COMMISSION RESUMES ON MONDAY 5 DECEMBER 2011 AT 10.00 AM

MR HANNAN ADDRESSES THE COMMISSION

JUSTICE COOPER ADDRESSES COUNSEL

MR HANNAN CALLS

HENRY JOHN HARE (SWORN)

Q. Is your full name Henry John Hare? You are employed by Holmes Consulting Group?

A. That is correct.

Q. Mr Hare, you have made a statutory declaration in this particular case and you have it there with you?

A. I do.

Q. Would you please read from paragraph 1 on page 1?

WITNESS READS HIS BRIEF OF EVIDENCE

A. I am a director of Holmes Consulting Group Limited (HCG). I have a Bachelor or Engineering (Civil) with Honours and am a chartered professional engineer. I hold professional memberships with the Institution of Professional Engineers' Structural Engineering Society of which I am the current President, and the New Zealand Society of Earthquake Engineering. In addition I am a licensed professional engineer in California. I have over 25 years of experience in structural engineering in New Zealand, England, Hong Kong and the United States where I was resident from 2000-2005. The majority of my professional career has been with HCG where I have worked at various times in Auckland, New Plymouth and Christchurch. My project experience has been mainly in buildings with a combination of both new building design and evaluation and strengthening of existing buildings. I am currently seconded for the majority of my time to CERA where I am the acting principal engineering advisor.

Scope of evidence – I, on behalf of HCG swear this declaration pursuant to the Canterbury Earthquakes Royal Commissions letter dated 17th of October 2011 relating to the Pyne Gould Corporation building.

Schedule of observations of damage – I am asked whether HCG as the engineers who inspected the PGC building agree that the schedule of observations of damage, as is attached to the Canterbury Earthquake Royal Commissions letter, dated 17th of October 2011, accurately records damage to the PGC building at the time stated. The extent of the damage as observed by HCG's inspection engineers who carried out site inspections on the 7th and 16th of September 2010, 15th of October 2010, and mid to late January 2011 are set out in the rapid level 2 evaluation report and site reports, copies of which have been previously supplied to the Canterbury Earthquakes Royal Commission under cover of my letter dated 5th of October 2011. Representative photographs were also taken during the course of the inspection undertaken on 16th of September 2010. These photographs were copied to disk and supplied to the Canterbury Earthquakes Royal Commission under cover of my letter dated 5th of October 2011. A schedule summarising the primary damage observed by the inspection engineers and the positioning of the damage observed during the inspection undertaken on 16th of September 2010 from recollection was prepared in April/May 2011. A copy of this schedule was supplied to the Canterbury Earthquakes Royal Commission under cover of my letter dated 5th of October 2011. Given the passage of time the inspection engineers advised me that they have little further specific recollection of the exact location or size of specific cracks or damage observed save as is recorded in the HCG documentation produced as a result of the inspections carried out. In the opinion of the inspection engineers who carried out the inspections of the PGC building the damage observed was relatively minor and not indicative of a building under immediate distress or having a significantly impaired resistance to earthquake shaking.

Original design and construction – I am asked to comment on whether the PGC building as originally designed and constructed complied with earthquake risk and other legal and best practice requirements that were current at that time. I am led to understand that the PGC building was designed in or around 1963 and constructed in or around 1966. The building was designed originally by IL Holmes, a predecessor company of HCG for the Christchurch Drainage Board. Beca Carter Hollings & Ferner Limited (Beca) has been commissioned by the New Zealand Department of Building and Housing to undertake an investigation into why the PGC building collapsed during a magnitude 6.3 earthquake that struck Christchurch on 22nd of February 2011. In its report dated 26 September 2011 Beca confirms that having had access to structural drawings dated 1963 Christchurch City Council's property file dating from 1978 to August 2010 its structural engineers report and site notes from 1997–2011 the building appears to have complied generally with the design standards and practices of 1963. HCG has not specifically considered in detail whether the building was fully code-compliant at the time of design but there is no reason to believe it would not have complied. Having considered Beca's report I do not disagree with its conclusion that the building appears to have complied generally with the design standards and practices of 1963.

Alterations and maintenance – I am asked whether the PGC building as altered and maintained complied with earthquake risk and other legal and best practice requirements that were current at the time of the alterations and maintenance. I am unable to comment on any alterations and maintenance works carried out to the PGC building save for those alterations and/or maintenance carried out with the involvement of HCG. As far as I am aware HCG has been consulted on various occasions but has never had any specific responsibility for the general maintenance of the PGC building. I am unaware of the level of maintenance carried out to the building. The only maintenance works carried out with the involvement of HCG of which I am specifically aware were carried out in early 2009.

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HCG was instructed to review the deterioration of the exterior concrete frames. At several locations there was evidence of what is known as concrete cancer where corrosion of the reinforcement has caused the cover concrete to spall. At that time repairs to selected areas were carried out by Contech who were engaged by HCG on behalf of the owner of the PGC building.

Alterations. In or around April 1997 HCG was instructed by Warren and Mahoney, Architects, acting for PGC as potential purchaser of the PGC building to evaluate the building for earthquake effects based on the requirements of NZS4203. HCG's report is copied at pages 1 to 21 of HJH1. As a result steel support posts were installed to provide backup to the exterior – sorry, column elements above the ground floor. Copies of the relevant building consent (strengthening detail) plans are copied at pages 22 to 25 of HJH1. HCG's calculations and sketches dated 2nd July 2007 the outlined structural specification report dated 31st of October 2007 to produce and construction monitoring statements dated 31st of October 2007 and the project features report dated 1st November 2007 are copied at pages 26 to 60 of HJH1. Further alterations were carried out by HCG in or around 2008. I believe that all works carried out by HCG to the PGC building complied with the accepted standards at the time such works were carried out. The seismic capacity of the building as altered was judged to have exceeded the minimum standard required by the Building Act for existing buildings at the time.

Seismic evaluation. I am asked to comment on the conclusions reached in the seismic evaluation carried out in 1997 and the recommendations made as a result. A copy of the seismic evaluation report prepared by HCG is as copied at pages 1 to 21 of HJH1. The conclusions reached by HCG are set out on page 17 of the report. At the time it was considered that the PGC building capacity was limited by the perimeter column rotation as to between 33% and 50% of the current code loading at that time. Assuming this weakness was addressed, HCG assessed that the building capacity would increase above 50% limited by the strength of the main shear walls. No specific capacity was given for the walls although it was noted that the assessed threshold for severe damage was predicted to be reached at approximately 60% of full code loading at the time. This exceeded the required capacity to satisfy the earthquake-prone building policy of the day. HCG recommended that at the very least secondary support columns be installed to mitigate the outcome of any column rotation failure. As a result of HCG's report and following consultation with the client, steel support posts were installed to provide backup to the exterior concrete column elements above the ground floor of the building.

The PGC's building, building's compliance as at 4th September 2010. I am asked whether the PGC building complied with earthquake risk and other legal and best practice requirements that were current as at 4 September 2010. HCG has not been engaged to perform any specific detailed quantitative evaluation to determine whether the PGC building complied with earthquake risk and other legal and best practice requirements that were current as at 4 September 2010. However, at the time of HCG's review in 2009 HCG concluded that the PGC building was unlikely to be considered earthquake-prone given the earthquake-prone building threshold of 33%. This conclusion was reached after having undertaken a brief comparison between the code as at the time of the 1997 review and that existing in 2009. As there was relatively minimal difference between loading standards NZS4203 1984 and ASNZS1170.5 2004 it was considered that there was little material change in the overall strength relative to the code. The Beca report dated 26th September 2011 comments upon this issue in greater detail.

Inspections post 4 September 2010. I am asked to comment upon the nature and effectiveness of inspections that were carried out between 4 September 2010 and 22 February 2011. HCG carried out inspections of the PGC building on 7 September 2010, 16 September 2010, 15th of October 2010 and mid to late January 2011. Instructions were received from NAI Harcourts who were the building owner representatives. HCG were engaged to carry out an initial earthquake inspection and securing measures as considered necessary. A copy of the documentation relating to our engagement was supplied to the Canterbury Earthquakes Royal Commission under cover of my letter dated 5th of October 2011. All inspections were carried out in compliance with level 2 post earthquake inspection requirements taking into account the verbal briefings given by Christchurch City Council, Civil Defence in regard to what was expected from level 2 assessments. Such inspections generally comprise a rapid visual inspection to identify any obvious signs of damage that might result in significant diminished structural capacity. The inspections are by their very nature brief and are not expected to include any plan review or analysis of the building or any sort of invasive inspection of the structural elements. This level of inspection was generally considered appropriate for determining whether buildings were suitable for occupation subject to recommendations for further detailed assessment. At no stage as far as I am aware was HCG requested to undertake detailed assessments. The extent of the damage as observed by HCG's inspection engineers during their inspections was not indicative of a building under immediate distress or having a significant impaired resistance to earthquake shaking.

Q. Right now you don't need to read the oath taken at the end of the declaration. I'd like first of all by way of supplementary questions to just ask you a little bit more about your qualifications and experience. Your statutory declaration says that you are a licensed professional engineer in California. Would you tell the Commission please something about your experience in California, how long were you there, what you did and so forth?

A. Yep, certainly. Um, I first went to California in fact in, ah, 1994, um, on secondment to a firm called EQE International. They were at that stage specialist earthquake risk consultants and I was there for the purposes of doing in fact post earthquake inspections in Northridge following the earthquake there. From that we formed an association with EQE doing risk assessments in New Zealand. I was in fact managing that business for a number of year down here. In 2000 I went up to the, to the States. We were purchasing a company up there to, basically to have an office in San Francisco. During that time most of my work was spent doing seismic evaluations of buildings up there, including those few buildings remaining that they had seismic damage from the Loma Prieta earthquake. Most of my time was spent working on older masonry or concrete buildings in that regard.

Q. Did you have to take any additional examinations or tests or other form of scrutiny to operate in California?

A. Ah, yes certainly I did. The requirements up there for gaining professional engineer status are that you have to go through a number of tests including tests on seismic design applications in the US.

Q. Thank you. Now just changing topic, I'd like you to look at a document which I'll call for on screen, this is CAM2330198.

WITNESS REFERRED TO DOCUMENT

Q. And we should probably enlarge this section by section. Commissioners I do have some hardcopies of this available if that would be helpful.

JUSTICE COOPER:

Well we can see it on the screen thanks Mr Hannan.

EXAMINATION CONTINUES: MR HANNAN

Q. Yes well will you tell the Commission please what is this document?

A. Um, this document is a, a standard form produced by the Christchurch City Council shortly following the September event, basically a statement they requested engineers to sign in support of reoccupation of buildings.

Q. Look at the base of the page where you've got CCC red yellow tag removal conditions revision A 12 October 10.doc. Can you comment on when this document was produced to Holmes?

A. Ah, well certainly it would have been very shortly following that date, I don't recall the exact time.

Q. Can you recall who, who gave it to Holmes?

A. I don't recall in detail who gave it to Holmes. I know it came through, um, from discussions with the City Council, um, potentially

David Hopkins and Neville Higgs who were working with the Council at that time.

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Q. And that tag on the bottom, "Red, Yellow Tag Removal", can you comment on that please?

A. Yes. Following the earthquake obviously there were placards put on buildings. Red and yellow placards indicated, um, that the buildings were not permitted to be occupied. This was a certificate that had to be signed as a condition of the re-occupation of those buildings.

Q. Right, so a certificate that had to be signed by an engineer as a condition of re-occupation where the red or yellow tag was being removed. In other words, the building was being green placarded?

A. Yes, yes.

Q. If we can go to paragraph (a) please by way of enlargement. It's about a third of the way down the page and this is what the engineer first of all required to certify. "Where the structural integrity and/or structural performance of the building (or part of the building) was materially affected by the Darfield earthquake or any aftershocks to date, interim securing measures have been taken to restore the structural integrity and performance of the building to at least the condition that existed prior to the earthquake of 4 September 2010." Can you comment on that paragraph and what you understood your task was as a result.

A. Yes, I think it was well understood that by implication we were looking at all buildings to determine whether their capacity was materially not affected by, um, the earthquake or had been restored to the condition it was in prior to the earthquake. Um, by implication any building with a green placard would be in that condition and therefore able to be occupied.

Q. Now I'll just get you to move on from that document. If we can go to CAM233.0026. This is the 1997 Seismic Evaluation Report – 233.0026. Just have a look at that. Now this is the report which you're referring to at paragraph 23 of your Statutory Declaration –

A. Yes

Q. – and if we can come right to the last page which is 0026.21 and in the references, reference number 7, is a reference to a book, "Seismic Design of Reinforced Concrete and Masonry Buildings 1992". The authors are T Paulay and N Priestley?
A. Yes.
Q. And so that was a reference that was used in doing this work with respect to PGC in 1997?
A. It was one of the references used, yes.
Q. Now if we can look please at document BUICAM233.196.17, 233.0196.17. Now this is, as you know Mr Hare, part of a presentation which, it is proposed Nigel Priestley gives to the Commission in terms of his assessment of the PGC buildings or issues in relation to the PGC building –
A. Yes.
Q. – and this page is headed, "Issues with Holmes' Analysis" and if we can have a look at that first bullet point please where Mr Priestley says that it appears that the critical region for the shear core was incorrectly identified as the base of the wall and says a couple of other things about that. What's your comment on that bullet point?
A. Um, I comment basically that we have verified, in fact the model included, although the report itself may have been slightly ambiguous on that point, the model in fact allowed for flexural yielding elements at level 1 and the model at that time was shown to have, um, the point of failure of the walls would have been, if anywhere, above level 1, exactly as stated there. So in fact we had that covered.
Q. Yes, now in support of that I'll just take you to another document. We will come back to this page but if we can look at CAM233.0035A4.13. This is a letter written by Holmes – in fact yourself – on 25 March 1997. 233.0035A4.13, and if we can enlarge paragraph 2 please. So first of all can you tell the Commission what is this letter?
A. Um, this letter is a preliminary report, if you like, ahead of the main report that was submitted. It was given to, um, our client at that point, um, I don't recall exactly the circumstances but I think they were

probably pressing us for, um, answers as soon as they could get them but this was certainly a summary of the findings of the computer model ahead of the final report.

Q. Ahead of the final report which was produced in April. If we can come back up in terms of enlargement to the first numbered paragraph 2 please. So if you read that second paragraph 2 and tell the Commission what that paragraph is talking about?

A. Ah, yes, 'at levels in excess of this significant cracking and movement develop in the main cross walls, (initially at the wall on line B, through the piers and the lintels of the walls adjacent to line C), and in the lintels of the main wall on line E. This damage is focused at the first floor level due to the presence of significant extra walls from ground to first floor levels'.

Q. Now where you talk about these walls here, you're talking about the shear walls. Is that correct?

A. That is correct. These are the central walls running in the east-west direction as part of the shear core.

Q. So in this paragraph you're saying this damage is focused at the first floor level?

A. Yes.

Q. Right if we can come back to 23340196.17 which is the Priestley report please. So looking again at that first bullet point and recalling what you wrote on the 25th of March 1997 what more would you like to say about that first bullet point?

A. Ah, well it would appear that we are in agreement as to the outcome, um, but we had identified in fact the critical regions of the wall.

Q. So if it was said to you that in fact you had not identified or indeed incorrectly identified the critical regions for the shear core what would you say to that?

A. I would disagree strongly.

Q. Then if we come down to the next two bullet points – Method for modelling plasticity at wall base inappropriate. Stiffness of columns and beams was too high – what do you say to that?

A. Um, I think it's important to view the context of the report. It was done in 1997. At that stage there wasn't as much guidance as there is available now and some of the, ah, limits if you like for assessment of elements and, um, methods of modelling were certainly different then.

Q. There was I think a United States document – FEMA 273 – produced shortly after you had done this work in 1997?

A. Yes that is correct. That document I think was published, um, in September/October of that year.

Q. And what is "FEMA"?

A. FEMA is the Federal Emergency Management Agency I think in the US, um, and they were an agency which has published a lot of, um, documentation which is used in the assessment of existing buildings.

Q. In the assessment of rehabilitation of buildings also?

A. Yes.

Q. So that was not available to you at the time of this work in early 1997?

A. No it was not.

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Q. So what is your observation about the proposition that stiffness of columns and beams too high and the other observation in that next bullet point?

A. Well primarily that's – we were performing the work to the best standards or guidance available at the time. We were using references where we could that assisted us in developing those models to the extent that we were able to do so.

Q. And of course in your 1997 report you referred did you not to a reference work of which Mr Priestley was an author?

A. Among others, yes.

Q. Any observations about the bullet point column plastic rotation capacity underestimated?

A. Yes, once again that was a measure which hadn't been at that stage published in FEMA 273, although I note that the range of .007 mentioned there was exactly in fact the limit which was published later in the year in FEMA 273. So we were if you like anticipating what was coming up, unfortunate or otherwise.

Q. So you would disagree with that proposition that the plastic rotation capacity was under-estimated?

A. Well not in the context of the time, no, it wasn't.

Q. Well let me get clear what you're saying. Are you saying that it wasn't underestimated in the context at the time?

A. Yes.

Q. Now the last bullet point. There are only one set of records used for the analyses. Firstly can you explain what's the reference to records there?

A. The reference there is to the type of analysis, so it's called a non-linear time history analysis. Essentially it takes shaking records from earthquakes around the world, and in this case scales them to the intensity relative to the level of the portion of code we're looking at if you like, and applies that to the model, so it's a reference to earthquake models which have been scaled.

Q. Yes, and just so we're all on the same page with this, what we're really talking about here, correct me if I'm wrong, is you have a computer model of the structural elements of the building, certain assumptions built into that?

A. Yeah.

Q. And then you feed into that computer model the actual earthquake records?

A. Yep.

Q. Of the lateral movement in the earthquake, the acceleration in the earthquake to see how the building behaves according to that model, if that earthquake happens?

A. Yes, exactly.

Q. Well what do you say to the – to what appears to be a criticism that only one set of records were used for this analysis?

A. Well again that was reflecting the nature of what we were doing in 1997, and also the fact I guess that there were relatively few records available at that time for use in that form. Certainly now we recognise that we would be using anything from three to seven records according to what type of analysis we were doing.

Q. And what do you say about the degree to which this approach would conform to NZS4203?

A. I believe it was conforming generally at the time.

Q. Now I want to change topic here and just ask you about an initial discussion that you had with Mr Buchanan of Harcourts at the time – immediately after the 4 September 2010 earthquake and he's given evidence that he had a discussion with you about arranging inspections of buildings?

A. Yes.

Q. You've said, you mentioned something about this in your statutory declaration. Can you expand on that so far as you recall your content, the content of your discussion with Mr Buchanan?

A. Certainly, yes. Mr Buchanan called me, I don't recall exactly when, it may have been on the 4th, it may have been the next day, asking if we could perform some inspections for him. He's mentioned specifically at that stage several buildings and I don't recall which ones, but I said that we would certainly be pleased and do that, but requested that he called Richard Seville who was organising, coordinating our efforts from the office.

Q. So far as you recall was there any discussion of what the nature of your work would be, what would you be assessing, how would you be assessing, what would you be doing?

A. So far as I recall the discussion was along the lines of performing some initial inspections to determine whether the building should be occupied or if they should be evacuated, or if further work would be required.

CROSS-EXAMINATION: MR MILLS

Q. I just want to ask you a few questions initially about the knowledge that you had arising from the 1997 report that my friend Mr Hannan just took you to and also the 2007 work which he didn't take you to, but I'm assuming you're familiar with both of those, I don't need to remind you –

A. No that's right, I'm familiar.

Q. – of what's in them. Now I don't know whether you've reviewed or watched or been briefed on the evidence that was given last week by your two colleagues, Mr Whiteside and Mr Boys, but I just want to ask you some questions around what they were asked and just get your reaction to some of that. Now as you probably know they both said that when they went out to do the four assessments that collectively they did at the PGC building, that they didn't know anything about the two reports, this 1997 and 2007 one in which you had been involved, or more generically in which Holmes had been involved, and they also said that they didn't know anything more generally about the issues that had been identified in those two reports about the potential seismic performance of the PGC building, and I assume you're aware that that –

A. Yes.

Q. - was what they said and as I think you know there's also a letter which came from your firm responding to questions the Commission counsel had put to you, which essentially confirms that as well?

A. Yes.

Q. Now the point that was of interest I think arising out of that, at least to me in the hearing last week, was that they both then went on to say that not only did they not know about the issues that were raised in those two reports, but it was irrelevant any rate to the assessments that they were doing. So in other words they didn't know but it wouldn't have mattered even if they had because it was irrelevant to what they were doing.

A. Mhm.

Q. And they made the same comment about the structural drawings.Again hadn't looked at them, it would have been irrelevant to what they were doing, and as you probably are aware, the reason that they gave for saying that those matters were irrelevant was that they were doing a level 2 assessment and that was – those sorts of issues were irrelevant to that. Now I take it none of that is surprising to you in terms of the way they went about this?

