San Francisco and say, "I'm gonna do this and here's the engineer that I hired and he's gonna tell you very quickly whether I need a red tag or something." I say it's positive rather than negative because these people are much more concerned to get a green tag from detailed evaluation than worrying about the risk. I mean society or the community has to worry about the ones that are missed on the red tag side but the community would have to spend a lot of resources in order to gain that information from all their buildings.

Q. Is this a by-law or is it an ordinance or...?

A. It's an ordinance.

Q. Yes. Could you give us the reference for that?

A. It's called BORP – and I can't remember what that stands for – does someone know. I can't remember.

Q. Building Occupancy Resumption Programme perhaps?

A. That sounds right. You can look it up.

Q. Anyway we will be able to find it I'm sure. Thank you Mr Holmes.

JUSTICE COOPER ADDRESSES MR MILLS RE EXPERT PANEL DISCUSSION

COMMISSION ADJOURNS: 10.59 AM

COMMISSION RESUMES: 11.05 AM

JUSTICE COOPER:

Gentlemen can I just in the interests of efficiency note that you have all sworn an oath or made an affirmation to tell the truth, the whole truth and nothing but the truth. Can I just ask you to all acknowledge that you will still be bound by that oath or affirmation whichever it was.

WILLIAM THOMAS HOLMES: Yes
ROBERT DAVID JURY: Yes
RICHARD SHARPE: Yes
MICHAEL JOHN NIGEL PRIESTLEY: Yes

JUSTICE COOPER:
Now at earlier stages in our hearings we have benefited from group discussions such as you are about to participate in. Slightly less structured than in the questioning process that applies when you are giving evidence as individuals, but if I may I will just start the ball rolling and we will see where we get to, and we might give an opportunity to each of you to say anything at the end which you particularly think should be said to give perspective and help our understanding. I would just like to pursue if I may the kind of discussion that I was having with you Mr Holmes about the desirability of first of all developing the engineering understanding of seismic or critical structural weakness as a consequence of the earthquakes in February and I take it you all see merit in the idea that the Royal Commission should, as parts of its obligation to study a representative sample of buildings, endeavour to identify and explain the structural failures of buildings in the earthquake and with a view to creating to the extent we can a record of what we found. That's a good idea isn't it? You all agree with that?

(no audible answers 11:08:45), but heads nodding.

JUSTICE COOPER:
And then it seems to me as a lay person that a number of things might follow from that. One is simply the benefit of future practice avoiding to the extent it

can the repetition of such features of buildings. Is that a realistic goal do you think?

MR HOLMES:

A. On new buildings?

Q. Yes.

A. I think it's very realistic. I think my own opinion is we're probably there except for design mis-judgements or errors.

Q. Yes.

A. You can't prevent that completely although plan checking and peer reviews minimise it but I think both countries' codes will prevent catastrophic collapse by and large.

Q. When you say we're there, are you saying that the existing code contains sufficient protections to avoid a repetition of some of the failures that have been observed in Christchurch?

A. In terms of life safety failures for new buildings –

Q. Yes.

A. – repairability issue is a different issue.

Q. Yes.

A. And of course we have again, and I hesitate to bring it up again but the size of the event is a big factor here.

JUSTICE COOPER:

I have not, and I am a lay person, so my understanding has its limits necessarily, but I have not heard anyone drawing distinctions between different kinds of structural or seismic weakness in the sense of saying, "Oh well that would only be, that is something that would only be a problem in an earthquake of this size." What I have heard you saying is that there are a number of elements for example of the design of the PGC building that would have been better avoided without perhaps saying that if they had been avoided the building would not have collapsed but it would have made it more robust and better able to resist what it was presented with. Is that right?

PROFESSOR PRIESTLEY:

Yes I think that that's quite clear, but again it must be remembered that the PGC was designed in the early 1960’s in a very different set of building code requirements then from now, so it is clear that if that building had been designed nowadays, it would not have looked like that, it would not have had the same detailing, it probably wouldn't have had the same structural configuration either, so it would be a very different structure if designed to current codes.

COMMISSIONER CARTER:

Yes, but I think it appears that this design was completely dependent on the tower behaving in the way that it would resist the earthquake on its own. I understand that in the United States there are some requirements that secondary frames have to carry a certain proportion of load, that it wouldn’t – is that something that would be more appropriate when one’s considering what sort of redundancy elements you might deliberately build into a building?

MR HOLMES:

It's an interesting question. We have requirements for a building of a certain height to have a secondary moment frame there. Those buildings are often core buildings of some sort, not necessarily concrete cores but they might be steel cores, brace frames, some other system and all buildings over 220 feet in height need a moment frame, a ductile moment frame that's good for 25 percent of the total load. That's what you're referring to now. Interestingly enough, a very popular and economical super high-rise building has been developed recently and that frame has been eliminated due to – I mean because developers love to not have that frame because it's difficult to form, it causes the windows in these condos to be less large and in most codes there's a thing called an alternate means of compliance, so that if an engineer goes to the jurisdiction and says, “I am not going to follow the code every step but I'm going to prove to you that it's still okay,” we have that and there's been several of those buildings built under that process. The city of San Francisco, if it's built there, proposedly built there, it has a fairly extensive peer review process to assure them, the city, that the building has equal performance to a code complying building, so it's just interesting that that very back up redundancy that you're talking about is being eliminated in these super tall buildings.

PROFESSOR PRIESTLEY:
I'm also – my personal viewpoint is that intellectually I find arbitrary rules which say 25 percent need to be carried by frames are not very convincing and why is it not 15%? Why is it not 35%? It is not based on calculations or anything, it is a belts and braces type approach. It can be difficult to implement in the design in that if you are doing something where in our design codes at the moment we use a ductility factor to reduce the forces, the elastic forces, or in the United States where it is a force reduction factor that is being used, they really are not the same for the frame of the building so that has to be taken into account but generally is not. So there are some issues with that type of approach and I do not think it is a substitute for good design. If the design is good it is probably not necessary. If the design is bad it is probably not going to save it.

COMMISSIONER FENWICK:
Can we just broaden this out a little bit? Something that has concerned me about earthquake action standards. Where you have got one element which is resisting a very high proportion of your seismic actions, if that element starts to fail or starts to lose strength it is gone. If you have a multitude or you have alternative load power then when that element starts to lose strength it can redistribute loads in a different way and give you redundancy. Now in our current loading standard we do not recognise that action. I just wonder though if you could go back to the 1970s we did to a certain extent that if there is redundancy on the wall which you could reduce the loading on the wall (inaudible 11:16:48) and so on, I just wonder if you could comment or do you think this is a wise provision where you do rely on one element you should perhaps be somehow ensuring this has got a high ductility or a higher strength and perhaps if you have a multitude of elements where you can get

redistribution between the elements, could you give me a reaction please to your thoughts on that particular problem?

PROFESSOR PRIESTLEY:

A. Well again I always had some problems with the concepts of redundancy because sometimes that leads you into a false sense of security. There is the old example that is sometimes quoted about aeroplanes when they change from single engines to triple engines which has an increased redundancy apparently but they used to crash. This is in the 1915 sort of period when they first came in. The crashes went up and the reason for that was that if you lost one of the engines the aircraft failed, whereas you had three times the probability of an engine failing. It is not a very good parallel but there is something or other associated with that and there are structural aspects which relate more to bridges than to buildings where that is potentially the case. I think that where you have different structural systems that are providing the response you have to look very carefully about how that load is distributed once one of those systems starts to deteriorate and that may make it more difficult for the other system to take up the load so as you are aware if you have got a frame and a wall system acting together they act in to some extent against each other to some extent in different parts of the building, at the top and at the bottom they carry different proportions. So you, I think you need to do an analysis which would take into account the softening effect of the wall and how that causes a distribution to the frame. It might make design more difficult. The idea of requiring a higher capacity is certainly something or other with this but I would prefer to see something or other where there is more emphasis on getting it right with the wall systems if that is what we are talking about and I think we are. I don’t think we are talking about if you got a frame building that there should be a wall in there as well to take a secondary system. We are talking about walls and walls in particular. I think you are talking here about a core made of walls, a single core which is the seismic resistant system and the danger of –

Q. Just let me sort this one out.

A. What is that?

Q. I am not trying to refer, saying we always ought to have moment-resisting frame with walls.

A. Yes.

Q. In terms of I was saying is that would it be desirable to have more than reliance on one wall? Now if you got a wall each end of the building then you are relying on those two walls. If one goes the building has gone. If you have two or more walls at each end of the building then it is not so susceptible and if one starts to fail prematurely due to bad workmanship or poor design then you have got a chance at least of picking up on the other wall. So I am not talking about the need always to have a dual frame wall system by any means. It could be any (inaudible 11:20:35)

A. Well would you call a shear core one wall or a multiple wall?

Q. If you are referring to this particular building I would have said that is one because once you lose one wall your redistribution effects due to torsion and due to shear obviously could not carry the resultant actions. You start looking at the shear actions and the torsional actions. They are greatly diminished by the failure of one wall so virtually the failure of one wall in that element converting it into separate walls has got a very much lower strength. I would say that is one element.

A. Would that be the case if you had your example of a floor with two walls at each end and you lose one of those walls is that more serious than, is that less serious than losing one aspect –

Q. No, no, if you lose one wall at the end earthquake action is 50% of the load will go to one end and 50% up. You lose that 50%, there is no way this wall can pick up and compensate so I would say that would be a statically determined system.

A. Well it is not statically determined –

Q. That would be no better –

A. – because of rotational inertia and things like that. It is not quite as simple as that.

Q. I agree but I mean that is still you have got no torsional resistance if you take one wall out and you are relying on just that one the other end.

A. But again you have got to have something in the opposite direction as a seismic resistant system. Either you have got walls in that opposite direction or you have got frames, and these are providing some of that resistance to it and of course the failure of one of those walls increases the torsional problems to it as well. Anyway I think –

MR HOLMES:

If I could indicate that we in fact have a redundancy factor in our code. It has been changed several times because it is so problematic. The concept was always exactly what you are talking about and it is pretty much a penalty requirement but we also have a continual criticisms of our code that it is too complex because there is just one series of requirements after another and somehow it is assumed that the average engineer you know is going to make more mistakes the more complex the code is. There were always pressures of time and resources and so on. So this particular redundancy factor was highly criticised because it was hard to understand, hard to implement, hard for the authorities to figure out whether it was applied so it continually gets watered down but that is our experience with trying to do such a requirement.