A. No.

Q. They then went on to say that knowing those background matters, looking at the structural drawings, none of that was necessary to the advice that they gave that the building was safe to occupy. Now there's a question that's coming, I'm just putting to you the –

A. Yep.

Q. – what they said so I can get your reaction to this, and they then said, I think they both said this, "That that was because what they're doing at that stage is about determining diminished capacity of the building and not the building's performance."

A. Yes.

Q. And that's a distinction I take it that you would have expected them to draw in doing these assessments?

A. Yes.

Q. Now what I'd just like on – having put those matters to you, what I'd like to ask you about is what you would have done. I want you to put yourself into the position where you're the one who's doing these assessments rather than Messrs Whiteside or Boys.

A. Mhm.

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Q. So you're going out to do this inspection of the PGC building, and I want you to assume some other matters which they said they were aware of. The first is that you know that the inspection is being carried out because there are worried tenants and they have expressed concern about the safety of the building and secondly closely related to that the tenants want an assurance that the building is safe for them to be in. So that is a second point that they were aware of and now I am asking you to be aware of that as well, and you know that what you say in the report you give will be conveyed back to the tenants and staff of the PGC building. So what I want to know really is given that you had this knowledge about the seismic capacity, if I can call it that, of the PGC building based on your 1997 and 2007 reports and you are going out to inspect the building, would you have considered the knowledge you had to be relevant in any way for the assessment you were doing of that building knowing that the tenants were concerned and were seeking assurance it is safe to occupy?

A. Well I clearly, we understand the concerns of the tenants and so even though we were doing work for the building owners we would certainly be wanting to address those concerns on the way round, quite understand that need. I think it is important to note that engineers were inspecting many buildings some of which information may have existed for many which it did not and so the standard instruction if you like and certainly my own approach to that is to at all stage when entering a building to make sure that we can identify the structural system in such a way that we can figure out what is holding it up and therefore what is critical and it is important regardless of information that people think they have or think they know about the building not to have pre-conceived notions in fact about how it will behave but to use your eyes and observations and engineering judgement to be able to determine whether in fact the building capacity had been diminished which was what they were searching for. So in that context I think the presence or absence of a report which may or may not have the conclusions right which may or may not reflect the buildings performance was almost a sideline. The most important thing was to understand what had happened. If in any engineer's estimation or their observation they are not able to form an opinion then at that stage a different approach would be taken. Certainly to bring in other people and at that stage look for any information that may be available to give them any guidance but at no stage in that building was that the case.

Q. So I think what you are telling me, tell me if I have got this wrong, that if you had been carrying out the assessment that you would not have found that existing knowledge that you had about the building of any relevance to what you were doing?

A. I would not say that but what I would say is that it would have allowed me to take less time, if you like, understanding the building than perhaps what I would have had I not known anything about it but certainly in the case of that specific building the structural system was quite evident to anyone walking in. You were met straight away by the shear walls and therefore it is quite obvious what would be happening. So to that end it would have, it made no difference to their conclusions, no.

Q. You recall and I can take you to the document if you want me to, but again I think you have said you are familiar with these things, the 2007 report where you recall Holmes was being asked by, I think, again by Warren and Mahoney to look at various alternatives for the development of the site or additional floors to the building and so on, referred to the presence of a critical structural weakness in that building which was not the one, as I understand it, that was referred to in 1997?

A. I do recall the report, I think in fact it was because we were still talking about the perimeter columns, in that case the offset I think from ground floor to the levels above, however, it is important to note the context of that report that we were looking at alternatives to add mass to the building, add extra levels to the building at which point the behaviour of the building would be quite different plus we would then have to consider it in light of the need to have the building fully code-compliant as opposed to the provisions which exist for existing buildings, and so contextually it is quite a different point I guess.

Q. But is it not relevant that in 2007 you were still describing it as having this critical weakness to it?

A. Well it was critical in the sense that if you were adding a lot of mass to the building and reviewing it in as for 100% compliance then that particular point would have been a weakness but with the building in its unaltered state that particular point was not considered a weakness in that sense.

Q. So again nothing in there that would have been relevant if you had been carrying out the assessment in –

A. No nothing at all.

Q. – 2010? We were taken last week to the council LIM report which had a listing of the PGC building as being potentially earthquake-prone. Now just in case you were not aware of this it became clear that what the council had done was effectively an automated sweep of all buildings built prior to 1976?

A. I was aware of that.

Q. And again both Messrs Whiteside and Boys took the position that it was essentially irrelevant for the same reasons. They were not aware of it but would not have been relevant. Would that have been your position that if you had been aware that this building was potentially earthquake-prone according to that assessment anyway, would you too have treated that as being irrelevant to the assessment that you are making in September?

A. Yes completely.

Q. And for the same reason that –

A. Yep.

Q. – you have given already?

A. Yeah, that is, as you have already noted, it was purely based on the date of design of the building and therefore was a very broad brush, it had no analysis behind it. I would not have given it any thought.

Q. And it would not have surprised you either should I assume that the building might have been earthquake-prone by current standards?

A. Well you would have to make a judgement on the spot but that was actually not relevant to the damage assessment which is looking to see what damage has occurred, not trying to make an assessment of the strength of the building.

Q. So it is looking backwards this assessment, not really looking forwards at what might happen to it in another earthquake? Is that right?

A. Well yes and no. It is looking at a building with a view to establishing whether there has been significant damage which might reduce its capacity beyond what it already had prior to the earthquake.

Q. Yes. I just want to put again a series of points to you, propositions I suppose, and they relate to this issue about how one identifies a critical structural weakness in a building and just again see whether you agree or disagree with me and in a sense I am neutral as to what you say, I just want to know what you say. So the first proposition when if one is trying to identify whether a building has got a – an existing building – whether it has got a critical structural weakness in it. The first proposition is that for an assessment to be meaningful then before it is done you need to know something about the building?

A. Yes.

Q. You agree with that?

A. Oh, assessment in the sense of evaluating its strength?

Q. Yes.

A. Yes.

Q. Or going out and doing an assessment of a building, to find out whether it has got a critical structural weakness in it?

A. Yes if you are doing an assessment of the strength of the building you would certainly look for that.

Q. And second point is that in order to understand a building the second thing you need to do is if you before you go looking really you need to look at the potential critical structural weaknesses that might be in the building and then you go out and look more closely at the potential areas of critical structural weakness?

A. Again that would be applicable to doing an evaluation of the strength of the building. Not so if you are doing a damage assessment where you are looking to see what has happened.

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Q. Okay. And the third point is, sorry yes a third point is that you'd then go and look at the building with this knowledge that you've already formed at that prior examination stage before you actually go into the building, then you'd go in and you'd take a look at it with that background information of the potential areas of weakness and so on and that's the sequence within, within, which one would want to do this. Is that correct?

A. Well again that is the sequence if you were doing a detailed evaluation of the building as opposed to, um, looking to see what has happened as a consequence of the earthquake.

Q. Yes.

A. A damage review.

Q. And if you were doing that kind of detailed assessment would you also agree that you really have to examine the key structural drawings in order to understand that building?

A. Um, I would agree that if you had access to those and you were doing a detailed assessment that would be a point to start, um, but again not applicable to a damage review.

Q. I want to turn then to the issue that my friend Mr Hannan touched on as well and this is the fact that what Messrs Whiteside and Boys did was a level 2 assessment, and you were asked about the contact you'd had with Mr Buchanan?

A. Mhm.

Q. And that you'd passed that on to Mr Seville for him to advance that as I understand your evidence. Now I just want to ask you a bit more about why this was done as a level 2 assessment. Mr Boys said in his evidence that he was never specifically told to do a level 2 assessment, that it was just his understanding of what he was supposed to do and then, and this leads to you, then when I asked Mr Whiteside why he treated this as a level 2 assessment he said, "Oh that's a question better answered by John Hare", so here's your chance. There's, as you know there's no reference in the contract that was drawn up between Holmes and Harcourts for a level 2 assessment. You were aware of that weren't you?

A. Yes.

Q. An initial inspection I think is the language that he uses. As I understand it the level 2 assessment was terminology that was developed during the emergency response period. Is that your understanding?

A. It was terminology which I think was adopted during that period. It in fact predates that because it comes from the earthquake engineering rapid assessment guidelines.

Q. Yes.

A. But certainly, um, in those very, very early stages immediately following the earthquake it was not common terminology.

Q. Yes. So what’s your answer to the question Mr Whiteside’s directed back to you about why this was done as a level 2 assessment when it was the result of a private contract between the owner of the building via Harcourts and Holmes Consulting Group and it wasn't something that was being done under the, specifically by reference to a level 2 assessment at least as far as the instructions were concerned?

A. Well I, I think that's really a case of the terminology catching up with what was in the contract.

Q. I see.

A. Um, as it, the contract I think referred to rapid structural assessment from memory.

Q. Initial?

A. Initial sorry. And, um, which later, um, the jargon if you like which developed around that was that that was a level 2 as commonly referred to.

Q. So do I take it that what happened here is that despite the wording in the contract the assumptions that were made by Holmes was that that's what was required a level 2 assessment?

A. Um, that was what was required, um, in the case of a building such as that which wasn't displaying any diminished capacity.

Q. Right, well the final point I want to ask you about which really that leads into is the disconnect I suppose which seemed to emerge last week in the evidence between what the owner of the building – Mr Collins – had asked for and what he got. He said that his instruction was that he wanted to be assured that that building was safe to occupy before the tenants went back in. After that he left it to Harcourts to deal with taking that forward. What is it that given that we’re dealing here with lay people who don’t know the engineering terminology, what is it that Mr Collins should have said if he really wanted to be able to assure his tenants and their staff that they were in a building that was safe to be in in what appeared to be an aftershock sequence that the city was then experiencing?

A. Well it's a difficult question in the context of that but obviously, um, with hindsight a detailed engineering evaluation may have been what he could have requested or words to that effect. However, I can't comment on the discussion between Mr Collins and Mr Buchanan, um, when we're dealing with Mr Buchanan we're talking about the entire portfolio of course.

Q. Yes, yes and I'm not asking you to.

A. I see.

Q. Yes, no I'm just asking you to, to take at face value what he said in his evidence that that's what he wanted?

A. Mhm.

Q. He had concerned tenants. He wanted to be able to give them an assurance that the building was safe to be in.

A. Mhm.

Q. Before they went back, and just wanting some guidance from you as to what he should have asked for if he did want to be able to assure his tenants that in what appeared to be an aftershock sequence that building was safe to be in.

A. Sure, and understanding that for a lay person they don't have the technical, um, jargon, um, the, the wording if, if we'd been asked for specifically a detailed study to go through then we would have given that but, um, by the same token in the, doing the rapid assessment that we did had we had cause for concern at that point that the building's capacity had been diminished then we would've undertaken or suggested further investigation as required.

Q. Mhm.

A. Um, certainly if we'd seen a building without, sorry with diminished capacity we'd've taken a different approach.

Q. Mhm. But as I think is quite clear by now the analysis that's involved in diminished capacity is simply asking whether the building's any worse than it was before the September earthquake and then again after

Boxing Day and not whether the building's a good one to be in if it gets hit by significant ground shaking?

A. Yes. Although I think you've got to put that in the context, the, um, the September event, a normal aftershock sequence we certainly expected there to be aftershocks.

Q. Mhm.

A. And the evidence, the visual evidence and the judgment involved in this, in finding that the building had no diminished capacity would therefore reasonably lead one to expect that it would be able to continue to resist earthquakes in the same way as it had previously. Um, what happened of course in February which was a dreadful tragedy was that we had an earthquake which was considerably larger than even the design earthquakes that we would design a modern building to and so couldn't have been anticipated by either of those, um, forms of analysis.

Q. So are you telling me then that in carrying out the assessments, these level 2 assessments that there is an element of looking forward at how the building might perform in future earthquakes or aftershocks?

A. It's, it's a combination of looking forward and looking backward. By, by reviewing what's happened in the earthquake you've just had you're already forming an opinion as to whether the building will continue to have as much resistance to another earthquake of the same scale.

Q. As it did before?

A. As it did before, correct.

CROSS-EXAMINATION: MR ELLIOTT

Q. Mr Hare, I'd like to start just by acknowledging your standing within the engineering profession and you've given evidence that you have an honours degree in engineering and that you have 25 years of experience in New Zealand, England, Hong Kong and the United States, that's right?

A. Yes.

Q. And you're currently president of the Structural Engineers Society?

A. Yes.

Q. And are you also a member of the Department of Building and Housing Engineering Advisory Group?
A. Yes. I am.
Q. And you're a director of Holmes Consulting Limited?
A. Yes.
Q. Which is an internationally respected engineering firm?
A. Thank you.
Q. You don't need to be modest here, you're under oath?
A. Yes.
1100
Q. So I imagine that you personally have put in hundreds, or perhaps even thousands of hours of unpaid work over the years in doing your best to maximise the safety of buildings here in New Zealand and overseas?
A. As best we can.
Q. And there would be many other engineers that you're aware of who have worked equally tirelessly and in a committed way to ensuring the structural integrity of buildings here and overseas?
A. Yes.
Q. And also I expect that the period after 4 September was a particularly challenging and stressful one for you and the other engineers at Holmes Consulting, was it?
A. As it was for many people.
Q. I'm just going to ask you if you will apply all of your expertise and your knowledge to identifying some lessons that might be drawn from the tragedy of the collapse of the Pyne Gould building. Will you do that today?
A. Happy to, yes.
Q. And I'm going to ask you really about three areas where some lessons might be learned. Those are firstly the way in which engineers identify life safety risks in buildings, secondly when engineers communicate risk to building owners, and thirdly the way engineers inspect buildings for safety following an earthquake. So turning to the first of those three points, the way engineers identify life safety risks, as we've seen that

you personally had some involvement with the PGC building for a number of years, going right back to the 1990's. Is that right?

A. Yes it is.

Q. You're aware that BECA and the Department of Building and Housing panel and Mr Holmes have all agreed that the collapse can be attributed to the failure of the shear core between level one and two. Are you aware of that?

A. Yes.

Q. Do you agree with that conclusion?

A. Yes I do.

Q. Turning to the 1997 seismic analysis that you prepared, or that Holmes Consulting prepared, do you agree that that analysis indicated that column failure would precede all failure?

A. That was the conclusion at the time leading to the recommendation of putting the prop columns in behind.

Q. You would appreciate I hope that families, bereaved families and those injured would be concerned that on the face of it at least a weakness appears to have been identified back then but not addressed as a life safety issue?

A. No I don't agree. The – unless I've misunderstood the question, the recommendation at the time was that some prop columns be installed behind the perimeter columns in order that if there was a significant movement of the building, enough to cause the failure of those columns, the prop would be there to take its place. It's at which point assessed to become a critical weakness.

Q. Could I just ask for document WIT.HAI0001A.8 to go up. So is this the letter setting out the advice in effect to the PGC Group from Holmes Consulting?

A. Correct.

Q. And just for the record, that report does refer to it being interim, preliminary, initial preliminary recommendations, but you made the point earlier on have you that in fact the full seismic analysis was then completed so that the recommendations here confirmed –

A. I don't recall it.
Q. – were confirmed in detail.
A. I don't recall in detail the timing but certainly this report was done after the analysis was completed but before the full report was issued.
Q. I see. So do we treat these, even though they're described as being interim and preliminary they won't affect the final advice that was given to PGC?
A. Almost completely, yes.
Q. And the evidence, just to put this into context has been that the Warren and Mahoney report to the PGC Group really picked up on advice that drew a distinction between life safety and damage reduction, and that is what this letter does, doesn't it, it draws a distinction between life safety and damage reduction, and am I right in saying that if we go to the next page, which is 5.13, I'm not sure if you can see this but it's in that first bullet point up the top.
A. Yes.
Q. You're talking there about damage reduction, consideration being given to strengthening the transverse shear walls?
A. Yes.
Q. Is that the only one of your recommendations that related to the shear walls in any way?
A. Yes it is.
Q. And in the paragraph below the bullet points you say, "Note, that we consider the life safety issues above are essential but the damage reduction measures are optional?
A. Yes.
Q. So in this advice to PGC you're addressing what you described as life safety, risks and damage reduction risks, and I'm just going to ask you some questions around that assessment of life safety risks. Is it correct to say that you assessed those life safety risks by reference to a particular percentage of code loading?
A. It was a two-fold assessment, certainly a percentage of code loadings the starting point, and obviously if it got too low it would be at the

earthquake-prone building end of the spectrum which it wasn't, but also in respect of the consequences of failure of the element, and so if one of those columns was to have failed it would drop a significant amount of floor which would be definitely a life safety issue, whereas we saw the failure of the walls as being more of a gradual issue, noting that this was, ah, assessed against the code, proportions of the code as opposed to something considerably higher than that.

Q. Well according to your statement you say in paragraph 24, assuming this weakness, and I think you're referring to column issues, assuming this weakness was addressed, HCG assess that the building capacity would increase above 50 percent limited by the strength of the main shear walls. No specific capacity was given for the walls although it was noted that the assessed threshold for severe damage was predicted to be reached at approximately 60 percent of full code loading at the time.

A. Yes.

Q. So just let me clarify, I understand that correctly, firstly according to that you were saying that the work that was in fact carried out would have increased the building capacity above 50 percent of code, but that was limited by the strength of the shear walls. Point one?

A. I could make a structural distinction there which was that the work done to the columns didn't actually increase the lateral capacity of the building. It was simply done to ensure that the columns would be able to support the load or that the prop columns, so that that building if you like could achieve the full strength of the shear walls, so it didn't actually add strength, it simply added a bit of robustness to the rest of the structure.

Q. Is it right to say that this evidence in your statement and what's in the report reflects a conclusion that in effect life safety risk was addressed by carrying out work which would bring the building to something much less than full code loading, namely 60 percent of loading?

A. I would say life safety risk in the context of something less to or up to a full 100 percent if you like code loading. We had actually run the model at 100 percent to verify that we wouldn't be expecting complete collapse of the building and that was in fact what – and that wasn't expected, and

so the onset of failure in that sense of the walls would be quite a long way further from the actual complete failure.
Q. You're not suggesting are you that the work resulted in the building being upgraded to 100 percent of code?
A. No, no.
Q. But it would be upgraded to something much less than 100 percent of code?
A. Yes.
Q. Sixty percent of code?
A. Well that's where we set it. The onset of that failure was starting to occur.
Q. So my question for you really is how an engineer might make this assessment of where the life safety threshold comes into play, and you can see that's an issue which is of relevance moving forward?
A. Of course.
1110
Q. So I'm just going to ask some questions about that and to put it into context. Firstly the Building Codes set particular levels of force which a building must be built to be capable of sustaining. Is that right?
A. In general terms, yes.
Q. And those forces are calculated or taken into account as one factor, a particular level of ground accelerations which are defined by reference to a particular magnitude of earthquake at a particular distance from the building. Is that right?
A. In general terms, yes.
Q. And is it right that it's still possible that a building built to code could collapse if the earthquake is greater than or generates forces greater than those forces which the code has contemplated?
A. Yes that is correct.
Q. So in effect the code represents the allocation of a particular level of accelerations but there is inherent there a balance between the desire for life safety and the cost of a building. Is that right?
A. Yes there's an inherent risk management behind that.

Q. So in giving advice to PGC about in effect the percentage of code at which life safety was addressed were you not making your own subjective decision about the level of force the building should be able to bear?
A. No. I think you have to look at that in the context of the requirements of the Building Act at the time. The only thing that applies to existing buildings, if you like, is the measure of earthquake proneness which was a considerably lower test than that. So we had a building here which was certainly well above earthquake prone by either the measurements then or probably measurements now, um, and we were looking to make, um, improvements to the building if we thought it necessary to, you know, specific things which we had concern about. We felt that given that the failure of the columns would be, under those circumstances, rapid and catastrophic. We felt that therefore represented a life safety issue as we would have defined it at the time, whereas the fact that the walls may start to yield around about that 60% level but could sustain their load for a considerably greater displacement didn't make that a life safety issue in the context of the code and the requirements at the time.
Q. Would you accept that if you like you can put the PGC situation to one side and just think in general terms but would you accept that in allocating a percentage to which a building could be up-graded there is a subjective assessment involved?
A. There is a subjective assessment where we could as engineers we can supply the numbers if you like and put the information in front of the owners in assisting with making a decision that they are guided not only by us but what's in the Building Act regarding what their obligations are as owners so we are only one part of the equation if you like.
Q. Where you have an owner who is willing to do a voluntary up-grade do you think there's a bit more scope than was indicated here for a discussion with the owner about how life safety is identified and how they might decide how much to spend on an up-grade?
A. Um, I guess we weren't specifically, um, we didn't actually engage with the owner at that point. We were working through the architects.

However, we had discussed what that meant with the architects at the time. I don't think, you know, we would necessarily use the same language today but certainly that was the way it was discussed at the time.

Q. Turning to the 2007 document that you've just been questioned on, you said in this document, and I'll just quote. It was under the heading, "Existing Structure". You said, "The existing building has an unusual structural form that may work to our benefit. The columns step across at the first floor to create the structural set-back that is a part of the existing architecture. This is a severe weakness seismically as this discontinuity has the potential for severe failure." You may wish to read the document but it seems that you are making an assessment of the existing structure as opposed to commenting upon how it might behave in the context of alterations as you indicated to Mr Mills?