MR JURY:

I think that in context of PGC if it had been considered, if you consider what you would do now with a building of similar configuration with the current code. It would have well confined walls. It would have not be shear critical in those critical areas. It would be well supported. It would have good attachment perhaps not to the full extent we think may now be necessary from the diaphragms to the walls but certainly better than was in that case. It would have secondary columns. It would be well confined. Joints in that secondary frames that would be well confined. I think it would be far more resilient than obviously that structure was in February. I would not see any real need to increase its load compared with other structures because that is simply what a redundancy factor typically does is it increases the strength requirement on a structure. I would not see any need for that for this particular structure if those essences of the modern code technology was included in its design.

JUSTICE COOPER:

Do I take it from what you are saying that it would be possible to identify those list of things that the building lacked comparative, compared with a building built under the modern code simply by looking at the plan?

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ALL PANEL ANSWER IN THE AFFIRMATIVE

MR HOLMES:

A. To a certain extent. I mean basic strength is important then you would have to do some level of calculation to do that.

JUSTICE COOPER:

Q. So in terms of learning from these critical structural weaknesses or seismic deficiencies whatever they are called, it would be possible in the fullness of time to understand how good the existing building stock was, thinking of buildings built prior to the current code.

PROFESSOR PRIESTLEY:

A. My view is that we're not going to learn anything from PGC. There were deficiencies all over the show which would not occur in a modern building but there are a number of buildings, a number of wall buildings that performed very badly in Christchurch which were designed quite recently and I think those are the ones which I do hope the Royal Commission will look at in some depth and those are the areas where I think we're more likely to find problems which may need addressing in the Code or we need to know why they failed – Was it Code deficiencies? Were there design deficiencies? I think as well it's important that the Commission looks at a number of buildings that performed well in the earthquake to understand that we would have predicted that they performed well. What is it? What is the difference between, for example, 76 Park Terrace if that's the one that I'm thinking of, and other buildings that have performed better. Why did wall buildings appear to have performed worse than framed buildings? That's just a general gut feeling that I felt. It may not be correct.

JUSTICE COOPER:

Q. Well I accept that and that's another part of this discussion and perhaps we can come onto that but before we do, it seems to me there are possibly lessons that can be learnt from the PGC in the field of post-earthquake inspections and that's really what I'm thinking of that in the situation where there has been a significant earthquake and you're examining the effect of that, it seems to me as a matter of commonsense you must be better off in the post-earthquake situation if you know more about the typical deficiencies of the buildings that may have survived the earthquake more or less well. That's commonsense isn't it?

A. Yes.

Q. You are all agreed on that. Because in the Beca report there's a statement that territorial authorities should be encouraged to include screening for critical structural weaknesses in their earthquake-prone building policies and any reference to earthquake-prone building policies come with a whole lot of baggage that we're going to have to sort out but I was just wondering whether or not you could have a shorter sentence and say territorial authorities should be empowered or perhaps even required to understand the critical structural weaknesses of the existing building stock, whether it needs to be under the label of "Earthquake-Prone Building Policy" but that's the point that you're making isn't?

PROFESSOR PRIESTLEY:

A. Yes I think it is probably quite a difficult question because you might know about it but what do you do about it and I think that's one of the issues that's come up in Wellington that we have been evaluating their, what we consider to be earthquake-prone buildings based on the current legislation and we have been doing that using the IEP process which is the visual inspection, not necessarily with plans although with difficult cases and with strengthening buildings certainly with plans but generally from a visual inspection and the reason why that is being done is simply from a practicality point of view that to get through 4000 buildings in a reasonable amount of time you do not have the ability to go and look at the plans of every building so you've got to apply a certain amount of judgement.

Q. But over a course of years you would?

A. I think it's been estimated in Christchurch that to do a detailed assessment of every building which is currently one possibility will take at least five years and probably longer. Buildings will be five years older of course in that time as well.

Q. You include the residential buildings or are you just talking about commercial buildings?

A. Commercial buildings.

Q. Well we deal here with earthquakes with return periods of thousands of years, don't we? I mean in terms of what's at stake a five year exercise some might think ought not to be so daunting a prospect that it was not embarked upon. But that's a policy decision that needs to be considered I think.

A. I would certainly not say it should not be embarked upon. I fully support the efforts that the Wellington City are doing and I implore other territorial authorities to do the same thing. To know what the buildings are is very important but I still make the point that once you have decided what they are, what do you do about it.

Q. That's another issue but dealing with the logical sequence is a starting point it seems to me.

A. Rob, when you talked about a detailed assessment, did you mean structural calculations there?

ROBERT JURY:

A. Yes.

PROFESSOR PRIESTLEY:

Q. Or did you mean the sort of thing that Bill was talking about more of the sort of $5000 thing which is a review of the plans and looks at it and says, "Well there's no transverse reinforcement around the columns, that's a structural weakness, there's problems in the shear core and the connections" and identifies these so that they are at least in a, well kept by the City Council in some sort of register of problem areas and problem buildings.

Q. Is there merit in doing that?

ROBERT JURY:

A. Definitely and I consider that to be an IEP with plans. The IEP doesn't preclude having the plans but so an informed decision with plans but the IEP does set out a framework for doing that and identifying critical structural weaknesses. In the IEP the critical structural weaknesses and primarily those that might be apparent from the exterior without plans because it was set up as a screening process but it can equally be done with plans.

JUSTICE COOPER:

Q. My impression is that at the moment that there might be a number of things that were simply not able to be seen by visual inspection, especially if we're in a city where there hasn't been an earthquake for many years and we have to consider the rest of the country as well here but which would be identified as critical structural weaknesses by a good understanding of the plan. Is that not right?

ROBERT JURY:

A. That is true but there are not always plans available and we're finding that in Christchurch.

JUSTICE COOPER:

Q. Well there ought to be of reasonably modern buildings because of the Council's record keeping requirements under the local government legislation but that's perhaps another issue. But as a policy postured for the future, you know, a re-emphasis of the need to keep records, plans and so on might be helpful.

MR HOLMES:

Q. Could I ask who's paying for these IEPs – the owners?

ROBERT JURY:

A. In Wellington the city is doing the initial IEP. If the owners do not like the resulted outcome then they pay for any additional cost to dispute.

JUSTICE COOPER:

Q. Tell us for the benefit of those who are following from the gallery the IEP?

ROBERT JURY:

A. The IEP is an initial evaluation process. It was developed by the Earthquake Society as part of its overall guidelines in terms of looking at existing building stock and it's primarily a course, a screening process which tries to identify critical structural weaknesses and, based on those, penalise a score which is presented in terms of safety building standard based on the original code of which the building might have been designed. So in a very stylised process type way it determines a percent MBS for the building and then downgrades it because of certain critical structural weaknesses.

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JUSTICE COOPER:

That is the sort of path that if it ends up with a percentage of NBS, but that's not a necessary feature of such a scheme is it?

MR JURY:

No, certainly not, and it's just one way of presenting the result, yes.

JUSTICE COOPER:

Yes. And you see there might be all sorts of uses, or ways in which this information could be useful, particularly in the post earthquake situation because it may mean that some buildings would have to immediately and automatically go through some more robust investigative process before they were able to be reoccupied and there would be a logical framework on which that was occurring, and its economic or its justification would have to relate to life safety of course, and the cost implications would have to be considered, but those are all things that can be dealt with and policy positions adopted.

MR HOLMES:

I think that would encourage owners to understand their buildings better. As Rob says if they get a bad answer, if someone else evaluates your building and you get a bad answer then you're going to hire your own engineer to try to get a better answer, and if in fact a selected list, and identified list of buildings would require a more detailed post-earthquake evaluation, that would concern owners from a time standpoint of getting their buildings open, so it might encourage strengthening in the long run.

JUSTICE COOPER:

Yes.

MR HOLMES:

We have been trying to get seismic safety into the market place for a long time, I think all over the world, and by that I mean for society and owners, and well everybody, the community to value seismic protection. Right now certainly in the US if you're building is better seismically than one across the street, it's not more valuable. You can't get more rents. It doesn’t really, it's not in the marketplace except maybe immediately after an earthquake people will ask, “Well is the building seismically adequate,” and one way that has been thought of by both countries is some sort of a rating system so that a building has a rating, it's been considered to use stars, one star, three stars, like a dining establishment or letters or whatever it is, and that if a building had that higher rating it would be more valuable so you're getting it in the marketplace so that if you can sell your building and it's more money, then there's reason to strengthen it. It's the whole process. We have been unable to do that so far, but we're trying to do that, so if there are more ramifications of having a bad building, whatever they may be, post-earthquake or no value, whatever, I think it would encourage a gradual improvement in the inventory.

MR JURY:

I certainly like the idea of a star system for buildings, but I would make the comment, for all its faults the percent NBS has become reasonably well known out there in the market, especially in Wellington and now in Christchurch. It is understood, well its intent is understood by tenants, building owners and engineers, whether they all apply consistently is, is always the issue, but it is a concept that is reasonably understood. If somebody receives a score for a building either as a tenant or as an owner of you know 10, 15 percent, they know they've got an issue. If they're up in the sixties to eighties, they know they you know may not have an issue, and if they've got 100 percent they’re reasonably satisfied that it's probably meeting close to new building standard.

JUSTICE COOPER:

Is a building that's at 50 percent of NBS half as good in an earthquake as a building that's 100 percent?

MR JURY:

And it falls over twice as often.

PROFESSOR PRIESTLEY

No, it's difficult to say. The intent is meant to be that it should be able to survive about 50 percent of the design earthquake without too much damage, but I dislike the implied precision which is not intended to be provided but there's going to be someone saying it's not 50 percent, it's 60 percent and we don't have the ability to judge to that level and I think the broad concept of the star system where, you know, three stars might represent something between 80 percent and 120 percent, and two stars maybe something between 50 and 80. There's something or other which is less likely to be argued over which would be probably a good thing.