A. Um, no not really. The, um, key point there is that there was a cantilever then which supported, which spanned across the ground floor columns to support the exterior columns and so in the, um, if you were adding a significant extra load to the building and/or if there was a, um, significant and much larger event you could potentially overload the cantilever which would be therefore a shear failure in the concrete which would have been regarded that way but we were looking at that building at that stage in the context of possible future extensions and therefore wouldn't have had the same loading onto that beam.

Q. Just so that everyone can understand exactly what we're talking about here because it can be difficult, can you just describe exactly where that weakness was and it may assist if we just produce the plan so that you can point out exactly what you mean. Would the ground floor plan assist or the first floor plant?

A. If you have the ground floor plan there I think it has a cross section where I can show you.

Q. So we're looking at this from above. Are you able to indicate?

A. I think I can if this mouse works. So these red columns you see around the perimeter.

JUSTICE COOPER:

Q. Is this the best plan to be using?

A. Well the original plan is actually a better one because it has a cross section of the building and part on it as well if you have that available. I'm not sure where that might be.

Q. Some plan which assigns numbers or letters to the various gridlines would be the best for the purposes of the legibility of the transcript at a later date because the mouse doesn't. It's very difficult to recreate what you said. Let's just see if we can find some other plan.

A. That would be good.

Q. So we're now looking at CAM2330159.6 all right.

COMMISSIONER FENWICK:

I think it might be clearer on CAM2330051A.

JUSTICE COOPER:

Q. So if you're going to be talking about this can you use whatever there is on the diagram to use words to describe where you are?

A. I will do my best. So if you look at the plan view which is now expanded there you will see on the perimeter, so gridlines A and H, 1 and 8 there are a number of small columns around the perimeter of the building. And now if we can go to the cross section which is towards the right-hand side of the page in the bottom right-hand corner there you can see the detail in fact where the exterior columns come down on the left-hand side of that cross section but then the vertical load has to transfer across the cantilever beam right at the bottom and into the column which is spaced in a wee way from the edge of the building.

CROSS-EXAMINATION: MR ELLIOTT

Q. That's what you're referring to in the letter?

A. The potential weakness there is with the beam which is transferring load and shear from the base of the columns above to the top of the column below and so that will have been designed for a certain load based on the design as the designer saw it back in 1963. I'm sure the gravity load at that time was correct but obviously if the gravity load was increased if the building was extended then that load would increase to the extent that that could become dangerous so it's a critical weakness in the sense of wanting to extend the building which was being discussed at the time.

1120

Q. Just so that the lay person might understand that can I ask for the photograph of the building, BUIC 233.0159.1. Can you indicate on that photograph where you mean?

A. Yes, you can see with the mouse here if you look at the ground floor level columns are all set back from the face of the building whereas at the levels above the columns are on the perimeter of the building and so it is the step, what we call the transfer if you like of the vertical load from the perimeter face back to these columns which are recessed back away from the facade.

Q. So, and your evidence is that although you say this is a severe weakness seismically as this discontinuity has the potential for severe failure you did not mean so much this is a severe weakness as this would be a severe weakness –

A. If you like the whole of that report was written in the context of looking to extend the building options for redevelopment.

Q. On the next page of that document which will come up in a moment you also say that, "Although we have not conducted any investigations it is likely that the concrete may have deteriorated to the extent that these elements will require extensive repair in the short to medium term," and you are talking there about the exterior columns?

A. Yes.

Q. Is that not something which appears to have been an existing potential problem with the building at that time?

A. Well clearly it was because we did some work on that only a short time later when there was in fact some damage and we did a review at that stage and repaired a number of beams and columns.

Q. So did that work that you have just mentioned address this potential deterioration?

A. Yes. Well it was, it was always going to be an ongoing maintenance requirement but not one that had a significant impact on the strength of the building but one which would have an impact on the integrity of those columns.

Q. Wasn't your work around the issue of the columns done as a result of PGC bringing it to your attention in 2009?

A. Yes certainly it was.

Q. Can you just explain why this potential issue of deterioration was not brought to PGC's attention in 2007?

A. Well the potential for deterioration I cannot say whether they had this report or not. This is a report prepared for the architect but had they seen it and read it then they would have seen that. However, clearly they brought it to our attention some time later. I believe that repairs had been done on those columns at an earlier stage as well although not with our involvement so it is simply a part of the continued maintenance of the building requirement to be looking at that and making sure you had it covered.

Q. This is a more general question and again as a learning point. Where an engineer identifies a potential seismic weakness in a building, what obligations does that engineer have to inform the owner of such weaknesses?

A. Well I cannot think of too many instances where an engineer would identify a weakness without dealing with the owner. Certainly if such a thing was identified yes the engineer is obliged to ethically to do so.

Q. What if the owner then decides to do nothing about it?

A. Then we go down the inevitable dilemma of as to whether that is a severe situation which needs to be, need to inform a local territorial authority or whether it is not that severe.

Q. Is that what an engineer would do? Inform the local authority if there was a severe weakness that the owner had not acted upon?
A. If there was a severe issue which was not being attended to that is one of the courses of action which may be taken.
Q. Just turning to my third point, which is the inspection of earthquake damaged buildings and talking obviously in particular about the PGC inspections post-September. The first point is that in your brief, paragraph 34, you make the point that the inspections were not indicative of a building under distress or having a significant impaired resistance to earthquake shaking. Just want to ask you some questions about that issue of earthquake shaking, arising from Mr Mills has really asked about whether you are looking forward or looking back or both. When these inspections post-September were carried out by Holmes Consulting it was evident there was an ongoing aftershock sequence, was it not?
A. Yes.
Q. And it had been publicised that there was still a possibility of a magnitude 6 plus aftershock?
A. Yes.
Q. Do you agree that it is not really the magnitude of an earthquake that an engineer is interested in but the ground accelerations generated by an earthquake?
A. Certainly do.
Q. And ground accelerations can vary depending on the location and depth of the earthquake?
A. Yes.
Q. The aftershock zone included Christchurch city. Certainly that was obvious by Boxing Day, do you agree?
A. Yes.
Q. Are you aware that GNS could have calculated ground accelerations within Christchurch of different scenarios of earthquake within the aftershock zone?
A. Within reason, yes.

Q. Would you agree that would have been invaluable to engineers doing inspections because it would have provided a standard of earthquake shaking against which this impaired resistance you speak of could have been measured?

A. No. The, I think the difficulty with that is that would require re-evaluation if you like of every building in the city to a level which was not going to be practically achieved in a timeframe and therefore the reasoning behind what was elected to have been done, which is to say looking at the damage, was to look at which buildings had been impaired and therefore were clearly deteriorating and which had not and therefore we continue to have the same level of strength they always did. It is important to note that you know there is always the possibility of a much larger earthquake which can damage buildings and as you pointed out earlier even ones which have been designed to current code standards.

Q. Mr Boys or Mr Whiteside said that Holmes Consulting did not take any advice from GNS about aftershock risk or anything else, is that right?

A. We did not take specific advice. Obviously GNS were informing the wider community or wider engineering community to the extent that they could. We were party to all the briefings that the City Council and the structural group meetings wherever else it may have been discussed so there was a fairly consistent view across the engineering community at least as to how they would proceed.

Q. Do you think that in future the engineering community really should obtain information from GNS about potential horizontal accelerations in the aftershock zone so as to carry out assessments of damaged buildings?

A. I think that is a point for wider discussion before we go forward and it would certainly have a lot of that. My own opinion is that you still have to view it in the context of what the buildings have been through.

Q. So we are really arriving back at that point that Mr Whiteside and Boys arrived at, of the distinction between building capacity on the one hand and diminished capacity on the other? Is that right?

A. (no audible answer 11:29:06)

Q. And the question that I asked of Mr Whiteside was in determining diminished performance you are not looking at whether or not a building is structurally good or bad you are just saying it is less good or bad than it was before?

A. In the general sense yes but obviously if a building is a lot worse than it was before then it is falling below the standard it needed to achieve even to resist that event whereas if a building has come through an event such as it had with no damage then it certainly got at least that much capacity so if you like there is some aspect of judgement in that.

1130

Q. This distinction between diminished capacity and actual capacity and the answers that Mr Whiteside and Mr Boys gave may well have really broken the hearts of bereaved families and those injured, because the people who used buildings may well have thought that engineers who are coming along to look at the buildings were actually looking at whether the building was structurally good or bad, you see, and they may well have believed it was structurally sound when told it was safe to occupy. Now that appears to be a problem. Do you agree?

A. I certainly understand what you're saying and I think it's important to note, you know, our hearts go out to the families who have been affected by this. It's a dreadful circumstances that we've all had to go through. I'm still not sure that we can – where we go with that.

MR ELLIOTT ADDRESSES THE COMMISSION

CROSS-EXAMINATION CONTINUES: MR ELLIOTT

Q. Well let's look at what we might do about that, because I was going to ask you about how we might deal with this problem in future, but then I discovered that perhaps you were already working to some extent on that.

A. Yes.

Q. So you're on the DBH Engineering Advisory group?

A. Correct.

Q. And the group has prepared a draft guidance document, is that right, on evaluation and procedure?
A. Yes it has.
Q. And that's document ENG.AEG.0001 which will come up hopefully on your screens. That the document there?
A. Yes.
Q. And if we turn to 005, which is the introduction section of that document, just going to read out a section which is in the middle paragraph, "Initial and rapid assessments for buildings are a basic sifting method for identifying the worst of the immediate hazards, but the fact that a building may have a green placard does not mean that it has behaved satisfactorily and nor does it mean that it will behave satisfactorily in a future event. It simply identifies that no significant damage has been identified, that is, it is not known to be unsuitable for occupation. This means it's important for the engineering community to reinforce the message that further evaluation is generally needed, even where a building has been green placarded." So I take it you agree with those sentiments?
A. Totally, yes.
Q. So would you agree then that rather than saying in the case of the PGC building, that it was okay to occupy or safe to occupy, what Mr Whiteside really should have said was something like the wording used here, "No significant damage has been identified and it is not known to be unsuitable for occupation"?
A. I would agree that with the benefit of hindsight that could be better wording although I note that the reports that he's preparing were simple, a brief report of what he observed when he was there and the end conclusion.
Q. I appreciate that, I'm just talking about that wording?
A. I understand.
Q. Could we also not have added or should these words not also have been added, "That the building's capacity to withstand an earthquake

can only be identified by carrying out a full detailed evaluation which I have not done"?

A. The building capacity, yes, I quite agree, that has – requires this level of assessment to be able to determine it.

Q. And I'm just asking you if you agree that it really would have been better if those words had also been added to Mr Whiteside's and Mr Boys' report?

A. It would be better if those had been added to the reports all across the city I'm sure, but it does say that on the green placard.

Q. I mentioned your presidency of the Structural Engineering Society and that society has produced a report to the Royal Commission. Is that right?

A. Yes it has.

Q. And one of the, am I right in saying that one of the recommendations that is made there is that there should be a time limit for the detailed engineering evaluation of buildings once the state of emergency's lifted?

A. Yes. Correct.

Q. So is it inherent there that really there should be a detailed evaluation of all buildings?

A. Oh, absolutely.

Q. And would you agree that in the case of buildings built before 1976 really those buildings should be closed before such a detailed evaluation is carried out so as to be sure that they are safe to reoccupy?

A. On that point I'm not sure I would agree. I think it depends, you know, there are buildings built before that date which are demonstrably quite safe; there are others which are less so, and so I think there is still an element of judgement required which has been certainly the way that the profession's been working through this problem obviously since September the 4th. There needs to be a lot more consideration of that but the timeframe over which that would occur is going to be significantly longer than – it's – we're talking years not months.

Q. Are there any other learning points that you think can be drawn from what happened with the PGC building?

A. Well I think in the fullness of time there are probably going to be many, and I only hope we learn them all. More specifically I'm rather hoping the Commission will tell us that.

Q. I'm just giving you the chance to assist the Commission further if you can, but you have already contributed by way of reports and so on (overtalking 11:35:46).

A. Thank you. I mean I think the – it's in the detail evaluation guidelines which you already have there. Certainly this building is one of many that have helped to inform that process.

COMMISSION ADJOURNS: 11.36 AM

COMMISSION RESUMES: 11.54 AM

CROSS-EXAMINATION: MR HERON – NIL

CROSS-EXAMINATION: MR LAING

Q. One question, could you turn to paragraph 31 of your evidence please.

WITNESS REFERRED TO BRIEF OF EVIDENCE

Q. Could you have a look at the first sentence there where you refer to the fact that you took into account the verbal briefings given by the Council/Civil Defence. Are you referring to briefings during the state of emergency?

A. Yes I am.

Q. Do you recall any subsequent briefings?

A. Um, I couldn't tell you exactly when the, the briefings, um, came to a close. It was, um, there were a number of briefings through that period initially following the earthquake. It certainly was all in the declaration.

Q. Do I, do I assume that you took place, or took part in some of those briefings?

A. Yes.

Q. Do you recall what was said about aftershocks?

A. Um, certainly to expect them, um, and sort of, I, I couldn't tell you exactly when it was said but certainly the, um, the comments at the time were around aftershocks potentially in order of magnitude less than the original.
Q. So that was just to clarify that, that was part of the actual briefing given to you and other engineers who were present?
A. I believe so yes.

CROSS-EXAMINATION: MRS COWAN

Q. Mr Hare I just have a couple of questions for you. If I can just take you to the phone contact that you had with Harcourts on the 5th of September which is BUI.CAN233.0054.18?

WITNESS REFERRED TO TELEPHONE CONTACT DOCUMENT

Q. Okay and you'll see there it says, "The scope is the initial earthquake inspection and securing measures as considered necessary". I understand that you didn't actually sign that contract on behalf of Holmes Richard Seville did but did you ever discuss the nature of the scope and the services with Harcourts or was it more just to formalise the engagement of Holmes?
A. Um, I would have had general discussion with Howard Buchanan when he called me, um, apparently on the 4th I think, either the 4th or the 5th before this anyway, um, and so we had at that stage I would have said we had a general understanding of what might have been required but obviously that was further, um, discussed with Richard.
Q. Okay and when Holmes undertook the second, third and fourth assessments this wouldn't have been constrained to assist initial assessments as is described in this contract, would've it?
A. Um, well we were still working under the same contract, um, and so any, the need for further assessment would have been determined by what had changed in the mean time if anything.
Q. But where it says "Scope" which is initial earthquake inspection, this is obviously gone on further when you've undertake the subsequent assessment so it wouldn't have been limited to initial assessments?

A. Um, I understand but it depends on what the triggering, um, point was for the, for the init- for the extra assessment, um, in some cases it was aftershock.
Q. Okay. Was there ever any discussion that you can recall with Harcourts about having an amended contract?
A. Um, no nothing that I can recall.
Q. Okay. And you spoke earlier in some of your evidence is that the level 2 assessment, there were level 2 assessment guidelines and these were the earthquake engineering guidelines is that correct?
A. Ah, yes, correct.
Q. Were these level 2 assessment guidelines ever specifically conveyed to Harcourts?
A. I'm sorry I don't know.
Q. As I understand it from your evidence the level 2 assessment was able to determine if the PGC building was structurally safe to occupy. Is that correct?
A. In general terms yes.
Q. And it would, this was the case for the PGC building based on the Holmes assessments that it was structurally safe to occupy?
A. Um, the conclusion was that given the lack of apparent damage it was acceptable for occupancy yes.
Q. Okay and in your evidence given today you've said that subject to, sorry at paragraph 32 of your brief, of your declaration you say that subject to recommendations for further detailed assessment and it would have been Holmes that would have recommended to Harcourts or to tenants any further detailed assessments if they were required?
A. Yes the circumstances of that would have been had we seen, um, damage which was giving us cause for concern or, um, ultimately, um, when repairs were commenced for the building at which point a detailed evaluation would have had to have taken place.
Q. And during those inspections or any further repair there was no requirement for further detailed assessments was there?
A. Sorry can you repeat the question?

Q. So during the course of the assessments or any repairs undertaken there was no need, or Holmes didn't determine there was any need for any further detailed assessments?
A. Um, I'm not aware of any repairs being undertaken and so certainly none of the assessments indicated the need for detailed, immediate detailed assessment.
Q. You would have been aware that the building or the label being, of the PGC building 'safe to occupy structurally' was a message that would have been conveyed to tenants in some form?
A. Um, I'm not sure if they were getting copies of the reports, the reports were being prepared for the owners, um, but obviously during the course of the inspections, um, the engineers would have been walking around the building with various of the tenants and, um, there would have been verbal discussions about the occupancy.
Q. I understand that you might not know how the, whether the reports would be conveyed or how they would get to the tenants but the message being, in the report that they were structurally safe to occupy in some manner that would be conveyed to the tenants?
A. I understand that.

RE-EXAMINATION: MR HANNAN

Q. Just one simple matter. You'll recall that Mr Elliott asked you about discussions with the owner about life safety with respect to the work being done in 2007, the report done in 2007?
A. Yes.
Q. And he asked you whether there was scope for a bit more discussion with the owner about life safety?
A. Yes.
Q. Now who were you in fact dealing with with respect to the report that you did in 2007?
A. Oh, we were dealing, um, directly with the architect at that point.

Q. And can you comment on what you would expect as yourself an engineering building professional architects to understand when language like that was used?
A. Well clearly we were, um, speaking as fellow building professionals, um, and so I would expect him to have a general understanding of the terminology we were using and/or ask for clarification.
Q. And as far as you're aware in this particular engagement in 2007 it was Warren and Mahoney who was interfacing with the owner of the building?
A. Yes.

COMMISSIONER FENWICK - NIL

COMMISSIONER CARTER – NIL

JUSTICE COOPER – NIL

WITNESS EXCUSED

JUSTICE COOPER:

Yes Mr Mills.

MR MILLS:

Well Sir as you know we're now going to hear from two representatives of Becas on the expert report but just before they're called to give evidence I thought that because none of the overheads or none of the power points that as I understand it Becas are proposing to use include the terms of reference that they were operating under nor the terms of reference for the panel report, I thought it would be useful just to put those up so that people can see that and know what the background is to the reports that have been done. So on that basis could we first have BUI.CAM233.0051.10 and if that can be enlarged at all that would no doubt help.

COMMISSIONERS REFERRED TO REPORTS

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Now this is actually taken from the Beca report but, so anyone who wants to see it in its context of course can go to that report which is on the website but these are the terms of reference that were given by the Department of Building and Housing to the consultants that were engaged to do the four buildings that the DBH enquiry enquired into. It's not specific only to PGC. PGC is simply one of them and I simply draw attention to the fact that this is what Becas and other consultancy firms were asked to do when they were engaged to look at these individual buildings. So they were required to look at the original design and construction, impact of any alterations, how the buildings performed in the 4 September earthquake – in particular the impact of the earthquake on *the* building. Then they were required to look at the assessment process and the ultimate question, as has become very clear by now from what has been said, why these buildings collapsed or suffered serious damage. And then there's a wider range of issues that the consultants were also required to look at - design codes and so on, knowledge of seismic hazards, changes over time to knowledge, any policies or requirements to up-grade the structural performance of the buildings. Then a description of what the consultancy firms were to make use of, records, interviews and so on and

then matters outside the scope of the investigation which I draw some attention to which is that issues of culpability or liability arising from the collapse of the building were outside the scope of the investigation. And then Becas have themselves set out in relation to the PGC building specifically the matters that they had had regard to in doing the report and it was interviews by witnesses to the collapse and the rescue activities following, structural analyses, materials testing, geotechnical investigations and site surveys. So that's the background as set out by the Department of Building and Housing for the report we're about to hear evidence on from Becas.
Then I just want to draw attention to the terms of reference of the expert panel report and that's at BUI.BAR.0017.12 and it's also on page 13. Now if you can just enlarge the part under "Terms of Reference" which is the only part that we're interested in and it does actually continue on to the next page but again just looking at this initially the structure as the Commission well knows was that along with the engagement of expert consultants to prepared the reports, an expert panel was set up under these terms of reference that are set out here and the essential function of the expert panel as I understand it from what's said here is that, as it says under "Outline Approach and Outputs", the consultant technical investigation report was output number 1. Output number 2 was a report prepared by the expert panel to the Department of Building and Housing, and then output 3 which is yet to happen is a report from the department to the relevant Minister on the outcome of the investigation.

JUSTICE COOPER:
I don't think it is correct to say it is yet to happen. I think it has happened and the way things ended up there was a very brief report from the Minister which I think has certainly been provided to us. I've seen it.

MR MILLS:
Thank you Your Honour. I stand corrected.

JUSTICE COOPER:

It's simply adopting the recommendations of the other reports. It is very very brief so you may be correct inasmuch as there may be another report. Certainly the same process has to be repeated with respect to the CTV building and, who knows, there may be another report. This might be called a final final report I suppose but certainly that three-part process mentioned there has supposedly been completed with respect to the three buildings in respect of which there has been an expert panel report.