DR SHARPE:

The star systems that have been discussed out there at the moment go further than just the raw structural strength of the building. They go, they contemplate such things as the extent of damage, the repairability, and the escapability from that. I remind the Commissioners that they only have one, they have four representatives of only one part of the design team in front of them, yeah, and I've been dying to say that after the earthquakes when all the old men came back to Civil Defence headquarters, and said, "Well guys, what did you learn today," and everybody nodded their head and said, "Back to the old principles guys. Regular buildings and stick to the principles that we've always been known and we've been taught," and that involves the rest of society to come along with that, so in particular there's always a tension and there would have been in 1963 between the engineer and the owner and the designer as to what the form of the building should look like, and when we're talking about redundancies and so on, we – I have my colleague beside me who is the chief structural engineer for Sky Tower in Auckland and perhaps we could put him on the spot and ask him about the provision of redundancies in the Sky Tower which as you know is the very slender one element going up there. But before that, just to say that as Mr Holmes said before, the engineers themselves have been fighting for a long time to get the society to come along and understand the seismic risk, and unfortunately it would have been good if this – we'd stopped at the September earthquake, because that was just the boost we needed as the New Zealand Society of Earthquake Engineering lobbied for some of the provisions the 2004 change to the Building Act which brought in the – eventually brought in the concept of earthquake-prone and things were coming along pretty nicely really in terms of the marketplace picking up on the impact and the earthquakes in Indonesia and in other places have all helped that, but we finally now, we've got what we've been hoping for, for a long time, which is the interaction with society on setting the risks, and there are lots of other risks in life and it's very good to bring back the simile of what we do about old cars on the road, and you can go right through the concept of a 1963 Ford Cortina is a legal car on the road at the moment if it's warranted, it's past its basic inspections. We know it may well not have decent crumple zones, it won't have airbags, it won't have ABS brakes but society tolerates having a small number of vehicles on the road which are of higher risk to particularly the driver, and I guess the difference with a building and building stock is that the public up to now didn't know what they were getting into when they went to work each morning, whereas if you now go and buy a 1963 Ford Cortina, you've got some idea of where it sits in it, so I think that that's something which pushes it along towards having some obvious grading of buildings that the public can understand and make their decisions, and the marketplace is certainly doing that at the moment.

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MR HOLMES:

I would like to bring up one other issue and it is, I do not know the answer, I could question whether or not it is better to try to explain the risk by saying what smaller earthquake the building is good for rather than defining some earthquake and say what is going to happen to the building. Okay, so a bad building in a whatever if you use a 500 year earthquake that may be too big because too many of them would collapse but if you name some earthquake and you say, instead of saying this building is okay for a moderate earthquake but you do not say anything about what is going to happen in a bigger earthquake. You standardise the earthquake and have different levels of performance then that is how you characterise the risk so I am not convinced that owners or tenants understand when you are saying the building is going to be okay for a moderate earthquake what that implies for a strong earthquake because it could imply bad things. So it is the same characterisation but how are you communicating it to the public?

COMMISSIONER CARTER:

Just looking at a couple of observations on the PGC building. One being the poor support of the girders that you described earlier Mr Holmes and another being the jacketed columns at the ground first level, and from my reading of the threats to life that was exemplified in Napier where there were complete floors fell onto the people within the buildings, a huge number of deaths, and here we have had buildings which actually pancaked down whereas we have got other buildings which are so damaged that they need to be demolished but there is still the people within them were kept intact and survived and so forth so the building was lost but the lives were not. Now we have been subject to some criticism in engineering that some of the poor detailing has been observed in New Zealand even in recent years that the provisions for those sort of well tied together elements and we know that in any building if one floor falls onto the floor beneath it will be so overloaded instantly that the suggestion that it will fall as well and of course that results in what we have seen happen even the New York disaster where sequence of floors falling on each other actually brought the whole structure down so we, I think there are elements of importance that are more than the performance of the structure as a whole. The performance of elements within a structure and some of those features do get down to detailing. Perhaps you can comment as members of the panel on that that might be thought of as a second order issue is actually a primary issue with respect to the loss of life.

PROFESSOR PRIESTLEY:

A. I think we have all said that detailing is a key issue and really was a primary cause of failure in this structure and that detailing, you mentioned, you started off with the steel columns in the ground to first floor. They certainly had a huge redundancy of displacement capacity. Putting a steel jacket round the outside of a column though this is not what happened here because they were cast inside the, I believe, inside the concrete column but if you have a deficient column and you put a steel jacket round it particularly if it is a circular steel jacket, then you increase its displacement capacity by a very, very large amount. So you improve that and that is something that can be done. And you could do that. That could have been done at a cost of course in the PGC but it would not have solved the problem in the joint region between the beams and the columns itself which also is a deficiency and very much harder to address and we know from the failure and the photographs of the failure that there was failure of the joints as well. Whether that was secondary or primary to the failure of the columns we do not really know. The issues of connection of the beams to the wall which Mr Holmes has addressed, now they could probably have been addressed by adding pilasters to the core so that I think and perhaps Bill could comment further on that but I think that that would be a viable retrofit to the system. The shear strength of the cross walls in the core. Certainly a comparatively easy retrofit to improve the strength of those very substantially. Again at a cost but probably the hardest one to do would be really to deal with the low reinforcement ratio and flexure in the walls and the propensity of them to fracture. I do not know. Others could mention on that but I am not sure if that is really what the question you were asking Commissioner is or –

Q. My question more relates to suppose you have an existing building in which we identify certain weaknesses in it. Our thoughts should go to not merely how do we make the building stronger but how do we make sure that parts of the building will not collapse on the occupants even if we cannot save the building? And just an added point of focus if you like on the purpose of the retrofit.

MR JURY:

I think one of the things that the engineers did do in 1997 is they did have mind to that and that is why they put the props in behind the columns. I think in all those sorts of measures though they are limited to what actually happens to the structure, how effective they might be. I think perhaps if the floors had been better attached to the walls and that would include perhaps the support of the beams, et cetera, then they might well have been that life creating space created between the floors. Zero gap of course between the floors is what really, what is causes fatalities, and so keeping the floors apart is the primary aim. I am not sure in retrospect, even retrospect, what else could have been done in 1997 for example when they were considering such matters to actually improve that, based on the knowledge that was back then but certainly now the tying those additional props in perhaps a bit more than was probably done in 1997 would certainly be advisable. We saw in the debris lots of props that were undamaged that had still had their bolts in the ends of the fixings so something happened that –

MR HOLMES:
It was probably the joint failure.

MR JURY:
So that obviously was not enough in this particular case with the displacements that did occur and the wall to keep those floors apart but other measures certainly could be considered for such buildings.

MR HOLMES:
I think some of your comments may also be referencing what we engineers call progressive collapse which became a catchword. I mean it has been a catchword for some time due to some failures I think in Britain but progressive collapse basically refers to a structure that if one element fails some place it will cascade and cause the whole structure to fail and it is a big concern from a terrorist standpoint because if a terrorist can find such an element in a building by putting one small explosive or something they can bring the whole building down and we have seen such collapses and in the US there was a lot, there had been a lot of studies of progressive collapse and how to prevent it. At one point it was suggested that it be put, those provisions be put in all commercial buildings but it has turned out it is a huge cost because if you, if you come up with provisions to prevent that, it has a lot of implications and for example you have to, you have to be able to take out every, any column in a

structure and have it remain stable so that means that you have to have the beams where the slabs span to two adjoining columns and, and from an engineering standpoint it could happen in fact but you have actually calculate that so the beams have to be much bigger. I mean it's, it's a big cost. There's a whole bunch of tests that you have to go through to, to prevent progressive collapse and I think good engineering can prevent it without having to go through all those tests. The tests are what cause engineers to put in a lot, additional structures, so we, we are not doing that. Some military and government buildings have progressive collapse designs, anti-progressive collapse designs but it is not common in the private sector.

COMMISSIONER FENWICK TO PROFESSOR PRIESTLEY:

Q. Just lightly reinforced walls is obviously a feature which probably needs some investigation. What about the more heavily reinforced walls but single central reinforcement. I'm thinking more here in terms of tilt-up structure. Do they have the same problem perhaps as the lightly reinforced walls?

A. Generally tilt-up structures are less high, one or two stories and I'm not sure about this but I understand the performance due to failure of the walls, due to the fact of there being a single layer of reinforcement is, is not a major issue in Christchurch. I may be wrong in that but my general feeling of that. I, I think that there are issues in more heavily reinforced walls with two layers as well that perhaps we need to be looking at the lower level of reinforcement in walls and the upper level of reinforcement in walls itself. As you get more reinforcement the tendency for buckling under inelastic response becomes more serious and if there are big bars at the end being subjected to axial forces after yielding in tension and then coming back in compression the bars are more likely to pop the end of the wall out if they're close together and of large diameters and there certainly seems to be examples of that and just generally buckling of the wall itself becomes more of a problem. So I think that there are several examples in Christchurch where it would be good to look at both the low, the performance of walls with low levels of reinforcement and walls with high levels of reinforcement in quite recently designed structures and I think that the work that the Commission is doing is going to be extremely important and valuable in this area.

JUSTICE COOPER:

Q. Can I just ask very briefly a process type question arising out of the Department of Building and Housing investigation whereby consultants have been appointed to prepare the reports and then there's been an expert review panel above that on which the consultants who wrote the reports have also been, and I'm just wondering whether this has worked from your point of view in practice and has it created any tensions? Perhaps starting with you Mr Jury.

MR JURY:

A. I think the, well the expert panel that has over-viewed these reports is quite extensive in terms of the number of people on it and the consultants hold three positions out of I think it's a dozen, eleven, dozen, 11 to 12 positions so in that respect I guess the, the presence of the consultants on that panel is just a means of conveying the information to the panel and as well as the input into the other consultants' reports. I don't believe in terms of our experience that that has caused any tensions in, in coming forward with our report.

Q. The authors of the reports have been structural engineers and other people on the panel who aren't structural engineers have been, well I think there's an architect, there's a seismologist, there's a representative of local government, is that right?