MR MILLS:

Thank you Your Honour. Yes I suppose had possibly rather optimistically anticipated that when all of the reports were in there might be some overall conclusions that might be drawn but perhaps I've read too much into what's intended.

JUSTICE COOPER:

You may be right but there's certainly been a report from the Minister bringing things up-to-date.

MR MILLS:

So that's the first page of the terms of reference and under "Roles and Responsibilities" I won't go back to that. Then over on the next page which has just come up on the particular roles and responsibilities of the expert panel I do draw attention particularly to the second to last of those bullet points which is that one of the roles is to review and approve the engineering consultant reports and I just do draw to the Commission's attention that language review and approve which I find interesting language really for the expert panel function but that's what they're required to do, and then finally they are to produce an overview report addressing the matters for investigation and so on. So that's the background to the evidence that we're now about to hear from Becas and on that basis I call Mr Jury and Dr Sharpe to come and present the consultants' report.

MR MILLS CALLS
ROBERT DAVID JURY (AFFIRMED)
RICHARD SHARPE (AFFIRMED)

Q. Now I will just take you through some of the formalities that you'll be familiar with. One at a time, your full name, dealing firstly with Mr Jury, your full name is Robert David Jury?
A. It is.
Q. You are a resident of Wellington?
A. I am.
Q. You have a Masters in Civil Engineering from the University of Canterbury?
A. I have.
Q. You're a Technical Director in Beca's Wellington office?
A. I am.
Q. You are a Chartered Professional Engineer?
A. Yes.
Q. You're a Fellow of both the Institution of Professional Engineers and the New Zealand Society for Earthquake Engineering?
A. I am.
Q. You were a member of the committee that developed New Zealand's current earthquake loading standards?
A. I was, yep.
Q. You're the Chairman, I think currently the Chairman, of the New Zealand Society of Earthquake Engineering Earthquake Risk Buildings Study Group?
A. Ah, that group is currently in abeyance but about to be reactivated, so yes.
Q. With you still as the Chairman?
A. Yes.

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Q. And as I understand it that study group produced the current guidelines for the assessment and improvement of the structural performance of buildings and earthquakes?
A. Yes.
Q. Among your many roles and significant structures you've been involved in, you led the structural design team for the Auckland Sky Tower?
A. I did.
Q. Now I just say beyond that, that your CV is on our website. It's much more comprehensive than that. Those who are interested can find it there. Now I then just want to confirm that you were retained by the Department of Building and Housing to prepare a report on the PGC building?
A. That is correct.
Q. And again just for the record I want you to confirm the report and I'll just bring it up so that you can go through the technical step of confirming it. It's BUI.CAM233005.1. At least I hope it is.
A. That is correct.
Q. Thank you. So you confirm that's the report –
A. Mmm.
Q. - that you were the principal author I think is probably the correct description is it?
A. That's correct.
Q. And can you also confirm that you were a member of the expert panel that I've just been describing?
A. I'm sorry that panel you –
Q. This is the DBH expert panel.
A. The one that's still sitting?
Q. Yes, you are a member of that?
A. That's correct, that's correct.

EXAMINATION: MR MILLS
RICHARD DEAN SHARPE

Q. All right, then turning then to you Dr Sharpe. Your full name is Richard Dean Sharpe?
A. It is.
Q. You too are a resident of Wellington?
A. I am.
Q. You have a PhD in Civil Engineering from the University of Canterbury?
A. I do.
Q. You're a Chartered Professional Engineer?
A. I am.
Q. You are the technical director of the Earthquake Engineering Office of BECA in their Wellington office?
A. That's correct.
Q. You have more than 30 years of experience as a structural engineer?
A. I do.
Q. You're a past president and fellow of the New Zealand Society for Earthquake Engineering?
A. I am.
Q. In 2007 you were made a Distinguished Fellow of the Institution of Professional Engineers New Zealand?
A. I was.
Q. And that was in recognition of your earthquake engineering contributions?
A. It was.
Q. Now once again you have a CV that goes well beyond those matters but those, again your CV has been put onto our website for wider distribution. Perhaps the one other thing I just note from the CV I've got in front of me that I might just ask you to confirm is in 1999 you led a group of New Zealand engineers and others to Turkey to examine the devastating effects of an earthquake that had just occurred there?
A. I did.
Q. And since then you've been involved in a review of the Romanian earthquake building code?
A. We both have.

Q. You both have?
A. Mhm.
Q. And in 2010 you were involved in a review of the seismic resilience of the energy sector in Rumania?
A. I was yes.
Q. Now can I just ask you, because I'm not quite sure myself, precisely what your formal role was in the preparation of the BECA report?
A. I concentrated on collecting the evidence and, in great extent and putting together the first part of the report. While my colleague Mr Jury undertook the intensely technical nature of the –
Q. Thank you. And can I also ask you whether you had any formal role in the expert panel?
A. Not any formal role but I did attend that panel on a number of occasions.
Q. And would that in relation to the PGC building have been for the purpose of discussing the issues that were arising in the course of the panel's evaluation of the BECA report?
A. That's correct.
Q. Right, well – and again I don't think I need to take you formally back to it, you saw what was up there before. That is the report that you were involved in that we had on the screen a moment ago? Just answer that confirming –
A. Yes that's it, sorry, yes.
Q. These are just matters for the record.
A. Yes.
Q. All right, well now as I understand it you're going to deal with this by reference to some power points, so I'm going to sit down and leave you two to run that in whatever way you choose to.

DR SHARPE PRESENTS POWER POINT PRSENTATION

A. Thank you. Commissioners at the request of the counsel consisting the Commission, this initial presentation that I'm going to give is a repeat of that that we made initially on the 30th of September to the bereaved families of the victims and then later to some of the tenants, and then who were present in the building at the time of the collapse and then was given in the same form to the media so this is not new information for the public.

I think just at the beginning it's worth pointing out that in the past week we've had a lot of feedback from the reports of the Commission that we may have in fact been involved in giving a report on the Pyne Gould Corporation building between September and February, and that's not true. We pay tribute to our teams. It's not just Mr Jury and I who've done this work, but we've had the great support from our team, particularly the analysts, Francis Tse and Kate Grinlinton in doing this work. I'm hoping to be able to turn the power points over myself with the mouse. You might so wonder, it's come up, I was going to say you might wonder why engineering consultants have investigated this and perhaps not university based engineers. Practising engineers in New Zealand have had a long history of driving the development of earthquake engineering for seismic resilience and that might come up later on.

So just to start off with, with the picture of the building taken about a year I believe it collapsed, about 45 years old when it collapsed, modern looking but not modern in terms of its structure with respect to our design practices today. This presentation doesn't want to move on. So this presentation covers the design and construction of the building, the reason for collapse, the history of the building and only then will we tell you about our investigations, the conclusions and the recommendations that we've made in our report.

The design and construction. We found that the building appears to have been designed in accordance with the standards of the time, 1963. It's worth pointing out that in, even when I started my final year at university as an undergraduate in 1968, earthquake engineering was an optional subject to be taken in the final semester and I think that's important to understand that in the context of the design, so although there was a lot of interest in earthquake engineering, in terms of what we now know, the middle 1960's, it was a very basic approach that was taken to design. As far as we can tell the construction appears to have been in accordance with the design.

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JUSTICE COOPER:

Q. Dr Sharpe, did you see a building permit at any stage? Were you able to find one?

A. I do not recall myself having seen that. It was variously reported last week by some people that they thought the Ministry of Works had built this building. It was in fact built by a local construction firm, not by the Ministry of Works. So the reason for the collapse, the shaking on the 22nd of February was several times larger than the building was designed for and it is clear that the east wall in the core failed just above level 1, the columns could not sustain the resulting horizontal displacement of those core walls going over and the connections between the floors and the walls were unable to sustain those forces and displacements and so they failed too resulting in a catastrophic collapse.

I have put up here everything that has been put up on this display is actually taken directly from our report and for many people they have had to learn about some of these graphs which we use on a day-to-day basis to describe earthquakes and to describe the design parameters from a building and this is a little bit hard if you are not used to it but this first one here is what is called a response spectrum and I would like to use a little model to show you how to interpret this. This is a little model of three lollipops and they are on little flat pieces of spring steel and they are different heights and I can put different weights on the top and these represent perhaps three different types of building and so that if you shake the ground you can see that, depending on the way I shake it and the direction I shake, different buildings with different characteristics will shake in different amounts. Each building has what is effectively a fundamental natural period in each direction, that is that if you could pull it back and let it go backwards and forwards it would vibrate in that natural mode. And so across the bottom of this graph we have the natural periods from 0 which is a very stiff building across the very flexible building 5 seconds so that means it would go backwards and forwards one cycle every five seconds. Now earthquakes come with built-in frequency contents. Different earthquakes have different directions, predominant direction, they have different frequent content. Unfortunately most earthquakes have a frequency content which tends to particularly resonate the natural period of the structure which is close to what many middle size buildings inherently have, and so looking at this graph and I use my mouse. This up the left-hand side is the force that each of, a lollypop of each of different natural period might feel in a particular earthquake. So in this case four of the records from around the CBD have been processed with lots of little lollypops to see what the maximum force they would receive from those four earthquakes of the 22nd of February and they have been plotted on here but because they are all different a grey band has been put over them to show the general nature of them, and what we have put in the different colours under there are graphs which represent our approximation of general earthquakes that we use for calculating the design of a building. So that a building that has a natural, a first natural period of half a second is going to have the highest forces generally designed forces on it. If you have a building that is quite flexible, say out towards three or three and a half seconds you can see that in most earthquakes it will not get that sort of building going and that is why in a case like the Christchurch Womens' Hospital which sits on rubber pads which are very flexible, it gets away with, in most earthquakes, of having much lower excitation because earthquakes naturally do not generally excite those ones. So going from the bottom one here, this green line represents the design level for a building designed to the 1965 code which is considered to be elastic and I will explain a little bit about that later on. The other one there is of the next one up, the green line, is a full ductivity is achieved but you can see here that in 1984 code is this red line here and this one up here is what you would expect to be exceeded about once every 2500 years on average over a very long period of time. This spectrum does not tell you the whole story because it does not tell you how long the earthquake has gone on for. It only talks about the maximum feeling that the building has received in an earthquake and that how long is very important as to how long a building can survive an earthquake.

We have other ways of showing this also where we instead of, we have plotted here the displacement of my lollypops. The maximum displacement that might be felt in an earthquake plotted against the maximum force and this is quite useful for us, not so useful to explain to the public, but useful for engineers to show that for buildings of different natural periods and they are represented by radial lines coming out from the origin down here so that the ones up here are for stiff buildings and the ones coming out here are for more flexible buildings. We have plotted over them the same earthquakes and then down here at the bottom we have plotted the code, equivalent code approximately that was in control of our design in 1963. The 1935 building code produced after the Napier earthquake was the, in fact the code that was in force at that time for working out earthquake design levels and that was slightly changed in the code that was produced in 1965 and of course it is knowing the calibre of the engineer who designed the building and it is quite possible he was already had in mind the draft that normally would have come out before ’65 but we cannot confirm that.

In contrast with the one on the 22nd of February we have here the equivalent matter for the 4th of September and of course it is a bit hard for you to compare the two here between that one and the previous one but of course the shaking generally in the CBD was much less in 4th of September than it was on 22nd of February.

Everybody talks about the shaking the buildings got in the CBD as if they actually knew what any individual building did get shaken at, what it received. In fact we do not actually know that very well at all.

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Here is the PGC site here and then the four nearest instruments are the red dots around there, you can see one was in the Botanical Gardens, one at the hospital, one down at the Cathedral College and then one up here just near Bealey Avenue. That's about 670 metres away from the PGC and you probably have picked up now that the rivers had run their courses all over this area and it's sort of a bit like a marbled cake underneath there. And so each building sits on its own little bowl of jelly if you like, but may have quite different characteristics just down the street from one place to another. And so we had to of course in looking at the PGC make a decision. Do we take the average of all the records round here? Do we take the closest one? Do we try to find a record, which record was on similar ground to where the PGC was? Now the reason that we believed that the building failed was because of the east wall of the shear core was approximately in the centre of the building, and the best that we can tell is that a compression or buckling failure in the east wall, the New Brighton side of that wall, occurred immediately above level one.

Here’s a plan view, a view looking down on the building but looking at just above the first floor level. Cambridge Terrace and the river is down at the bottom, the river is orientated at I think almost exactly north south and east west, and you can see this is the wall which we believed failed. Except for a column here and a column here, the floor system was held up by the external columns going from floor to floor and held up by its connection with the walls here.

Now if you look at the building there, you’ll notice this is taken just within a few minutes of the collapse by in fact one of our staff, because our building, we had three floors, our Christchurch office has three floors in the Price Waterhouse Cooper tower which is immediately across the river and this was taken by a member of our staff. You will notice it's very clear that the ground floor was almost intact. So why was that like that.

Very simplistically this is at the ground level of the building and you can see that there are more walls between the ground level and the first floor level and so it was stronger for horizontal forces because earthquakes predominantly produce horizontal forces on the building, and you can see that those walls were generally eight inches, 203 millimetres thick, reinforced concrete. The slab is about five inches thick and on the ground here. The building was – is not on piles, it's actually on pads and those pads are actually embedded in the ground to about to the ground level, the building is up on a little bit of a rise at the ground floor, so the reason that the floors collapsed is that the columns and the joints in the perimeter frame were unable to take those very big horizontal displacements that occurred as the core wall fell over.

Here's a section taken through one side of the building on the left here, that's as if you cut it down through the building with a knife and looked in sideways and you can see down at the bottom here, you can see the pads. This is the ground level here. There were steel encased columns coming up to this level and a very large steel beam ran across the top of those and it cantilevered out to pick up these columns were up here, and as you've heard a number of times in the retrofits that went on in about 1998, there were hollow steel posts of about that sort of size in cross section.

Q. Give us words for that will you please?

A. That's a 200 millimetres by 100 millimetres rolled hollow sections were installed behind the reinforced concrete ones that were there.

There you can see the catastrophic failure of those perimeter columns. You can see that they have failed predominantly at the tops and bottom of those, and the joints. So that's consequent to the wall falling over and you can see again here how the ground to first floor level has almost – has stayed completely intact. So many of these columns running between the floors around the outer edge are effectively intact over their length.

Now here is a closer up one where you can see that a joint which has partially survived between a column at the top coming down into the joints where it goes, for the beam that goes around the edge of the building has completely failed. It is not providing any resistance above, the beam is not connected strongly to the column above or below or vice versa.

Here's another example. It's worth saying that this is now very different to how we design buildings. In the late 60's and the 70's and as early as the code in 1965, we introduced the concept of ductility into design and the easiest way of describing ductility is to look at a piece of ductile wire, this comes from a coat hanger. You do have to be careful sometimes, because some coat hangers are made of quite brittle steel as is sometimes steel used in construction, but you know very well generally with a piece of copper or a piece of good mild steel you can bend it backwards and forwards and deform it a number of times before it will actually break. So what's happened in the late 60's and through the 70's is that this was recognised as a way of providing extra resilience in a building. If we could build this characteristic into our reinforced concrete members and in fact into our steel framed buildings as well. And it's very obvious from these photographs that none of that was available there, and the way that that is achieved is the ductility of a reinforced concrete member is achieved from a brittle substance like concrete by putting steel into it, and then binding a wrapping steel around it to confine the concrete so that even if the forces in the concrete are exceeded to the point that it breaks up into pieces, it will graunch away backwards and forwards for quite a long time before it in fact finally gives up.

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The detailing that was common of buildings of this time did not have that facility to do that. So that's the difference between the building of that era and the building now.

And of course once the, once the core started to move over to that extent the, the connections between the floors into the walls were never designed to take that amount of wrenching and they failed. They just broke.

And here is a photograph taken along the centre there you can see the edge of the core wall where the, and, and it's, you're able to see there little black dots which are in fact the very small amount of steel which was coming out of the wall into the floors to hold them together. In the, in the last 40 years a lot of work has gone into making those sorts of connections much greater as we now understand the forces that go on there.

Here's another shot of the same thing and these black marks across the centre of the photograph are simply the, the fractured steel where it was never designed to take those forces associated with that displacement.

You can see here that the, the core really stayed extremely intact above there and on the east side which is from the right to the left here you can see that because the building was pushing against the walls we didn't quite get so much fracture there except for the top, the roof which actually fell off and slid down over the other ones.

So our investigations. It's worth remembering that we did not begin to investigate this building until early April at which stage at the site all that was left was the ground floor, over half its extent and we were able to go in and just look briefly at the, at the structure while we got them to stop the demolition and it was amazing that in those bottom columns there was, it was hard to see a crack at the top of them where they went into those massive beams that cantilevered out. We believe that all, we had all the documents that were available. We, we have the property file from the Christchurch City Council which is about that high. We have a full set of what, what we believe's a full set of about 20 structural engineering drawings describing the whole structure. We have a lot of photographs taken particularly by the urban search and rescue people who were able to take some of those before the demolition and the removal of parts of it to, to get to the victims was undertaken and we have a lot of witness statements including some of our own colleagues who saw it across there. It's very interesting from a structural engineer's point of view that almost everybody described something different. We were particularly interested to know in which, approximately when did the building collapse after the start of the earthquake and some of those witness statements said that it happened after the earthquake had stopped but I think the consensus was that it happened quite early in the piece. Material tests were undertaken by another consultant before we started our work before the, a lot of the stuff was taken out, a lot of the material was taken out to Burwood and placed under control out there for the Coroner's Court. And, and generally the tests of the concrete and the steel that came out of there showed that that was in accordance with what you would have expected for a building of that age and expected to have been related well to what we'd've expected to be used for the design of that building because we didn't actually have the specification saying what that should be. We found that we, as, as we realised that this core wall might have failed we, we didn't have any concrete tests for that but of course by that time it had been completely demolished. So we had an inspection made of the, of the rubble done at Burwood and it was, we managed to find about five or six pieces that were obviously of the wall because of the spacing of the steel within them, the size of the steel and the width of the concrete and we were able to have tests done of that concrete and that steel and there were no surprising results from that at all.

Of course we have all the earthquake records available to us as I was alluding to before and so we undertook a simulation of the building in earthquake, in the earthquake of 22nd of February by computer and that technique is known, is a simulation technique known as, inelastic time history analysis. I've been doing those for 40 years. They, it is not, it's not something particularly new but it requires a lot of judgment in choosing the parameters that you will put in to model your, model your structure.

So here's a history of the building. 1963, designed as offices for the Christchurch Drainage Board. Building consent in 1964 so I guess we've seen that. 1989 with the demise of the Christchurch Drainage Board it was taken over by the Christchurch City Council, and in the LIM of 1996 we saw some reference to the fact that it might be peat and wood under there. There's quite a big history of all the soil tests that have been taken in Christchurch over the last hundred and something years. They've all been recorded very well and they're in a big data base and again in 1997 there was a concern that there might be presence of peat underneath there, under this building, on this building site.

Q. That data base is that held by the City Council, is it?

A. Yes it is. I believe, I believe it was inherited from the Drainage Board and it's referred to as the "Orbit" O-R-B-I-T data base.

Q. I thought that was something that had been constructed by Tonkin and Taylor?

A. It may well have been but I'm not sure of the details of that, Your Honour. But, certainly we, those of us who were designing buildings here in the 70s well knew about the "Blue Book" I think it was called that the Drainage Board had of all its information because of course they, they made a lot of holes in the ground for drainage.

Q. Yes. So you think there's, there's comprehensive information in the hands of the Council about the soil conditions in the CBD. Is that what you're saying?

A. Ah, it, it, there is a lot of information but of course it's variable as to depth and how well it was logged at the time and what it's purposes were for but in fact there is, there is quite a bit of information around.

Q. Mhm.

A. And you've already heard about the 1997 report, structural report by the owners effectively the owners' engineer with respect to the loadings code for earthquakes at the time which was New Zealand Standard 4203, and you can see we've recorded here that as the proposed refurbishment did not constitute a change of use there was thus no legal requirement to strengthen the building seismically and as we've heard in, before in about 1998 those steel props were put in behind the columns around the outside. Also there were penetrations, extra doorways put in, in those walls and there were some holes that were filled up. 2006 major refurbishment, and in 2007 as you heard earlier this morning, there were some options looked at to strengthen the building with a, perhaps by tying it to an adjacent new structure and it was not proceeded with. In 2008 a mobile phone site was added and additional openings made in the, in the ground floor and of course at that time there was compensation put around those, reinforced concrete was put around those holes to compensate for them. 2008 it was sold to the owner at the time of the collapse and in 2009 additional mobile phone cabinets and panels added.

1254

We do not consider that the extra mass or weight of those was really important in terms of what we are looking at. 2009 as you have heard there were repairs made to cracks in the perimeter columns and that is, normally that sort of thing is because there has been a little bit of ingress of water through a crack and it has caused the rust to expand, the steel to rust a little bit and expand and that happens quite often particularly on panels on buildings and it is normally treated by digging out and injecting, painting the, priming the area with some sort of protection and re-mortaring the area that you have cut out. And then in September the 10th of last year the site report by the owner's engineer. January 2011 we have been, you have heard about those already.

Here are some models just of parts of the building which show where things have been taken out and areas filled in. I think I am right in saying the blue is where holes have been concreted up and have I got it right Rob, red has been taken away, so there were in the earlier ages, there were some decorative members around the edge of the building that were taken out for the style of refurbishment we would expect, and also many people in Christchurch will know that this building had some sort of concrete umbrellas on the top which were removed during the earlier refurbishment.

Now you will see a little bit of this later on when my colleague takes you through some of the model shots of this analytical model. The model is made up of mathematical elements that follow the structural form of the building and then we take one of these earthquake records that has come from GNS Science, in fact, from their instrument, and we mathematically apply that across the base and we get all the directions, both north/south/east/west and up/down/vertical at the same time and we actually simulate the building's response, but the main thing is that we build in this plastic or ductile behaviour at the places where we know it is going to happen so we model that and so we can actually see where stresses get so high that the concrete or steel is likely to fail and we will see some of more of that later on. The little red spots that you can see on that one are in fact during the snapshot taken during the analysis where we have got it displayed on our computer screen and we can see that there are areas of damage occurring at those points. We say rubbish in, rubbish out. The analysis results are only good as your modelling and it is not a precise science, it is an art to many degrees. And so here you can see I think this is buildings going the other way and you can see we are showing that around the edge of the bottom of the columns we can see that those columns have exceeded their capacity that they have been designed for.

It is worth pointing out that, here is another technical one, here we have a graph which really is like my piece of wire, in general terms elements, structural elements in a building follow a pretty predictable behaviour pattern when you take them up so up the left-hand side of this graph here you have force being applied to an element and across the bottom in general terms you have a displacemen. So as long as you keep your forces within the elastic part of the characteristics of your member it will return to the place it started in. If you take it over the top and I am talking about pushing it in one direction and that is taking it past this point where it is no longer elastic and becoming ductile you can see that as you push it, it will stay in position from then on and generally if it is reinforced concrete if you keep on pushing it and pushing it you will get to a point when it cannot take any more load and it will start to reduce its capability to take load, keep on pushing it and it starts going down and then it sort of sometimes reaches a bit of a stable area but if you keep on, keep on, keep on pushing it, it eventually fails, and you can apply that sort of thing to what went on with the wall of the structure. I think I neglected to say earlier that we carried out some other investigations into the appropriate nature of the record that we were using. We actually put down a test bore into the soil at that site where we were taking the record from up near Bealey Avenue to see what sort of ground it was sitting on and when they had cleared off the first level of the PGC site we went in there and took tests there and from what we could tell it was not the same but it was not too bad a match. The interesting thing was that the test bore at the site showed no sign of liquefaction having occurred underneath the pads of the building and so we were listening last week to evidence where a gentleman said that the street outside on Cambridge Terrace to the south was covered in the ejected material from liquefaction. There was liquefaction certainly in the general area. That may have well flowed down to there from somewhere else but from what we can determine there was no sign that this building's response in the earthquake was affected by any liquefaction beneath the structure or in fact any failure of the ground beneath the structure.

This one here is of the, these are traces taken from the instruments, the nearest instruments. This is the 4th of September one, and these are plots taken of the various components. You have got down the page there at the top. This is the 4th of September one taken up near Bealey Avenue. The top records here, this is the acceleration in the east/west direction. This is the acceleration along here with time and the north/south direction and this is the vertical one, and you can see that this is quite a long shake that is going on there. The lines underneath – this is velocity and this is displacement, and they are actually can be mathematically derived from this top one which represents acceleration. So you can see a long earthquake with quite a, we are interested in the length of this intense shaking here in terms of the building performance.

This is the Boxing Day one and I am just not sure of the reason but we did not have these easily available for the Bealey Avenue site. This is for the Cathedral College site some further distance away and it is not exactly the same scale but you can see just a couple of quick pulses in there, in the three directions.

Q. Well it is not plain to me how I know this is Boxing Day?

A. It has the date –

Q. You have told me it is but –

A. – just around there, you see it might be hard to read but it has actually got the time stamp up in the top line, second to top line, in the middle. And then there's the 22nd of February one.

1304

Q. Why does it say 21st of February then?

A. Because that's in universal time so it was in the northern hemis...... in the, ah, other part of the world. It was still the 21st, universal time. You certainly have to keep your wits about you when you're trying to locate the records.

Q. Obviously my problem but what's universal time?

A. That's what I think used to be called Greenwich Mean Time. You can see here that's it's a much shorter period of intensity. If you look at the displacement here you can actually see that there are some sort of quite violent movements, displacement which is something that you certainly, ah, you would feel that and that's interesting in terms of what we think happened to the building. So east-west in this top panel – consideration of velocity and displacement. This is the north-south and this is the vertical one. Now there was definitely a difference in directionality for the main shaking between the 4th September earthquake and the 22nd of February and you can actually show that with our analyses but it was also very obvious in the Price Waterhouse Cooper tower just across the road where, ah, the shaking and the damage, the cracking that occurred in the beams in that building were definitely in the frames that were running north-south in the 4th of September earthquake and then they didn't get much bigger in the 22nd of February but we got a lot of new ones in the east-west direction. So certainly the ways in which the buildings responded around that area we could see some evidence of this directionality, and that's in accordance with our knowledge of the sources of the earthquake.

So here are some sketches done by Mr Jury's fair hand here to try and show graphically what our analyses have effectively shown us. The beginning of the earthquake the building intact. At about is that 4.8 seconds the ground motion was to the east and so the building would have appeared to go, lurch towards the west and then within a second it's reversed the other way and the building has flicked off towards New Brighton towards the east and kept on going and that is consistent with some of the witnesses' statements including the people who observed it from the Ernst Young building immediately to the west of it who reported that they saw the building shudder, come towards them and then sag away from them. And you can see there that consequent to the shear core failing the other elements gave way and the pancaking of the building occurred.

So to our conclusions. Because of the way it went over one way and stretched the east wall first a possible fracture of the tension reinforcement in the shear core is not likely to have been the significant factor and may have delayed the collapse. It would have allowed the wall to rock because sometimes we actually do make structures rock a little bit to make them survive an earthquake and an example of that is the South Rangitikei Railway Bridge which rocks like that deliberately. We've concluded the perimeter frame was unable to sustain the imposed building deformations up to the point that the compression failure occurred in the shear core at level 1.

Q. Should that be "unable" then? You read it as "unable".

A. I'm sorry. I'm back one now. I'll go back to that one. It was unable to take those deformations certainly.

Photographs from the engineers' inspections – these are the ones that were provided by Holmes. At the time we received these they were not exactly sure of where they occurred but these were the ones after the September the 4th earthquake and we also felt from this evidence here that these were unlikely to show that significant degradation of the walls had taken place in that earlier shot.

Other factors – we've concluded that the following factors were not significant contributors to the collapse - the ground conditions, as I said before, previous damage or vertical accelerations, and we also concluded that the modifications to the building structure in our opinion including the additional opening in the shear core on level 1 made in 1998 did not contribute to the eventual collapse.

We recommended in our report that territorial authorities, that is, City Councils generally should be encouraged to include screening for critical structural weaknesses which you could think of as being the Achille's heels of the building in their earthquake-prone building policies and the existing Building Assessment Guidelines that the New Zealand Society of Earthquake Engineering has produced be reviewed to confirm that buildings of this type which have lightly centrally reinforced shear walls where the horizontal seismic resistance is provided solely by the shear walls should be identified potentially poor performing in earthquakes and if necessary the guidelines should be revised to ensure that this is achieved and the performance of this building during the 22nd of February earthquake has highlighted the potential vulnerability in large earthquakes of lightly centrally reinforced shear walls without confinement, that is, extra steel to keep the concrete around the main steel bars, especially where the horizontal resistance to earthquake is provided solely by the shear wall. It means that there are no other elements to sort of hold hands and provide some resilience if that first element goes and so the further investigation of the seismic performance of these existing walls is considered a priority. So that's my last power point and the intention is now that my colleague, Mr Jury, will give the Commission some more insight into some of the technical things that he's been able to see from the computer analyses.

EXAMINATION CONTINUES: MR MILLS

Q. Just go back to that, Dr Sharpe. This is this other conclusions page and at least as written and my understanding of it is that you would (inaudible: 13:12:53) is that it was able to sustain it until the point that the compression failure occurred in the shear core. So it's correct as written isn't it?

A. It is correct as written. We have concluded that the perimeter frame was able to sustain the building deformations up to the point that the wall really started to take off.

Q. Yes I thought that's what you intended to say. So it's correct as written?

A. It's correct as written.

Q. Thank you Dr Sharpe.

COMMISSION ADJOURNS: 1.13 PM

COMMISSION RESUMES: 2.16 PM

DR SHARPE CONTINUES:

A. Towards the end of my presentation before the break I skipped a slide that is now showing. Any warning of collapse – we reviewed the provided information on the damage sustained in the 4th of September and the Boxing Day earthquakes and we have concluded that there were few, if any, signs that the building had been significantly distressed in the shaking that had occurred or that collapse was a possibility. Then I did show the next slide which showed the pictures of the damage.

ROB JURY:

We have thought about how best to convey the information that we have determined from analysis, the analyses we have carried out, and we felt that probably the best way of doing that was to show the videos that we have taken of the inelastic time history analyses that we have completed for all three of the earthquakes we have been asked to investigate and take the Commission through those. They will probably stimulate some questions and what have you along the way. Could I have the file of videos?

VIDEO IS PLAYED TO THE COMMISSION

It has already been mentioned this morning about the inelastic time history analysis and what is possible to do with these analyses but I have to preface my presentation with the comment that although they seem to be very sophisticated they are reliant totally on the input assumptions that are made and perhaps unlike other analyses that structural engineers carry out those assumptions that are made can very much determine the result that you get. So what we are aiming for in carrying out the time history analysis such as those that I am going to show you the results of is, we are trying to pick up the general trends of what we think performance of the building might be. We test a lot of the assumptions that we are making through sensitivity analysis and that really involves running the analyses several times with varying some of the input parameters and we try to pick up those points that are making a different to the overall response. As I mentioned before we have run these analyses for the September earthquake. We have run them for the Boxing Day earthquake and also finally for the 22nd of February and although I do not have the video with me today we have also run it with all three earthquakes in sequence. They are very time consuming analyses to do. That one with all the earthquakes strung one after the other took 26 hours to run on a very powerful machine, involves many millions of calculations of the status of the building at various stages through the earthquake. So what we are attempting to do is to take a computer representation of the building which you can see up on the screen there and I will go through that in a minute, and we are putting an earthquake input motion to it and then the analysis is calculating the status of the building at relatively small time steps as you go through the earthquake. While it is going through the earthquake it is also monitoring what is happening to each of the elements in the model and varying those, if necessary, and I will show you that in a minute as well what I mean by that. Just in terms of what is on the screen here so that you have got some feeling for it. We have in this area here where I am showing with the mouse, we have the representation of the earthquake and you will see this vertical cursor will move across the screen as we are going through the earthquake and you will see what is happening to the structure as it does that. We can slow that down or make it faster but because this is a video I cannot adjust it on line today but I do have a representation which is a little bit slower than the one I am going to show you initially. I can stop it where and discuss various points along the way where that might be necessary. Up in this box here we have various colours which describe the state of stress in the member from white which means normal stress so nothing in particular is happening to this blue colour here which is typically yield in the material so either concrete yield or steel yield, right through to when the theoretical capacity or when I say theoretical it is theoretical as in terms of this analysis capacity of the member has been exhausted and that will show up as red. So as we run these videos you will see the various colours in some of the members changing and that is an indication of the stress as we run through the earthquake. Just looking at the model itself we have, this is looking from Cambridge Terrace slightly to the west and up a bit and we have the shear core that we mentioned quite a bit here, in here represented by this forest of lines which each line is an element in that shear wall, either a vertical piece of wall or a horizontal piece of wall. Every wall, piece of wall is modelled in its correct location and then around that we have the gravity frame which is this extent of it is around here, to the point where you have the step back that we have been talking about this morning cantilevering pieces of beam here onto the perimeter columns which are running round at ground level.

Q. So just to make sure I am understanding the orientation of this, the viewer is looking at the building effectively from the southwest?

A. From the southwest, yes. The core is centric to the main building plan so it in terms of the east/west direction it is symmetrical but in terms of the north/south it is more to the north than to the south. That obviously has some effects as you might see. The earthquake that we are running through these analyses for these representations is the rest home record. That was the one that was 670 metres to the north/west of the site, and as my colleague, Dr Sharpe, mentioned this morning that is the one that we have decided is, all things considered, the most representative of what might have gone on in this site. We cannot, obviously, cannot be absolutely confident that that is what it is. There are certainly differences between the ground conditions on this site and the one where we are taking it but so there are in all the other records. This is probably the best because of its proximity of what we can gauge happened at this site on each of these earthquakes. We are indeed lucky that we do have these records because certainly the September event was well recorded as earthquakes go and so we do have a lot of information of what happened on September. Following aftershocks of course are generally well recorded because instruments come in from all over the place as people try to pick up the aftershocks so we are indeed lucky that we do have the September events.

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A. This model is a little bit more sophisticated than normal in that in the area of the shear core between level 1 and level 2 the wall is modelled as a series of fibres, vertical fibres, so we actually have the reinforcing rods modelled so we monitor what's happening to the reinforcing rods and we also have the concrete, um, or the wall divided up not probably quite as we would like it but within the confines of what's available in the programme to divide up the core wall into sections so we can monitor what's happening to individual parts of the core as you go round the various walls. At each of the analyses that I will show you, we have assumed that the building is essentially undamaged at the beginning of the earthquake. That is just an assumption that we have done for these videos. As I say, we have run it for all the earthquakes in sequence and we find that the difference is only minor, relatively minor, in terms of the overall result and the conclusions certainly do not change as a result of having that pre-damaged state before the event occurs.

JUSTICE COOPER:

Q. So just to make sure I'm understanding that, you've run them in sequence which means that you applied the Boxing Day event to the structure as it was after the September event, ditto with the February event compared to Boxing Day, and that's no different in terms of this analysis from the approach of running each earthquake separately, assuming at the beginning of each that the building is undamaged?

A. That is correct. Ideally we would want to run all our analyses with the earthquakes in sequence but the time, 26 hours I mentioned in terms of analysis time, this means it's impractical to run many multiple analyses that are necessary to test the assumptions so that's the reason why we've run each earthquake separately.

COMMISSIONER FENWICK:

Q. Can I just check that there's no significant difference. I assumed there was a difference but no significant difference?

A. Ah, the end result for the February earthquake, almost to the second, fraction of a second, was the same.

Q. So no difference at the end but there would have been differences up to then?

A Yes but I think if you watch the videos for those earlier earthquakes you'll see the state of stress as we are predicting, um, at the end of each of those earthquakes. I wonder if I could have that document BUI.CAM233.051A59. I mentioned that we modelled the materials particularly in that area of the shear wall between level 1 and level 2 with the constant materials and we have, um, made some assumptions in terms of what those materials might look like and this diagram here comes from our report which shows what we think is a reasonable representation of what the stress/strain relationship might look like for the concrete members and so we have, um, as the concrete takes loading compression, ah, the stress builds up, the concrete starts to crack and gradually yields or gets a bit of damage in it until it reaches a peak in its stress. If we keep on loading it, if it's unconfined, then it will fail and we've assumed that it fails in something like this. Over to the next page please.

In terms of the reinforcing steel the diagram is similar but different but the steel takes on load until it reaches its yield stress and then there's a plateau where if you keep on loading it, pulling it or pulling it in tension then it doesn't take on any more load but it will plastically deform, and then you reach a certain point where if you put more strain into the bar the stress will gradually rise. This is what we refer to as 'strain hardening' until you reach a peak level of stress and then that is maintained until the bar fractures, which will occur out here somewhere. Could I have the next diagram please?

Perhaps go two on, skip the next one. Now in our analyses we are limited to what the programme will allow us to do and we had modelled each of these – first for the concrete and then for the steel using these representations which is the best the programme will allow us to model so if you recall what I was showing before was the curve that went over like that, the concrete one's not too bad, by a series of points and straight lines we can model that behaviour quite well, but in the steel that was a little limited and we had to make some approximations particularly in this area. Now in terms of the colours I was showing you on the screen in the video shot initially, red is up to this point in the concrete, up to this point in the steel. The peak level I think is in the blue area, maybe into the green, and then red is out at this level of strain here for the concrete and out of the rupture strain for the steel.

JUSTICE COOPER:

Q. You've lost me I'm sorry. You started with red.

A. Sorry if I confused. White is the representation up to this point, not red, sorry. Then green and blue and then finally red and the same with the steel – white up to this point, green and blue and red once you reach this point in terms of the strain.

COMMISSIONER FENWICK:

Q. Was the unloading linear?

A. Ah, the unloading follows, it's just a linear type of unloading, yes, Richard. It's not a degrading stiffness model if that's what you mean.

Q. Parallel to –

A. Parallel to the yield.

Q. Initial pre yield?
A. Yep.
Q. And the same on the concrete?
A. Yes.

WITNESS JURY CONTINUES:

Can I have the videos. So I'm going to run through now. This is the September earthquake. The displacements are exaggerated so when you see it wiggling around it's about 18 times I think the magnification on the displacements. Now the time is a little bit distorted too but you'll get the picture of it so I'll run it all the way through. This is the September earthquake. So watch this cursor running across here. You can see at this point there is a little bit of twisting in the plan. You can see some colours just appearing down here. So this is the area where you might expect to see a bit of damage at this point. That's around the lift shaft. You can see the columns over here on the end of the shear wall that are out near that back stair.

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This one is showing signs that it's reached a limit, but that is not a material limit, that's an element limit. Just the next colour up in here but not yet reaching the failure of those materials, and then just coming through to the end of the earthquake now. So just in terms of what we think has gone on in this earthquake to this building based on what this analysis is telling us, is that we have some damage in the base region of the wall between level one and level two, we might expect to have some quite reasonable cracking in these rear columns that are immediately adjacent to the back of the shear core, but nothing that in the terms of the shear core that indicates signs of rupturing of the steel or reaching the compressive capacity of the concrete.

So I go on and look at the Boxing Day earthquake. So this is starting as I said before with it effectively undamaged. It's the assumption. You can see signs of cracking down in here, or the yielding of the reinforcing. The other thing I should say is that these lines indicate cumulative damage so once the colour goes on it remains on until it's succeeded again, so the stress may well have been coming off it as the earthquake progresses. And then we're in the decaying mode of the earthquake as it just ramps down. So then in the Boxing Day event as we might expect when we look at the response spectra that Dr Sharpe was showing before lunch, we were only expecting relatively small amounts of damage, probably less than September so we would expect to have seen maybe cracks slightly opening up, but really not enough shaking there to really cause a significant amount of additional damage would be our assessment.

And then lastly the February event. I might run it a couple of times, so we're now into the earthquake and see almost immediately damage to the base of the wall, damage to those back columns but if they had been damaged in September that would have already been there and then a very large lurch creates yielding in the columns around the perimeter, and then you can start seeing the walls now are starting to fail both in terms of tension yield in the steel. I'll go back a bit and show you that but also you're starting to get compression failures in the base until finally the analysis can't track it any further, just going to start that one again. So just running this one a bit slower, take you through it. So here we're seeing just getting beyond yield in the steel across this particular wall and this wall here, this is the wall at the front of the lift shaft and over the back here. You're seeing the building lurching towards the east and you're seeing quite a lot of steel yielding in around here, is what this is signalling. The same in here, getting more and more steel yielding across the walls and a lurch to the west, another lurch to the west. Now you can see the wall almost rocking on the western side, so this is indicating breaking the steel in these walls.

JUSTICE COOPER:

Q. Did you say rocking?

A. Rocking, rocking about the edge of the wall, yes. So that the steel has yielded, fractured in some parts obviously not all the way across, and it's now lifting on its western edge, and then coming right back towards the west, it's now rocking right up on its east wall now, you can see all this area here, this is the concrete going around this area here and then finally slumping, this analysis showing slumping slightly towards the south east. These columns here, just the representation here, this is really a plastic hinge at the base of the column, it does indicate that it's reached its theoretical moment or curvature capacity, but remembering that these all have props behind them, that tends to take the vertical load if those columns are finding it a bit of distress and they don't fail completely until the wall fails completely and the building is rocking right to the east. Now the, at this point the analysis can't keep track of what's going on with all the elements. It's said, "Give me help", but it just terminates the analysis – there are far too many members that have gone but in reality it's this round here where as this analysis predicting the initiation of compression failure. Now this, this model only models compression and tension, it's not modelling correctly the shear in the wall and I can discuss that further if, if anybody wishes to ask that question. That was all I had.

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Q. Can I just ask, you assign as the reason for the collapse that the east wall in the core failed above level 1, which is your key sort of determination as I understand it. Is that dependent on this analysis or does this analysis confirm a conclusion reached by other means?