A. That's right.

Q. Now, but in terms of the key issues as they have developed and been reported on they have been structural engineering type issues, haven't they? So I imagine that when it all comes down to it it has in fact been the structural engineers whose inputs have been determinative, would that be a fair inference?

A. In terms of the technical results that would be true but I know that in the expert panel there has been discussion regarding the seismicity. There has been discussion from the Territorial Authority representative on, on the bylaws that might have been in place at the time of the various buildings.
Q. Yes well that's all knowledge that is able to be ascertained reasonably readily I would have thought but the result of the process has been the report which Beca Carter's name on it written by a structural engineer.
A. Over the top of that of course is the panel report.
Q. Yes. And, but the, in terms of the key conclusions that were reached in that report I infer that the main inputs have been from those with structural expertise?
A. That is correct.
Q. Yes. And from the point of view of the panel Professor Priestley, how has it worked from your point of view?

PROFESSOR PRIESTLEY:

A. I think in general there has been a significant advantage in having the consultants on the panel as well because this has created the situation more of a partnership rather than of a group of structural engineers preparing a report and issuing it to the panel to review. And this has been necessary because one of the terms of reference has been the requirement that the panel approve of the, the report of the consultants. I think because the consultants have been there, if it had not been in this sense there would not have been the interaction between the consultants which I think has been quite valuable in some cases. In other words the people who have been looking at the Hotel Grand Chancellor have communicated their ideas as to what might have happened and so forth and discussed work on the PGC building as well and this has been possible because it's been in a panel which has always had the full group of people in there. There inevitably are some problems associated with that. I don't believe that there are insurmountable problems and certainly not with the PGC building.

Q. Yes thank you. Mr Holmes you've sort of looked at the process as an outsider. Is there anything that you want to add to what you've told us about the process in your report?

MR HOLMES:

A. Well I said in my report that I thought the process was an excellent one in terms of having a panel to overview individual engineering reports because that has been mentioned. Different engineers can come up with different opinions. It's not wildly different but certainly subtleties so having a panel to overview that process I thought was, was good. Having now been here at the conclusion of the report or some of the reports we know there's at least one building for which there has been some holdups for whatever reason and Professor Priestley has actually made a presentation which had some slight differences with the reports so there is an issue of approval but I mean those are my observations. It's the, the process was set up in an excellent fashion. It's only as good as you know the co-operation of all the people like in most endeavours.

1205

JUSTICE COOPER:

Any other comments?

PROFESSOR PRIESTLEY:

I would like to actually bring up another subject.

JUSTICE COOPER:

Yes.

PROFESSOR PRIESTLEY:

And I'd like to start by asking Mr Jury the vertical accelerations, that's the general subject area I'd like to discuss. It's – vertical acceleration to me is sort of like the weather, everybody talks about it but nobody does anything about it. Your model that took the vertical accelerations – was there a vertical degree of freedom in the horizontal members that actually got excited by the vertical accelerations?

MR JURY:

Potentially there was, but it didn't appear to be a significant influence. I think the issues are that I alluded to were that we can't always be absolutely satisfied that the vertical record is correct, but it's the best we have to go on, but certainly any vertical degrees of freedom would have been excited. The masses were placed on the wall as appropriate. There was the self weight of the various beams but not necessarily the slab in the floor areas, so you could argue that the full mass wasn't there in terms of exciting that floor system, but it should have modelled correctly the excitation of the walls.

MR HOLMES:

I note that in all three of the building reports which you have not dealt with two of them yet, but vertical acceleration is mentioned as a possible contributor but again none of the analysis have been definitive about well we took it out and it didn't do anything, or to what degree was it important? I know there is a speed of loading issue that people have talked about, very high frequency content of the vertical accelerations.

MR JURY:

We took it, we put it in and took it out of one of the other buildings that we looked at and its effect was not significant as we'll report later to the Commission, but that was the only case where we thought vertical accelerations might have had an influence.

MR HOLMES:

I notice that Professor Priestley said something about it earlier. I would like to ask him.

PROFESSOR PRIESTLEY:

Well I think that, yes I think there is a number of issues there. I do feel that it was significant. I'm not quite as negative about the influence or the actual integrity of the records in the vertical direction but certainly if I'm correct, Rob, and I think you're saying that it did not seem to have any significant difference to the displacements, the lateral displacements of the structure. Perhaps in terms of the columns themselves would the increased axial force have been incorporated and interpreted correctly in terms of the influence on displacement capacity of the columns and further given the time step which I think is quite – about .01 in the analyses.

MR JURY:
.02.

PROFESSOR PRIESTLEY:
What, .02, well that's rather large in terms of the, you know, to get a full representation of the vertical acceleration effects, you probably would have had to have had a time step of maybe a fifth or a tenth of that, so it may not be really representing the influence as much as –

MR HOLMES:
Ten times 28 hours, right.

MR JURY:
Yes. I said we used .02, but there was one of the parameters we did tests as you always do with these analyses and we didn't take it to an order of magnitude different, but we certainly doubled it and halved it, and we could not see any difference in those results. In looking at a large number of results from time history analyses for various buildings, it always seems to be a second order effect. In other words it's a super-imposed effect over the top of what was generally happening and you've got, in order for it to be a significant effect it's got to be co-existing with the primary effect, so if you have an axial load variation in a column the vertical acceleration's got to – to have a real effect it's got to occur at the same – it peaks at the same time as you're

getting peaks in the axial load and I think invariably it's not, it's slightly off that and so there is an averaging type effect in that.