A. We carried out a series of different analyses from elastic analyses through to a push-over analysis which is effectively taking a structure and just gradually pushing it over.

Q. Yes.

A. We've also looked at the displacement and the, or the rotations and what have you that occurred at the base of a wall of this type as a single element so we referred to that in our report as the "stick" model.

Q. Yes.

A. It all, all that, those walls condense into one element and we push that over as well. We've, we determined fairly early on that if you push this wall far enough it will reach its compressive strain, its fracture strain in the concrete. Effectively the steel yielding is something that's predicted out of those section type analyses as well but it's only when you put it all together that you see how it might have happened. Only saying "might" because it's only a representation.

JUSTICE COOPER:
Mr Mills.

MR MILLS:
Well I did have a few questions of both of the witnesses on the recommendations coming out of their report but I am concerned about time and just wondering whether they might come out quite adequately in the panel discussion.

JUSTICE COOPER:
All right well will you see they do?

MR MILLS:
Yes all right. So on that basis I'm happy to move this along.

JUSTICE COOPER:
All right, Mr Elliott.

MR ELLIOTT:
Your Honour, two particular areas of interest for families and those injured would be the seismic evaluation in 1997 and the condition of the building after September 2010 and I see that Professor Priestley addresses those issues in his presentation.

JUSTICE COOPER:
Yes.

MR ELLIOTT:

And I envisage that if there were any areas of disagreement between experts they'll be dealt with in the panel so I wasn't anticipating engaging with these witnesses on those matters.

JUSTICE COOPER:

Very well.

MR ELLIOTT:

Given the interests of time as well, so it's really just two, two areas which I'd like to discuss briefly.

CROSS-EXAMINATION: MR ELLIOTT

Q. The first is just to ask the report contains some conclusions about the strength of concrete and reinforcing steel. And am I right in saying that the Hyland report referred to some analyses but that those samples weren't taken from the shear core at all?

A. That is correct.

Q. And that's no criticism of him it's just because at the time he came to examine things partial disconstruction had taken place and it wasn't probably clear what he should be examining?

A. That's correct.

Q. And your own investigations, did they address the strength of the shear core?

A. Yes we, we went out to the Burwood landfill and attempted to locate pieces of the shear core and we located approximately half a dozen pieces of structure which was broken concrete with steel sticking out of it, yep, about that sort of size. We had those pulled out of the rubble. We assessed that they were pieces of the shear core simply by the thickness of the concrete and the spacing of the reinforcing that was in those particular samples. We have no idea where they in particular in the shear core that they came from but we can be pretty certain that they were pieces of the shear core concrete.

Q. I see so would it have assisted you in the conduct of your investigation if you'd been able to get a chance to look at the building before it was deconstructed?

A. Absolutely essential and knowing what we know now it would have been invaluable.

Q. And the second issue is just around this issue which has come out today and last week with the Holmes Consulting witnesses around the inspection following the earthquake and it emerged last week and again today that there was this distinction between building capacity on the one hand and diminished capacity on the other and there was an expression that the Holmes witnesses used which was the extent of the damage was not indicative of a building under immediate distress or having a significant impaired resistance to earthquake shaking. Can I just show you document CAM233.0051.7?

WITNESS REFERRED TO DOCUMENT

Q. And just the last two lines of that document. If they could be highlighted. So this is from the Beca report that you prepared and it seems that those same words are used there, "Not indicative of a building under immediate distress or having a significantly impaired resistance to earthquake shaking". So it appears from that that the question that Beca, that you were posing yourselves in considering the inspections was the same question that Holmes Consulting was asking itself at the time the inspections were carried out and then of course your report was approved by the, by the expert panel so my, my question really just is do you appreciate that the public might consider that engineers have been asking the wrong question? Is this a question that all engineers ask and if so what comments do you have on that?

A. I think the, the difficulty from my point of view is there's obviously a mismatch between what the engineering profession believe they are doing and what the public have thought they were doing. That's not necessarily solely the engineering profession's making, that lack of understanding because I, I know there have been a number of us for many years trying to explain what it is we would be doing once, when an earthquake did occur and in the aftermath of the earthquake. I think it was, it's quite important for us in terms of PGC to, to make a comment like this in terms of the evidence that we saw and the evidence that we saw has been primarily provided to us we haven't been able to go out and have a look at it ourselves so it's been based on information that's been provided to us but we would certainly make that conclusion based on that evidence and I think also in terms of what was being done in those days following the, the main earthquake shock and also the subsequent aftershocks was really to try and establish whether the building's condition had seriously changed to the point that in any further shaking that it might be detrimentally affected.

Q. Would you agree that as a learning point it might be a better question for engineers to ask, and this is no criticism, but it might be a better question to ask about the building's actual capacity, if possible, rather than diminished capacity in making an assessment about whether the building should be reopened for occupation?

A. I, I think so, but just another comment. I think in terms of that I think you're alluding to how safe is the building following, when the engineer's doing that inspection and safety is a very relative term and I don't, I don't think that either we have been explaining at any point what we mean by the words have been used "safe to occupy". I would not deliberately use the word "safe" because I think it means different things to different people. No building – even modern buildings – are absolutely safe in earthquake shaking. The older the building the lower its resistance the more unsafe it may be but if you get an earthquake like February earthquake it's not beyond the realms of possibility that a real modern building could also find itself in distress for various reasons and so I think it's important to preface those terms and think about what the engineer is trying to look at immediately after the event with that idea of what is safe and what are people really trying to achieve. I know from my personal observations immediately after September a lot of people wanted to get into their buildings very quickly to be able to get their personal belongings out. If buildings had been closed at that point because of where they sat in the continuum, even though they were not damaged significantly so, then that might have really affected the way the city became operational again perhaps.

1456

Q. Given the uncertainty around different types of buildings and this need you've mentioned to at least get some open, is one way of dealing with it to close say pre 1976 buildings subject to a full detailed inspection rather than opening them without such an inspection?

A. I think it must depend on the level of shaking that has occurred. We were thinking about the earthquake that we've just experienced in Wellington over the weekend. Is it seriously being considered that we would close all earthquake-prone buildings until detailed assessments have been carried out on those in Wellington? It must be a matter of degree and the degree is determined somewhat by what people observe to a particular building immediately after such an event. Some buildings will be damaged to a minor degree, some to a more significant degree, so a lot relies on the judgement of those looking at the buildings immediately after how far they take it.

CROSS-EXAMINATION: MR HERON – NIL

COMMISSIONER FENWICK:

Q. Yes, it's important from our point of view that we learn as much as we can from this building so we can apply it to other buildings, so I've got a number of questions some of which you have some warning about which really come from I guess that people who are practising structural engineering in the 60's, I've been one of them, would call a back of an envelope type assessment. You've carried out a very sophisticated assessment of the time history model and perhaps a less sophisticated assessment with the stick model. Now time history models of course are very sensitive to the way in which you assess the parameters and the time so the first point I've got here is on the way on which you modelled the section properties, you have reduced the stiffness by a factor of .4 of the finite element model and you have then somehow or other reduced this stiffness for a stick model to give you comparable values. Now what I really want to ask you is, do you think that the use of the production factor on the stiffness would have had an appreciable difference in the response of the structure and exactly where did this stiffness reduction factor come from?

A. The initial stiffness that you referred to really only affects the structure while it remains elastic, so the minute it goes plastic then the stiffness changes completely in the overall structure so to a certain extent it's only relevant while the structure is elastic. The .4 that we adopted to allow for that initial cracking if you like, came from what I'd refer to as being generally acceptable, accepted values for that initial cracking stiffness. In terms of its overall effect, we have run sensitivity analyses changing that value. It doesn't seem to make a great deal of difference for the February event. We didn't actually run it for the September and Boxing Day, but my feeling would be it make a bit more difference to those analyses because they're far more elastic in their behaviour.

Q. Given the fact that the cracking is limited in its area to level one and above that, the remainding [*sic*] of the structure, presumably remains relatively elastic, so do you think that might have had some influence on the overall behaviour?

A. I'm picking that you're asking about the change in stiffness as you go up the building maybe, and maybe go down the building. We didn't test that in our sensitivity analyse. My own view would be it might have made a difference in terms of the exact way in which we see the building respond to the actual earthquake, but I'd be surprised if it made a big difference in terms of our overall conclusion and the overall performance of the building.

Q. Do you think it's possible that that increased stiffness higher up give a lower cracking, could have concentrated the inelastic deformation into a shorter zone than perhaps the analysis would have produced?

A. Yes I think that's entirely possible. I think also if you look at the moment curvature diagram for our stick model which was putting all the elements of the wall in together, which is in our report –

Q. I could not find that diagram, so I'd be grateful if you'd show me where it is.

A. On A4.6.1. The section moment curvature plot.

Q. You refer to it in the text as 4.5.1.

A. Oh. My apologies. That was a guess. But what could potentially concentrate the actions in the base of the wall is the fact that you get a drop off, as you come you get good behaviour of that section across from yield, across almost to a curvature of .01, and then you start getting the fracture as the tension reinforcement in the flange, you get that drop off in capacity. That has the real potential to concentrate the actions in the base of the wall.

Q. Perhaps we can return to that particular aspect later on. Just while this figure's up, I take it that level one should have 900 tons of mass, level two, 675. Is that correct? In figure 4.3.2.

A. Table 4.3.2?

Q. Yes.

A. The masses are missing. Yes, no I didn't realise they were missing. Oh yes sorry they should be the levels, yes. No, that's correct. The ones, the numbers under the level column are the masses and tons, the levels will be - yes, one, two, three, four.

JUSTICE COOPER:

Q. But the 900 et cetera should be moved over to your right-hand –

A. That's right, that's right, just the 900 part, yes.

COMMISSIONER FENWICK CONTINUES:

Q. Just returning to your stick model if I may please, I assume from looking at the table of properties this, you've purely used this as a pushover analysis in the east west direction, you didn't try to run it in the north south direction or the vertical direction. It's just a pushover was it, not a time history model?

A. It's just a pushover yes.

Q. And just in the east west?

A. That's right, and it was primarily run simply because we realise that trying to proof the time history analysis was extremely problematical. It's quite complicated and we wanted to get a handle on approximately what we might have expected. That's the reason why we ran that very simple model, we –

Q. Pushover with actual load?

A. That's right, that's right.

Q. If we can have the sketch, this is BUICAM233.156.3. In the east wall, the wall towards the north east corner is supported on a slab, so on the left-hand side you see an elevation of the east wall?

A. Yes.

1506

Q. And on the right-hand side of that there's an opening, there's another opening on the left-hand side but there's a beam above it, there's another opening on the right-hand side there, so that when actual load comes down on that, it's got to somehow dodge. Can we go back to the sketch number one, which will be the same number only it's just .1 instead of .3. There we are. So that’s a plan on level 1 showing the shear core through the middle and then there’s a section 1.1 which looks through part of that wall that’s on the right-hand side and you can see the wall coming down and there’s an offset to the slab and the wall going down below that. Because that slab will be fairly weak that wall is really not effective, not effectively supported. Now there’s very little mention in the report. Your model I think, as far as I can see from your final (inaudible: 15:06:50) model, you model that but I’m wondering what effect do you think that offset would have on the performance of the wall?

A. I think we would have expected it to have the effect that you’ve shown in that diagram on page 3 and it does require a load spread to the hard points which are the walls on either side on gridlines C and B, so you would expect concentration of effects at those areas on B and C. What happens after that I am not absolutely certain. I know when we run the analyses we find that there is some indication of some damage over in that area but it doesn't seem to be where it initially occurs. Now that could be because of the particular earthquake, could be because of the particular direction but it doesn't seem to be the area where the main damage is predicted to occur from our analyses that we did run.

Q. When you're looking at the wall, if I understand you correctly, you're saying that the wall measures the tension forces and the reinforcement and the compression forces in the concrete but do you think it's possible, looking at the figure on the left-hand side, the elevation of the wall where clearly the compression force coming down the wall has to move towards the hard points. So in that wall on the two sides you're going to have high compression stresses coming in with high shear stresses. Now would your model predict, do you think your model would predict the sort of shear diagonal tension resulting from that with the compression forces. Would it predict the failure of the concrete due to that action?

A. Our model would not be terribly good at predicting the shear because the shear is not modelled precisely. Um, it certainly models the, um, compression and the hard points under the wall at those points, um, for both tension and compression. Ah, the effect on the shear I cannot say from the analyses that we had.

Q. You can't give me any idea the sort of compression stress you're thinking of at those points?

A. Well I know that the analyses in the video that I showed you showed all the steel gone along that wall b, little b, um, but it didn't necessarily show a very high compression stress along there maybe because of the soft spot that's caused by the fact that this wall is not there.

Q. The point I'm intrigued in is whether that offset in the wall, whether you think that offset in the wall could have played any part in the collapse of this wall or if it had been there would it have just been possible it would have rocked backwards and forwards without giving some distortion which might have affected either this end or the far end of the wall. Do you think that's a possibility that could have been a fatal weakness in the structure?

A. It could be, yes certainly. Um, when you certainly look at the drawings of this building that does look like a potential weakness in the building.

Q. One of the points behind this is to try to identify what are potential weaknesses so we can carry it forward. I'm not disputing your results of your analysis or anything like that so please don't take that comment. If we look at the shear core and if we go to figure, um, it's all right I've got it. Where would you expect the most significant crack, first initial significant cracks to form in the wall? I think you've indicated before this would be at level 1?

A. I believe it's level 1. I believe it's immediately above the floor and –

Q. That's quite likely given the fact that you're casting old concrete on new concrete which is probably not a hundred percent bond?

A. Yeah.

Q. So I mean I think we can say that's highly likely. Now the question I have is you've got five eight bars about 200 square millimetres crossing that crack at 15 inch centres about 380 millimetres in an eight inch wall which is about 200 millimetres wide. Now given the tension force that can be transferred across that crack could you get another crack in the close locality of the original crack. Is that possible?

A. I suspect not. It would be fair to say that the issue of these lightly reinforced walls is causing quite a lot of thought within the engineering profession at the present time. At the time we wrote most of this report there was still quite a lot of thought about what that might be. It is of relatively recent times that there has been quite a lot of talk about the tensile capacity of the concrete and the inability of the small amounts of steel going across a potential crack to generate a further crack. I think that's quite possibly a good explanation for why we had seen on other buildings. We don't know whether it was the case on this building but on other buildings where we had seen a single crack, while a crack that has almost closed up but is hidden away, fractured steel.

Q. Just my crude calculations again, the stress which you induce in that concrete is in the order of a third to a half which you would expect on average?

A. Yeah.

Q. So I think we're agreeing there but given that the answer is now you have assumed in your analysis a plastic hinge length of 400 or 800 millimetres. I think the 400 you were referring to was a fibre length which means presumably the stress in that bar will exist or the strain in that bar will be uniform over that 400 millimetres which would roughly correspond to what I would define as a 'ductile detailing length' of 800 millimetres. Is that right?

A. Approximately so. We did test that assumption in our analyses and changed that length and we found that at the sort of level that we have reported on it's pretty much where it trends towards. In other words you make it any smaller it doesn't seem to change the overall behaviour very much at all.

Q. Can we go to the Beca report, 51A.65 please. You've also got in there, 'yielding over a length of six bar diameters'. Now I don't know if you're familiar with the work carried out by some of the people in the engineering advisory group looking at the length over which reinforcing bars will yield but it looks as though six bar diameters, three bar diameters each side of the crack, is pretty well what they're measuring or appear to be measuring in some tests. So the question I have for you is if it was just yielding over the order of 70-80 millimetres in length, and that would be a peak strain midway between the crack and decreasing to that, how do you think that would have affected your analysis?

A. I can only repeat what I said before I guess is that without knowing what it was we tested that assumption of that length and ran it with a number and found that it did make a lot of difference around about that level so we could reduce it further and it didn't seem to make a great deal of difference in terms of the overall performance of the building.

Q. Did you reduce it by an order of magnitude one-tenth of that?
A. Yes.
Q. You did?
A. Down to that order of six to ten times bar diameter.
Q. So in terms of the diagram which is now up on the screen behind you that yielding plateau length that you have would be greatly reduced?
A. Probably but that was the stick model. The model I'm talking about bearing was the actual inelastic time history analysis so all we were doing was blindly changing the parameter which is the height of that element that we were modelling as being the yielding fibre.

1516

Q. The actual rotation you could have sustained if it snapped, the length yielding over that length rather than over that length, sorry, 1.6 metres, the initial one being around about 80 millimetres? That must have influenced how far you could go before those bars snapped, would it not?
A. Certainly it would. All I am saying really is it did not seem to have much effect on the overall result. I wonder whether the reason for that might be that the steel is actually participating in a very small way to the overall moment resistance of this particular structure.
Q. You are right of course. It is only applies to about a third of it.
A. Yes so in that respect it is a portion of that third so maybe that is the reason why it is not showing up as a significant influence in terms of the performance that we see.
Q. I think you are right there but we will come back to that one shortly. If a crack formed in the wall, so if we can go to sketch number 5 please? So this will show a section through the wall and acting on that wall you have (inaudible 15:18:15) same series –

JUSTICE COOPER:

Same series as the ones that were, these are Commissioner Fenwick's sketches.

COMMISSIONER FENWICK CONTINUES:

Q. 233.156. So on the left-hand side there you see a section through the tower as a whole, you have forces coming in from the beams and the gravity load of each level four, three and two, critical section is one and we are assuming a crack at section one on the west wall, left-hand side, you have a tension force in the wall of the reinforcements uncracked but yielding of my rough figure 2500 kilonewtons and then you have gravity loads coming in pretty roughly at levels two, three and four on each side of the wall of 1250 kilonewtons plus or minus 20% but it was only the back of an envelope calculation. My question to you is that force on the left-hand side of the wall, the gravity loads and the tension force, has to be transferred to the compression side for equilibrium so there is a vertical shear force which in this case is operating on the uncracked concrete or only partially cracked concrete above the doorways so you have got that being resisted by three blocks of concrete. That is a simple equilibrium figure. How that shear is transmitted, there are four walls across the shear core. There is an end wall, there is a middle wall and there are two other walls at the south end which are close together. My question to you is what magnitude of shear stress do you think is in that concrete between the doors?

A. I am not sure of the actual –

Q. Sorry, I should have explained. There has been some warning of this question. One of the reasons for this is –

A. Picking up your question, we looked at the vertical shear stress through the coupling beams but that is not what you are asking is it? You are asking for the horizontal shear across the wall?

Q. I am asking for the vertical shear force in above the doorways.

A. Okay, that is the answer then. The number that we expected from our analyses was about one and a half megapascals of shear stress in that area. You would have seen from the plots or the videos that there was no sign of excessive yielding in those elements.

Q. But you cannot pick up shear deformation –

A. No that is true, that is true.

Q. Well I am a little surprised by your answer because I was getting twice that value and I was just working off the statics so either my loads are wrong or something is missing somewhere and when I put that shear stress on the member the next question was, could those beams resist that flexural action? And you cannot tell me that because we have different values in order of magnitude different but the conclusion I am coming to is, sorry my crude analysis led me to the, you are going to get diagonal cracking in those members and which corresponds to of course what the Holmes analysis found that those walls were likely to fail in shear.
A. So certainly if the actual shear stress was double what we were predicting then that would be an eventuality of that. We are obviously predicting much less shear across those elements.
Q. It could be useful if you could just do a quick mental calculation, not mental, a calculation later on just to confirm your figure or not but it does seem to be a difference there between your analysis and the Holmes analysis and the overly simplistic analysis I have made –
A. We were also –
Q. – it would be nice if we could sort that out.
A. – we were also predicting very small amounts of shear going across that wall C so that might be where the difference is coming from the quick analyses perhaps –
Q. Yes but it is hard to see how you cannot pick up the gravity load when it is acting on the wall of the tower and the tension force in the bottom, as I mean you know it should be able to sort that one out I think between us but it is a point –

JUSTICE COOPER:
Q. Do you have view on that Dr Sharpe?
A. No I do not have any view on that.

JUSTICE COOPER:
Q. Mr Jury, I think you said you have done calculations on that issue?

A. What we did is we interrogated our analysis results so we know what the shear is from our analyses in that element and that is where we picked that number that I have responded back on the questions -
Q. So could you share that with us?
A. Sure, sure.

COMMISSIONER FENWICK:

Q. I would also like the static calculations if you can.
A. Yes, sure. Remember that the wall is rocking through the most intense part of the shaking so the actual distribution has to take that into account as well.
Q. It could be much worse dynamically, we do not know –
A. Or it could be better.
Q. Moving up and down you get all of that sorts of effects, btu it is hard to see this not occurring at some stage. We talked before about the effect of plastic hinge length. Okay, if we accept that it is possible and I think you do that that reinforcement might have only yielded over a 70-80 millimetre length in which case it almost certainly would have snapped fairly early on. What do you think the consequences of that snapping might be?
A. I think the snapping of the reinforcement helps in some ways but is obviously detrimental in others. Snapping of the reinforcing actually allows the wall to rock which could limit the shear forces in it but at the same time once the steel has gone the shear resistance has gone as well if the wall is uplifting. So I am a bit in two places on this one. I can see benefits from the rocking mode and also can see there could be some detrimental effect on the overall shear resistance. It is a bit hard to say which would overall...
Q. Can we go to sketch 6 please?

WITNESS REFERRED TO SKETCH 6

1526

Q. So if the, the reinforcement snapped on say the west wall. Right now you lose one third of the moment resistance. It's got a potential now to rock backwards and forwards but if you look at the figure on the, any one of those top figures actually the, the mass of the floor is highly eccentric to that shear core. Now when that reinforcment's acting, of course that shear core is very stiff in resisting both flexure and torsion. But what would be the consequence on the torsional resistance if those bars snapped and it started to rock two or three or four millimetres up, can you say what do you think might happen?

A. I think it, if the, the shear or the shear flow around that wall would be disrupted, I accept that, you not only though theoretically have lost the steel in the flange, you've also lost it in the webs as well so it's, everything is up, it is uplifting on top of the compression wall if you like and so all your torsional resistance is having to come from that single line of the wall.

Q. Now do you agree that coming down through the tower from the top down to nearly level one you've got an intact concrete admittedly pierced rectangular hollow section?

A. Yes I accept that yes.

Q. But when you suddenly get down to level one you're, as far as the torsional resistance is going you're going to lose the west wall and presumably if you lose the west wall because torsional reaction is balanced you're going to lose the east wall and so your torsional action whatever it is, is going to be thrown on to the, to the outside transverse walls and to central ones, so the two central ones won't meet much so do you think, do you think there's a possibility that those bars snapped. You've lost your torsional resistance and whatever torsional motion you had there could have been quite a big torsional inertia pause it wouldn't have been going far I suspect, I think you probably agreed with me there because the wall was stiff?

A. Yep.

Q. But once you released that torsional stiffness what do you think would happen? If you think it's a possibility?

A. Well I think it's a possibility, yes certainly I wouldn't rule it out, I think that theoretically you'd expect that if you lost the torsional resistance that you'd get rotation. I, I don't think the evidence is necessarily there that this building had rotation though when it failed so –

Q. The torsional resistance of those two outside walls would be very small wouldn't it?

A. You mean the – (inaudible 15:29:10)

Q. – capacity was very small, the torsional resistance would be very small because I mean enormously we had a very high torsional resistance which would stop it moving far but suddenly we're dropping it down to a very low torsional resistance?

A. You're talking about the loss of the two long walls are you?

Q. Yes.

A. No certainly I think that that's quite, quite a reduction in torsional stiffness as you're going down to that lower level, deep – you see it's not unlike any rocking system though is it that where you're up on the tip and you're rotating about the compression edge that you do have that situation and, and yet structures still are able to rock without necessarily sliding and rotating around the base.

Q. But that could have in effect do you think could have concentrated the collision courses?

A. Definitely, definitely. I think our analyses of predicting it's concentrated at the southern end on that west – eastern wall –

Q. But excuse me your analyses were based on the fact that it was still resisting torsion?

A. Exactly, exactly. But it does show some rotation as well under that, you know, degree of actions say.

Q. When it actually drops do you think it could maintain that torsional rotation or do you think in the dropping process all the beams leaning against it and the rest of it would just cancel it out?

A. Yeah, no, could, could easily cancel it out, yeah.

Q. Just, just quickly on the, the vertical excitation. You indicate that your analyses indicate that or show there's very little vertical stiffness and I'm delighted to see you've, you've ranged through high soil stiffness to low soil stiffness because what I don't know you show me the, you've given me figures for the compaction of the, the stiff, original stiffness of the soil but I'm not too sure what you've done for the unloading stiffness of the soil so the question I really have here is this particular structure's been through quite a few earthquakes, several earthquakes. It has been, the structure's almost been acting like a vibrating roller on the, the soil, so what I want to know is how, how did you chose the vertical stiffness you made and do you think, is it possible that that stiffness could have been underestimated and in underestimating transmitted higher vertical forces to the soil than perhaps was initially assessed. Is that possible?

A. Certainly possible, yes. What - our modelling of the soil was elastic, so it wasn't inelastic other than the fact that we didn't, we put a gap element in to allow the wall to uplift underneath the –

Q. So it was, it was bi or tri-linear inelastic was it?

A. Bilinear yeah, I think well tri-linear, tri-linear yep.

Q. So in fact if I understand you rightly it could have actually stiffened up quite markedly due to the sort of compaction effect then?

A. It's quite possible yeah, certainly the investigations that we carried out on the, on the bits that we could see which was the ground floor level would, would have indicated that there was very little distress in the, or apparent distress of course covered in dust and swept clean would probably destroy anything that you might have seen perhaps but certainly there didn't appear to be any signs of uplifting or significant uplifting of the, of the foundation or pushing down of the foundation on the compression side. We're rather hopeful that we, when they start to pull up the slab we might get to look as well in the, underneath the slab and underneath the foundations to see if that was the case but in the boreholes that we cut through the ground floor slab we couldn't see any signs that would lead us to believe that there'd been major movements in the foundation. Vertically. And our, and our analyses we did test, we doubled and halved the stiffnesses that we finally used to see what effect that was. It didn't seem to make much effect at all but then it could be argued that we had high numbers anyway.

Q. So that was an elastic analysis –

A. (inaudible 15:33:40)

Q. – not a compaction?

A. No they were, they were the inelastic analyses but –

Q. I mean the soil?

A. But soil was, remained elastic and it was elastic model we assumed.

Q. Yes, it’s just that when you, you look at the response factor for the vertical earthquake it’s very sensitive to –

A. Yes.

Q. – vertical stiffness –

A. Yes.

Q. – which again is largely as you realise a function of the ground stiffness. But thank you very much that's very helpful I appreciate it.

A. Can I comment on that vertical acceleration?

Q. Yes, sure, please.

A. My own feeling is that the vertical accelerations were large in this earthquake and may not have helped but there are so many other reasons for the failures that we’re seeing that it just seems that to blame it on vertical accelerations may be a little bit of a copout perhaps but there are many other reasons that we have been talking about that are more likely to be reasons.

Q. In connection with this can we have sketch 7 please.

WITNESS REFERRED TO SKETCH 7

Q. Sorry the point of bringing this up is just to indicate what you’re talking about. The top one shows the acceleration response horizontally in the east west direction and the vertical, and the bottom one shows the accelerations in the vertical direction. You can see there are an enormous number of accelerations in the vertical direction and my conclusion, well I may be completely wrong but my conclusion was that chances are you’re going to get a peak vertical acceleration when you’ve got a fairly high horizontal acceleration so the two are likely to coincide just because of the high frequency.

A. There is some issue about the accuracy of the vertical accelerations as well.

Q. Sure.
A. So –
Q. Yes.
A. – which is another problem of the verticals yes.
Q. Yes.
A. Mhm.
Q. If we have the, just the last one sketch 8. That just also throws a little bit more light on that.

WITNESS REFERRED TO SKETCH 8

Q. And that shows the displacement spectra for the horizontal and we're down on the left-hand side we're down about the .5 second mark and less for the period but in the vertical acceleration you can see there's a peak at around about, high peak at about .1 of a second which of course would only transmit, means you're only going transmit appreciable displacement which could compact I think, if you're at very low period which implies a very stiff soil.
A. Mmm.

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Q. Thank you very much, that's helpful and hopefully we've got a few points where we can look for other buildings which might have some of those features.

QUESTIONS FROM COMMISSION CARTER – NIL

JUSTICE COOPER ADDRESSES DR SHARPE:

Q. Can I just ask, I am probably behind everybody else, certainly on the bench here, in the inputs that you make to your time history analysis, you base them on records which are kept of the shaking some distance away. What do you assume in terms of the direction of the induced shaking? Is there one direction which predominates or do you assume equal directional effects around the points of the compass or what?
A. I think it was fortuitous that the instrument's location – the way in which the record was taken was in fact almost exactly north south, east west.

Q. Yes.
A. As was the building.
Q. Yes.
A. And there's no doubt that one of the records, I think the Cathedral one –
Q. The Catholic Cathedral or the Anglican?
A. No, the Catholic one, the school, I think that one was at odds with north south which cleared for all – all the buildings that were being looked at were in north south, but we can quite easily re-digitise that, it doesn't make any difference.
Q. Well what did you in fact do?
A. Well we, for the record that we took it was – we didn't have to do anything because we had the actual record in the directions of the way we were putting them into the building.
Q. Yes.
A. So that's an exact match or close match.

MR JURY:
A. In the other building we looked at that has a directionality which is at 45 degrees to the north south.
Q. Yes.
A. And so we rotated the earthquake around, used the same earthquake but rotated it around to see the effect of what might have happened if it had been coming from a slightly different direction.
Q. So the analysis that you carried out deals with induced movements in north south and east west?
A. Yes, simultaneously with the vertical.
Q. Right, and that I suppose pretty much a worst case scenario is it?

DR SHARPE:
A. Well it was in this case, it is a very similar to we would expect.
Q. Yes.
A. Just a matter of 700 metres away.

Q. Yes, it wouldn't be – by adopting that approach you would not be underselling the forces in induced by the earthquake, if I may put it that way, or do you think you might be?
A. I think it's one variable too many.

COMMISSIONER FENWICK:
Q. Yes, if however the building had been turned through 90 degrees, it would have been on its strong axis?
A. Mmm, mmm.
Q. Probably would have –

MR JURY:
A. But probably would it survive because of the –
Q. As it did in the September earthquake?

DR SHARPE:
A. Yes, I was just going to say the September earthquake was predominantly the other direction so I don't know, what would have happened then, we haven’t tested that, but –
Q. No torsion.
A. Yes.

JUSTICE COOPER ADDRESSES DR SHARPE:
Q. Just let me understand the measuring site you're talking about is the Catholic Cathedral College. Is that right?
A. That was the one where I said that – I think that's the one where the instrument was orientated about 45 degrees away from north south, but the ones that we used up to the north near Bealey Avenue at the rest home site –
Q. Yes.
A. The instrument was orientated or it traces almost north south, east west and so we could use it directly on our building.
Q. Is that the site with the initials RE-

A. HS. REHS that's correct, yes.

Q. Thank you very much.

A. This is the least of our problems I think, the orientation of the instruments.

COMMISSION ADJOURNS: 3.41 PM

COMMISSION RESUMES: 3.57 PM

MR MILLS CALLS

NIGEL PRIESTLEY (SWORN)

Q. Is your full name Michael John Nigel Priestley?

A. It is.

Q. Now I hesitate when I ask you about your residence because when I look at your CV I think you're probably a citizen of the world but for present purposes I'm going to ask you whether you are a resident of Christchurch?

A. Yes I am.

Q. Now I'm also going to ask a few points about your CV – again it's much more extensive than I'm going to ask you about and it is on our website in its full form but for present purposes you have a PhD from the University of Canterbury?

A. Yes I do.

Q. You were in the late '60s and through to the mid '70s the Head of the Structures Laboratory at the Ministry of Works?

A. I was.

Q. And, more latterly, you have been the Professor of Structural Engineering at the University of California, San Diego?

A. Correct.

Q. And you are still a Emeritus Professor of Structural Engineering -

A. Yes.

Q. – at the University of California, San Diego. From 2002 to 2008 you were the Co-Director of the European School for Advanced Studies in Reduction of Seismic Design at Pavia in Italy?
A. Correct.
Q. And you remain the Emeritus Co-Director of that school?
A. Correct.
Q. I will just ask you one or two other things. You have I see a whole range of Honorary Degrees and Doctorates but among those you are an Honorary Fellow of the Royal Society of New Zealand?
A. Yes.
Q. A Fellow of the American Concrete Institute?
A. Yes.
Q. A Fellow and Past President of the New Zealand Society for Earthquake Engineering?
A. Yes.
Q. And a Fellow of the Institute of Professional Engineers of New Zealand?
A. Yes.
Q. And that amongst your many distinguished prizes you have been twice awarded the Raymond C Reese Award of the American Concrete Institute which is their premiere structural research award?
A. Yes.
Q. And most recently the FIB Freyssinet Medal which is the premiere international structural concrete award. You were awarded that in 2010?
A. Yes.
Q. Thank you Professor Priestley. I think you are going to go through some power points with us and I'll just ask you to do that.
A. We've seen the view, the similar view of the building before, just to again identify a few points on it, notice the columns at the lower floor here, rather well confined in that they had steel jackets round them so they were cast inside steel jackets. That meant they had good ductility and we can see also the offset that has been referred to a number of times with the perimeter columns here being offset outside these

internal ones. These are rectangular columns and, as we have heard, have very poor detailing which was not uncommon in the 1960s when these were born, made, sorry.

I don't propose to dwell anything on the collapse of the building as we've seen many slides of this before and because of time I'll move through as much as I can. I should mention that I believe I'm here in sort of two roles – one as a member of the expert panel which is overviewing the work that is being done by the consultants, for example, by Beca Carter Hollings & Ferner and this expert panel which is chaired by a lawyer, Sherwin Williams, and includes representatives of consulting structural engineers, architects, building officials, seismologists, geotechnical engineers and academics as well, and the role of this expert panel is to assist and review the work by the consulting engineers appointed by the Department of Building and Housing and which were charged with investigating the collapse or damage to four buildings. These being the Pyne Gould Guinness, Forsyth Barr, the Hotel Grand Chancellor and the Christchurch Television buildings, and to provide a report to the Department of Building and Housing summarising the consultants' reports and placing them in a wider context. So that's what the role of the panel actually is. I've already mentioned some of the features but looking in the plan as well again noting that the area of particular interest which has been brought up by Beca Carter representatives has been the shear-core and this is shown here at level 1 itself where it was essentially the same as this from there to the roof but it has been mentioned that in the ground floor there was a step in the wall if you like and as well as having additional wall elements the wall underneath part of this was essentially missing in offset by about a metre and this we believe did have some structural significance to the response of the building. The columns are essentially just round the perimeter of the building apart from two slightly larger columns in this region here and here and the beams which support the floor run in these directions but you'll note that underneath these intersections there are no columns. So there's rather large spans of the beams from here that's seated on the core and out to the perimeter columns here and these two columns here have just been placed to ensure that there's not a region where there is no support of an adjacent column close by. We've also seen a lot of information on the acceleration response spectrum. That's essentially the amount of, it's related to the amount of force that would be resisted by a structure of unlimited strength as a function of the period and Dr Sharpe has illustrated what the period is to some extent and we note that at the building period of about .7 seconds. The response under different earthquake records which were recorded is rather similar to the design spectrum for current design itself. You'll also notice that there's a big lump out here which fortunately there were not many buildings in this region because that's very very much higher in the Darfield earthquake than we would design for. Perhaps more interesting and perhaps easier to understand for the lay person is what's called the displacement spectrum and what this is, is just an idea of how far a building will actually move in its upper levels in an earthquake as a function of the period.

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So this is in for example here we are saying that if the building period was two seconds and we were looking at the REHS record which is this one here then we would say that it might move as much as 350 millimetres, a very large amount and very much larger than most buildings that are designed for but found in the area where we are concerned in rather similar to the design spectrum and a displacement of about 50 millimetres, rather smaller. This now is just a comparison for the 22nd of February earthquake and this was the red line that we were seeing before which is the 500 year return period earthquake which we mainly designed for whereas this one as also has been shown by Dr Sharpe is the 2500 year return period earthquake and we do not normally design structures for that level of seismic intensity but you can see that the records that were taken in the CBD tend to even exceed the 2500 year return period motion at the range in the area that we are involved with, around about .7 seconds. Perhaps something that should also be mentioned is that the period of .7 seconds which have been mentioned is not something that stays put during the earthquake. If the building starts to go into the non-linear range as is anticipated then the period effectively increases so we tend to shift from around about here out into this region here, where the activity is rather higher. So it is no surprise that a number of buildings were damaged and that some failed, particularly those which were designed before the basic principles of ductile design of buildings were well understood and that started to become apparent in the late 1960s and the early 1970s and is still an evolving process. This just shows the displacement spectrum for the February earthquake, again compared with the normal design envelope used nowadays and this is the 2500 year return period event. The vertical acceleration has been discussed a little bit and the important thing to remember here is that the recordings of vertical acceleration in this earthquake are as large as have been recorded anywhere in the world. They really are quite phenomenal. The Beca Carter engineers have said that they do not think it had a significant influence. I am not so sure. I think that it may have had an influence on the response. The important thing is that it is not necessary to invoke vertical acceleration to explain the failure of this building and I think that that is the Beca Carter approach is to say it may have made things worse but it certainly would not have made things better and it would have failed now if there had been no vertical acceleration.

Now the building issues I think have been identified to a large amount. One critical one is the single layer of reinforcement in the wall with no confinement of the concrete and no restraint of the reinforcing steel inside it for buckling. The second point is the low reinforcement ratio. That is the, if you like, the area of reinforcing steel divided by the tributary area of concrete surrounding it and this was very low in this wall, about .25% which is much lower than we would like to see and we note that the strength of the reinforcement is less than the concrete tension strength, and this has been mentioned in the questions that were asked of Rob Jury by Richard Fenwick a little while ago.

Considerably less, now what this does is that it restricts the cracking, implies a very short plastic hinge length. That is the region over which inelastic action is spread in the wall itself and fracture of the reinforcement at level one is predicted at low displacements. The wall concrete compression strength is rather low, 25 megapascals but not unrealistically or unsafely low for a wall built in the 1960s. Very poor detailing of the columns above level one. That means very little transverse reinforcement and the transverse reinforcement wound round the vertical bars is intended to hold the core of the concrete in tact if the cover concrete which is the concrete outside that reinforcement cage spalls off which it tends to do at comparatively low strains itself. The short lap of the splices of the flexural reinforcement. That means when one bar runs out and you need to put another bar in you lap it by a certain amount. The amount of that was rather small and these aspects tended to lead to low displacement capacity of the columns. The capacity of the shear core under simultaneous north/south and east/west attack would be very suspect because under situations like this you tend to get all of the compression force concentrated in one corner of the core itself and that can create extremely high compression stresses and I believe that may have been a contributory factory to the failure here.

Now the Department of Building and Housing expert panel findings, essentially were an endorsement of Beca Carter's report in that it agreed that failure was initiated by tensile fracture of the flexural or vertical reinforcement of the west wall of the shear core followed by a compression failure of the east wall at level one. Large displacements subsequent to core failure caused failure of the columns and connections of the beams to the shear core with floor collapse resulting. Strength and detailing satisfied the building code in place when the Pyne Gould Guinness building was designed. Displacement capacity and detailing would not satisfy current, that is current February 2011 building code seismic intensity. The February 22 aftershock exceeded current February 2011 building code seismic intensity. Site foundation conditions were not instrumental in the building collapse. An inspection after September the 4th and Boxing Day shakes did not indicate significant damage to the building. So these are basically the conclusions of the expert panel which were essentially in agreement with the Beca Carter report. No fundamental issues. Some issues that perhaps should be aired a little bit though is one that the Holmes report, the analyses of 1997, came to different conclusions about critical weaknesses of the Pyne Gould Guinness than did the Beca analysis, and the question is, is this a concern? My view is that it is not and it is to be expected and I will come back to that in a minute. And the second one which may be more of a concern is, was condition of the PGC building after the September the 4th earthquake really as good as indicated by the post-earthquake evaluations? I am not so sure about that but I want to make it clear that this is not a feeling. I am not emphasising anything of the sort that might indicate that the work done by Holmes was sub-standard. I am mentioning that it is possible that we need to re-look into the way in which we do earthquake, post-earthquake evaluations of buildings which have perhaps known deficiencies and there has been a lot of time devoted to that in these hearings.

Comparison of the Holmes and the Beca analyses, first of all the important thing is that both found the building to be sub-standard. They just differed in what area was more sub-standard than the other. Both consultants used non-linear time history analyses which has been called inelastic time history analyses in these hearings which is just another name for it and similar computer programmes to form their opinions. The Beca analyses indicated that the walls of the shear core were critical and the columns and hence floor failure would occur subsequently. The Holmes analyses indicated that the column failure would precede wall failure which was not seen as so critical. Both analyses indicated that the columns were poorly detailed. The Holmes analyses used a representation of the current 1997 code seismic intensities. Beca used the accelerogram recorded near the site in the earthquakes and aftershocks.

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The analyses required subjective judgment of various aspects and Dr Sharpe has emphasised that again earlier today particularly about plastic hinge lengths and shear performance but there, it needs to be emphasised again that considerable decisions need to be made about how to represent the building in terms of characterising the different seismic resisting elements, how to characterise the non-linear performance, how to characterise the strength and various other aspects and so we emphasise at the end that although non-linear time history analysis is the most sophisticated analytical approach currently available, it's still an approximation to your actual performance and behaviour.