PROFESSOR PRIESTLEY:
Well I'd comment on that and that which Commissioner Fenwick has already mentioned to some extent that looking at the records is one thing, but in terms of this structure moving with a large displacement halts as it responds. It's very likely that there would be high vertical accelerations at the same time because that's at a very high frequency and the other thing I would mention that time history analyses of other buildings investigated under the expert panels have shown increases in axial loads of columns of up to 80 percent and these were in columns that were heavily loaded already, and that seems to be quite a significant effect or a potential effect.

DR SHARPE:
But it comes down to the basic things that we've always designed for vertical acceleration, particularly where there are cantilevered members, and cantilevers like when you hold your hands out or wings of a plane, because they can be excited up and down, but for a general member, a load bearing member is already in compression from gravity and so that minor fluctuations in the vertical load in many cases are leading to an unloading of the gravity load or an increasing of it, and I would think that the example that you are referring to is a case where there could be a wing, a cantilever effect enhancing that perhaps.

PROFESSOR PRIESTLEY:
No, I don't think so, (inaudible 12:22:22) the building. So I don't think (overtalking 12:20:26), but anyway I think that it's something we need to look at, sometimes in these older buildings the axial loads on the columns are a higher percentage of their capacity under pure axial loads than perhaps would be the case in more modern buildings or at least should be the case in more modern buildings, and so it's possible and this is only – can only say possible

that these older structures were more sensitive, or some of them were more sensitive to vertical accelerations, or would appear to be from calculations.

MR HOLMES:
But the extremely high values are really the response values.

PROFESSOR PRIESTLEY:
Yes.

MR HOLMES:
So if you have this response going up a column it's got to excite the floor which in fact often could have frequencies about the same as those high frequencies, but it's very complex because it's not all the mass in the building that is being generated, those extremely high values. It's incredibly complicated that's why I asked you how it was modelled, because it would seem to me it would be very difficult to model that, and our use of vertical accelerations in cantilevers in my opinion is probably due to the lack of redundancy of a cantilever so if you have a cantilever you probably only have steel on the top side, and if it's thrown up you could have a problem and there's no redundancy. I think that's what the US requirements for cantilevers are, because we also use it for post tensioning that has the same reversal problem, that you don't think you're ever going to overcome gravity, but you could with vertical accelerations and so on.

MR JURY:
I think also that if you have a column that's very close to its capacity in compression, then anything that you haven't thought about is going to potentially be a hazard for it, and it could well be that we have not only in existing buildings, but also in new probably being taking the columns to far higher a degree of axial loads than perhaps was justified, and ignore – and not calculating the vertical accelerations to justify that wearing away of the margins.

1215

JUSTICE COOPER:

Do any of you have any further thoughts that you wish to leave with us?

MR JURY:

I just had one really. The February event was a very large earthquake in all, all the respects. It significantly would have tested any structure that received some of those motions we'd seen recorded at the various sites. There's no doubt that some more modern buildings were distressed more than we would have liked but invariably they held up and allowed people to get out. I think that was a testament to the engineering of them, notwithstanding that we always learn from earthquakes and we will always be putting those learnings into practice. But one, one of the things that I have struggled with in terms of understanding what it is, what makes a design acceptable is understanding really what is acceptable behaviour and in this case we had an earthquake that arguably was two to three times stronger than the event that we designed for. Sure we had a number of fatalities and some could have been avoided but what does society think is an acceptable performance? And was February really that unacceptable in terms of where we try to pitch our design from an economic point of view and dealing with all the other risks that we have to deal with in everyday life and so I just make the comment that it was tragic that we did lose life in February but we lost nobody in September which was arguably also a large earthquake from a design point of view.

DR SHARPE:

I've got one final comment and that is that if the public think that the proposed plans for Christchurch having buildings no larger than five stories is some sort of comment on the safety of high-rise buildings or their confidence in the design, they should not be worried about the safety of a new building above five stories. I think that we, we do have very good understanding of buildings, tall buildings and earthquakes and as I have often said to colleagues I would have been far happier in the aftershocks being in some the slightly damaged high-rise buildings in the city than in some of the accommodation that we were in round the city, low-rise.

JUSTICE COOPER:

All right well these are all issues that we have to grapple with. Thank you very much for your contribution to our work and this concludes our hearing into the reasons for the collapse of the PGC building. We have much to consider and the result of our consideration will be set out in some detail in our final report when that is delivered next year. Thank you everyone who has participated and I acknowledge again those who were directly affected by these tragic events. I said at the outset that some aspects of the hearing may appear to be clinical in nature and I am sure that has been one reaction that you have had but there are difficult issues that we have to grapple with and they need to be considered in a calm and structured environment such as I hope we have been able to provide for this hearing. So we will now adjourn briefly where we are going to hear from you on another matter Mr Laing, I understand, we'll just take five or so minutes.

COMMISSION ADJOURNS: 12.19 PM