Now the issues with the Holmes analyses that I've got here are not serious issues, they're just things that I feel that perhaps if they were doing the analyses again now might be different. It was mentioned by John Hare this morning that some of these aspects would have been changed. Here I think it's been, the first point which it says that it appears, and this is from a read of the report, that the critical region for the shear core was incorrectly identified as the base of the wall. In fact it was at level one due to increased area of walls between the ground floor and level one, and it's been stated by John Hare that though that appears to be the case from the report, it was not in fact the case. They did have inelastic modelling of the wall capacity at level one and therefore did predict, though it is not mentioned in the report, that inelastic action would not occur at the ground level but at level one, so you can if you like ignore this. It's just a matter of the report perhaps being condensed too much to be appropriate. Methods for modelling plasticity at the wall base appeared to me to be inappropriate and that the plastic hinge length was very short in comparison with what we would model currently. The stiffness of the columns and the beams was based on information that was current in design codes in the 1970’s and so was appropriate for that but we have moved on a bit since that and know now that the stiffness of these columns and beams was overestimated. That would tend to make it the columns to appear to be more critical relative to the shear core than was actually the case, or at least would be our case based on current assessments of stiffness. It appears that the beam column joints were modelled as rigid elements. I still say that this is an appearance. Problems with the shear strength model for the wall, additional flexibility was applied because of the way in which they had interpreted some experimental results, some published experimental results. This would have tended to make the shear core a bit more flexible than anticipated so we can see what their analyses seems to be doing would be making the columns stiffer, that's the frames stiffer and more likely to be more critical, and to make the core rather more flexible and less likely to be critical, so there are reasons why it came up with the results. The next point, my own calculations of the rotational capacity of the columns above level one come up with a rather higher value than were used in the analysis. These are based from first principles analyses. The values that we used were essentially out of design codes so I think that the analyses were predicting failure of these columns at displacements that might be about 50 percent low. Only one set of records was used for the analysis. It's mentioned in the report that three sets, a minimum of three sets of analyses should be used but only one set was used for these analyses, though these were scaled to different intensities to see what would happen. But I do not see any of these issues as being particularly serious. The point in raising them is to understand why the Beca analyses came out with slightly different results. There are some issues that I have with the Beca analyses, but these are even more minor than I do have with the Holmes analyses. There is some confusion with over the plastic hinge length in the report, whether this was 400 millimetres, 800 millimetres or even 60 millimetres as mentioned but in terms of determining the displacement capacity, but I believe that the, finally the displacement capacity is based on this estimate here and these ones here are describing more a spread of plasticity rather than a region over which inelastic action happens rather than what we call the equivalent plastic hinge length.

JUSTICE COOPER:

Q. So the one that you think, I'm sorry to interrupt, but we've got to think of the record in the future and we don't pick up the indications with the mouse, so you're saying that you think they put their hat on 60 millimetres. Is that right?

A. I think the 60 millimetres is low in terms of that, so I think that something rather higher. Similarly the plastic hinge length for the columns I believe is reported as being 40 millimetres and I believe that's also too low. To some extent these are dictated by the type of modelling, the fibre modelling that they were using in their analysis. It's not clear how concrete tension capacity is dealt with in the analysis but I believe it was modelled. The displacement demand in the September 22nd earthquake is not clear in the report but it appears to have been exceeded according to the response spectrum analysis so this might be taken to indicate that perhaps failure might be expected, but we'll come back to that. The assessed displacement capacity of the full model seems to me to be a little low but not too much. Some adjustment to displacement demand capacity ratios also seems appropriate but I would note that the conclusions are not affected by any of these issues. It's a matter of degree rather than its substance.

The BECA displacement demand capacity ratios, and ultimately this is what we used to determine whether or not a failure is predicted, is on how far the structure can displace and how far we believe it was required to displace in the earthquake itself, so one is the demand, that's how much it's required to do, capacity is how much it can do before failing and I would emphasise that the demand is probably moderately difficult to determine in a particular earthquake but the capacity can be even harder in terms of displacement, so again judgement is required in both of these aspects. The pushover displacement capacity should be related to ductile, not the elastic spectrum and I'll show you what that means in a minute. The equivalent viscous damping I estimate as being 10 percent whereas it's been compared with five percent in the BECA Carter report. The reduction to demand is approximately 24 percent. The September 4th demand capacity apparently relates to the average of the horizontal spectra at four sites, and I maybe wrong in this but the February 22nd demand capacity to ratio apparently relates to the average of a square root sum of the square combinations and I realise this will mean nothing to some of you and I apologise but it seems to indicate to me that there may have been a small error in the way in which this was calculated. The displacement capacity could be about 30 percent higher than the stick model estimates and about 80 percent higher than the full model. Now that is based on my own very simple hand calculations and I wouldn't stake my life on them. So one of the types of comparisons that have been shown is what's called a capacity spectrum approach where we plot on the vertical axis the spectral acceleration of either the code or an earthquake and the displacement here and that enables us to plot a force displacement response which are these red and blue lines shown here to compare with the code requirement, and this is essentially the code requirement currently as shown here, and for five percent damping. I've said that the response itself, here's the stick model response that they used, here is the pushover response using the non linear time history analysis, and here's my estimate over here, somewhat different but pretty similar. The important thing here is that if you take into account the additional damping associated with the non-linearity of this response, then we pull this line back to about here and on the basis of my calculations it would say that it sort of would just about be expected to survive the current design requirements and the same thing would happen if you take the stick method approach from their analyses but not if you take the time history results.

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If you, now this is a real mess because this includes all of the earthquake records in the CDB [*sic*], it includes that line that we had before as the code requirement and this is again for the 4 September earthquake. This is an average of the combined worst possible direction of the earthquake action and here is the average of the horizontal components which is probably more realistic to take into account. Now what comes out of the, this little red line which is down here is an indication that the structure should not survived the Darfield earthquake if you take that as being the case because here's the average of the earthquake records itself. If you take the pushover response that I've calculated myself and then take this back by the 5% damping you find, sorry take this line the dotted red line which is the average of the earthquake records, you find that you would expect it to survive without collapse. Again these are only approximate estimates. If we do it now for the, for the September the 4th you can see here again this little –

Q. This February.

A. Thank you, apologies. This is for February 22nd, here is the predicted response here and this is when compared I believe by Beca with this red line here which is the square root sum of the squares, in other words, it's the worst possible direction of attack whereas I think that for consistency with the previous one it should have been compared with this curve here. Doesn't make much difference it's still way above this level and it's still way above this level here which would be the maximum that I would predict just based on, on a very simple hand analysis. So we certainly expect failure no matter what method is used under the, the February earthquake. There is some ambiguity but I believe we would not expect failure in the September earthquake.
The Pyne Gould Corporation condition after the September the 4th earthquake. The Beca response spectrum analyses indicated a demand capacity ratio greater than one which might indicate failure but both the non-linear time history analyses and my own analyses indicate no failure but yield of the wall. Eye witness accounts indicate increased

liveliness of the building post September the 4th. These again are subjective statements of course and it's hard to know whether these are the, correct or whether one is just more sensitive to them because an earthquake has occurred. Damage inspection indicates diagonal cracking and spalling in the shear core, the shear wall and cracks on the bottom of some columns. Crack widths were small after September 4th but it should be understood that the reinforcement ratio in the walls in particular is so low that the gravity loads would close the cracks after shaking ceased so we would expect even if the cracks had been quite large in the September the 4th earthquake that they would look small afterwards and this can give a false impression of safety. Spalling of the concrete surfaces in the shear wall indicative of sliding on the cracks which in my view may be considered as significant damage. Even if the wall reinforcement had fractured in September, which is considered unlikely, this would not necessarily be apparent. Again due to the very low reinforcement ratio of walls and there are examples in the Christchurch earthquake where this has occurred. There's been walls with low reinforcement ratios, they have not appeared to have suffered anything more than a single crack but when invasive inspection including knocking out of the end of the wall is concerned it's been observed that the walls, that the reinforcement has fractured. And these have been shown before and rather better examples than my rather blurry scans of them. I'm a little concerned here about the intersection of these diagonal cracks which indicate shear performance. The fact that the concrete has spalled off here and in these regions to me would seem to indicate that there has been some sliding movement along those cracks. It's very hard to estimate how important that is but it may be a more significant aspect than the crack widths itself which are extremely small.

And this brings me to essentially my final slide which in my view is, is very important and it is under discussion by many people throughout the engineering community in New Zealand which is that perhaps the visual assessment procedures need review. With reinforced concrete structures, visual assessments tend to be based on three aspects: the crack widths, the presence or absence of spalling of the concrete, and residual displacements. That means, is the building out of plumb? The significance of these aspects depends on the quality of detailing and construction of the building and I don't think that this has been properly identified and disseminated to the engineering community so there is a tendency to go in and look at a building and say, "The crack widths are small, there's no spalling, it's vertical therefore there's been no significant damage to the building". Well a structure such as Pyne Gould Guinness which had poor detailing may not display significant apparent damage even if it is taken quite close to its capacity. Small increases in the displacement demand may result in greatly increased visible damage or even failure. On the other hand a very well designed and detailed structure may be able to tolerate significant apparent damage quite wide crack widths, significant spalling of the concrete and maybe significant residual displacement without significantly affecting the capacity to sustain additional shaking, so I'm saying that there is a difference and there perhaps should be a difference when we're investigating buildings. Maybe not in a level 2 but certainly in other cases, to identify the fact that maybe we will see a difference significance to cracking in an earlier poorly designed building than in a later well designed building. Now this has been brought up in Beca's report and a suggestion for an active approach by Territorial Authorities recording critical structural weaknesses of older buildings and it has also been suggested in these hearings that this could be related to all buildings pre 1976. It would be very nice to think that if after an earthquake a building was being inspected it would not take too much to find out whether the age of the building was pre 1976, particularly if the, if the local Territorial Authority had information on this and perhaps we would look in rather more detail at some of the older buildings. So older buildings in this case might not be given a green status until both visual inspection and a review of the, the critical structural weaknesses were carried out. So it was known that there were critical structural weaknesses of this building. It is not certain by any manner of means that the September the 4th earthquake resulted in a diminishing of the response of, or the capacity of the building, but it might have been that in hindsight now that if we had known the age of the building and known that there were critical structural weaknesses that perhaps the, the review might have gone to the next level up which might be a good thing. In the Pyne Gould Guinness there were several critical structural weaknesses. The fact that there was a single layer of reinforcement in the walls, low reinforcement ratio in the walls, poor confinement of the columns, poor connection between the floors and the walls and the walls and the detail so it was not a happy building certainly. And that I believe is, is it.

No, I have one more which is something that bothered me in earlier stages which is the first stage of the failure which has been identified as probably a tension failure of the reinforcement on the west wall is, seems rather certain but it is not clear why we would have the, not immediately clear why it would fail in compression on the outer wall. In fact the reduction of the compression force corresponding to fracture of the tension reinforcement has been identified by Rob Jury as possibly a good thing in that it would mean that the compression stress on the wall would not be too bad. In fact if we look at the compression stresses if all of the weight of the, and – of the wall itself and the supported floor loads are considered and this was carried by the east flange alone as the west flange, or west wall is uplifted, then the compression stresses would only be about 12% of capacity on average. However, the east wall was not supported directly or between ground floor and one over the full length, there was about a 50% support, this has been mentioned earlier, and considering a stress distribution from zero on one side of the flange to maximum at the other, the maximum stress would be 50% of capacity on average over the, the wall.

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Simultaneous response in the east/west and north/south direction would tend to concentrate the compression at one end, either the north end or the south end, reducing the compression area and increasing the compression stresses still further, and I think that this is a significant aspect. If a wide crack occurred at level one which is what we believed happened because the reinforcement ratio was so small that it would not create a spread of plasticity, then the shear force would have to have been transmitted through the compression zone. Normally if we get a well-distributed pattern of cracking there is what is called aggregate interlock on the cracks which stop them moving relative to each other and transfer a large proportion of the shear force. With a single wide crack this would not occur and all of the compression force would have to be transferred through the compression zone and the calculations show that the shear stress would be very high even after the shear has reduced due to fracture of the flexural reinforcement in the wall. This combined with a high compression could cause failure of the compression zone particularly if we consider the additional effects of vertical acceleration, so I do believe that compression failure of the end wall influenced by shear possibly causing buckling is a real possibility in this case. That I am sure is it.

EXAMINATION CONTINUES: MR MILLS

Q. I just have a couple of matters I want to raise with you Professor Priestley. The first is I want to put up a document which is BUI.CAM233.0177 and I think I have given a copy of that to the Commissioners this morning. Now Professor Priestley I just want you to look at that and tell me whether, it is described as comments by Nigel Priestley, and I wonder if you can just explain what that document is in general terms?

WITNESS REFERRED TO DOCUMENT

A. Yes it is something that most of the expert panel members did on drafts of the various reports on the various buildings. Issues that they felt need to be addressed or that they were uncertain that they came through clearly and what this is in fact is a compilation of two different sets of comments. The first set of comments are in black on an earlier version

to the Beca report and then the red ones are ones which I updated in a later version of the report.
Q. And what are the yellow sections?
A. I am not sure. The yellow ones are points that I feel are more important so some of the issues are trivial and some of them are perhaps more important.
Q. And are you content that all of the points that you have raised here have now been dealt with in the Beca report to your satisfaction?
A. I think most of them have. Some of them I have raised, I have also raised in the meeting now here and probably one of the most problematic one has been the compression failure of the west wall which is predicted by their analyses and I have now described in that last slide that I believe it is the correct most probably mode of failure.
Q. So nothing still in here that ought to be identified to the Commission as an issue that is still problematic for you?
A. Not beyond what has been mentioned in my presentation because several of these points have been identified. For example, s 5.5.2 which is the photographs of the damage after September which I have seen there so I do not believe there is any aspect there which has not been either addressed by them and changed in their final report or that I have not brought up in this, in my presentation here.
Q. Thank you. The only other point I wanted to raise with you just for clarification, for me at least, relates to the issues you have just raised about the assessment process and you have been here today I think, haven't you, so you have heard –
A. Yes.
Q. – questions around that being. I do not think you were here at our last sitting?
A. No, I was not.
Q. You are probably aware that there were a number of questions put in a variety of different ways to the three Holmes Consulting Group witnesses about the assessment process and then this morning you may recall that I put to Mr John Hare questions around his existing

knowledge about the building and whether that would have had any relevance to the assessment had he been doing it. Remember I put that those –

A. I do.

Q. And I am just wondering whether the points that you just made a moment ago about assessment issues indicate any difference of view that you have about the extent to which existing knowledge of the Pyne Gould Guinness building should have had to an assessment of it post the September and then the Boxing Day earthquakes?

A. I find that very difficult to answer. First, the assessment, the visual assessment of a building is not easy to do. As I have mentioned in one of my slides there, it depends on how good the building is at the start and I do not think that that information is as widely known by the general engineering community as it is perhaps to academics who have spent their lives testing structures in laboratories, and I find it very difficult to say that John Hare or any other member of Holmes should have recognised that because it had some sub-standard details in it, it would respond in a different fashion and that the cracks might mean something more significant than it would in another sort of building. So I suspect that what basically has been said is that they would not have come to a different conclusion about whether the structure had diminished capacity and I am not sure that even an invasive investigation afterwards or an investigation based on the knowledge of the plans would necessarily come up with that information. So it is a very difficult one to answer.

Q. Are you making the more general point in your, what you said previously, that taking it away from what actually happened on the PGC building that in general before an assessment is done you need to know something about that building if that assessment is going to be really meaningful?

A. No I am not fully saying that. I am saying that it depends on the building a little bit. I think that it would be of great value to be able to, after an earthquake of this sort of nature, if you are investigating a particular building to go to the territorial authority and say, what have you got on this that I should know about? I am just doing a visual inspection of it at the moment. Tell me its age, are there any critical structural weaknesses identified in your files? And if they have something or rather which is half a paragraph that would be I think extremely useful information. As a community we are going to learn a huge amount of extremely valuable information from this earthquake. It is a tragic way to get that information but there will be major changes I think occurring, not just here but overseas, in terms of the way that we assess buildings and we view them and the way in which information is recorded and I think that we have to do everything that we can to help that process.

Q. Thank you Professor Priestley.

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CROSS-EXAMINATION: MR ELLIOTT

Q. Professor, you make the comment about eye-witness accounts indicating increased liveliness of the building post-September 4, and you can appreciate that that sort of thing to a layperson may indicate that the structural integrity has been compromised in some way. Can you just explain is it possible for there to be increased liveliness in a building without the structural integrity having been compromised?

A. Yes I think certainly it can. I mean even the mere process of developing flexural cracking in an element which was previous uncracked, for example, a beam might have been uncracked under a low level of earthquake, develop some cracking. There will be some additional liveliness associated with that so the cracking would not indicate a reduction in the capacity of the structure itself but it is just indicating a change in state, if you like, of it. There's also a lot of unknowns. What has the earthquake done to the ground in the region around it? Has there been some sort of change in the ground in terms of its stiffness which may influence the response of the building, the liveliness of the building. And the third thing I think associated with that I'm sure anyone who has been in Christchurch after or during these earthquakes itself developed a greatly increased sensitivity to earthquakes. I know certainly I did. The aftershocks seemed to be much bigger than I would expect from my knowledge of them. I seem to be noticing if a truck was going by on the road, then it seemed like an earthquake and our senses are heightened towards this so the apparent increase in liveliness may have occurred or it may not have occurred but we can't be certain of it.

Q. There's been some evidence in this hearing about a particular crack that was seen to run on level 1 around the shear core at about one and a half metres from the floor around much of the east, north and west of the shear core and that the crack was visible on both sides of the shear core. Are you able to indicate what the significance of that sort of crack might have been?

A. I wasn't aware of this crack I should say for a start. I haven't seen it in the reports of the damage but you're saying this was about a metre above the floor level between levels 1 and 2. Is that correct?

Q. At level 1 between levels 1 and 2 and I think I'm right in saying a metre and a half above floor level running right around east to north and west?

A. And it was essentially horizontal?

Q. Yes.

A. Right and it was, if I am correct in hearing you, on the north, south and east faces but not on the west face?

Q. North, east and west but not the south.

A. Okay, okay, that's less likely. It would seem to indicate response in the north-south direction with a flexural response with a crack. If it is a continuous crack at that level. I guess there is two possibilities – one is a flexural crack that has occurred during the September the 4th earthquake. Cracking is expected but I wouldn't have expected to see it at that level for reasons which have been discussed here, that we would expect to see cracking on a construction joint which we would anticipate was at the floor level and because of the high tension strength of the concrete relative to the tension capacity of the reinforcement we would not really expect to see additional ones but I wonder whether there was a cold joint in the pour at that level. In other words, during construction concrete was placed in the shutters that would have been used in construction up to that level but then before the next load of concrete was placed perhaps the concrete had hardened and so it created a weak joint and this does happen sometimes so either of those are possible explanations I think.

Q. Is that the sort of thing one could test by doing some concrete sampling of the shear core at level 1 post collapse?

A. Post collapse, well, um, it would depend on the condition of the wall and whether you could get into it. I believe it would be if it was a construction joint. I think it would be apparent. You would be able to see a difference in the concrete above and below. Normally when you get a cold joint like that it's rather apparent.

Q. If you'd seen that sort of crack running around the shear core at level 1, would you have considered that it was possible that the reinforcing steel had fractured and that some further investigations should be undertaken?

A. I'd be surprised in the, it indicates response in the north-south direction which admittedly is the direction of strongest shaking in the September 4th earthquake. I would believe on the basis of the analyses that had been done but it seems unlikely. The response from the time history analyses that have been done here indicate that even in the Darfield earthquake the response was fundamentally in the east-west direction and that's not because the shaking was higher in that direction. It's just that the structural capacity is much lower in that direction than it is in the north-south direction. It could well be a crack developed in the north-south direction due to shaking in that direction but I would be surprised if it would have been sufficient to cause fracture of the reinforcement in that direction. So, as it was, we predict with the slightly lower shaking in the east-west direction than in the north-south direction that there would not be fracture of the reinforcement in the east or west walls and I would think that that would make it less likely that there would be a fracture of the reinforcement in the north walls. I can't rule it out completely though.

Q. And just finally you may not be in a position to answer this but do you think the scope for more training as part of an Engineering Degree or Post-Graduate Degree for engineers to learn more about the way to diagnose what cracking might mean in an earthquake damaged building?

A. I think there's definite scope for doing that and I would very much expect that this will happen. The engineering profession is well aware that there are short-falls and short-comings in what we're doing at the moment. I suspect that what will happen is that there will be professional seminars run by the Structural Engineers Association or something of this sort of nature. That will happen and I suspect also more emphasis will be given in academic courses as well.

Q. Are you aware of any existing training packages or courses around the world somewhere that could be adapted for that purpose right away?

A. There are various courses which I don't know are at the level which we require. There is a lot of information in the United States in documents based on the learning from earthquakes series organised by the Earthquake Engineering Research Institute and there is also information here as well but I think that it needs to be up-dated and really this earthquake and the damage to buildings in these earthquakes provides a very valuable tool which earthquake engineers throughout the world are looking to to be able to help up-grade such aspects.

CROSS-EXAMINATION: MR HERRON – NIL

COMMISSION ADJOURNS: 4.56 PM