Canterbury Earthquakes Royal Commission

Te Komihana Rūwhenua a te Karauna

UNDER	**THE COMMISSIONS OF INQUIRY ACT 1908**
IN THE MATTER OF	**CANTERBURY EARTHQUAKES ROYAL COMMISSION**

Before: The Honourable Justice M Cooper
Judge of the High Court of New Zealand

Sir Ron Carter
Commissioner

Associate Professor Richard Fenwick
Commissioner

Appearances: M Zarifeh as Counsel Assisting

Helen Smith and Katie Benson of Duncan Cotterill for Structex

John Hannan of DLA Phillips Fox for Holmes Consulting Group

Duncan Laing Simpson Grierson for Christchurch City Council

Nadine Daines, in-house counsel for Christchurch City Council

TRANSCRIPT OF HEARINGS RE HOTEL GRAND CHANCELLOR BUILDING

COMMENCING ON 17 JANUARY 2012 AT CHRISTCHURCH

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0930

OPENING STATEMENT OF MR ZARIFEH:

Introduction

Over the next two days the Royal Commission will examine the failure of the Hotel Grand Chancellor – a building that has become a landmark in Christchurch since the February earthquake.

The Hotel Grand Chancellor was a 21 storey high-rise reinforced concrete building located in the Christchurch CBD. Built between 1985 and 1988, it was one of the tallest buildings in Christchurch at the time of its construction and as at 22 February 2011. The building had a 15 floor upper tower containing hotel accommodation above 12 half floors comprising car parking.

In addition to its height, the building had, unusually, both vertical and horizontal irregularity. The vertical irregularity arose from the fact that he upper tower relied on reinforced concrete frames for its seismic resistance, while the lower tower relied on reinforced concrete shear walls. The horizontal irregularity arose from the fact that the eastern side of the building was cantilevered out over an existing right-of-way.

Failure of the Building

Engineering assessments carried out following the September 2010 earthquake did not reveal any significant structural damage to the building.

The Hotel was in full use when the 22 February earthquake occurred. In that earthquake the building suffered a major structural failure, in particular the rupture of a shear wall in the south-east corner of the building. As a result that corner of the building dropped by approximately .8m and deflected horizontally approximately 1.3 metres at the top of the building.

This major movement induced other damage, including column failure, beam yielding, pre-cast panel dislodgement and notably the collapse of most of the stairs. However there was sufficient resilience within the overall structure of the building to halt the collapse. There were no fatalities or serious physical injuries.

Terms of Reference

The Terms of Reference, as they relate to this building, are set out on the Royal Commission's website. The main issues the Commission will have to consider in relation to this building are:

Why the building failed in the February earthquake.
The nature of the land associated with the building and how it was affected by the Canterbury earthquakes.
Whether there were particular features of the building that contributed to its failure, including the design and construction of the building.
Whether the building as originally designed and constructed complied with earthquake/risk and other legal and best practice requirements.
The nature and effect of any assessments of the building following the September earthquake and the Boxing Day aftershock.

Failure of the Building

In relation to why the building failed, there appears to be a substantial level of agreement amongst the experts.

The Grand Chancellor Hotel contained a critical structural vulnerability, namely the fact that the capacity of the shear wall in the south-east corner of the building (D5-6) could be exceeded by the demand actions that could be expected during a code-level earthquake shaking to the extent that a brittle and abrupt failure could occur.

The 22 February aftershock induced actions within that wall that exceeded its capacity and caused failure and partial collapse of that wall.

The factors that contributed to that critical vulnerability were:

1. The horizontal irregularity. This resulted in a disproportionately large contributing area being supported in the south-east corner shear wall (D5-6). The initial design of the building was advanced on the basis that the foundations, columns and walls could be constructed along (and within) Tattersalls Lane which was the right-of-way on the eastern boundary of the property. Construction was reasonably well advanced in the western half of the building site when legal action effectively prevented construction of any structure within that right-of-way. That reduced the footprint width of the building and required a structural redesign and the result was a cantilever adding to the structural irregularity of the building.

2. The vertical irregularity, as I have said, was the framed structure on top of a shear wall podium with transfer beams at the interface of those two.

3. The extremely high axial or vertical wall actions.

4. The fact that the D5-6 shear wall was too slender for the levels of axial load.

5. The fact that there was insufficient confinement (by way of reinforcing steel) at the base of that wall.

It seems that in all those factors the slenderness of the wall and the low level of reinforcement confinement were probably the most significant factors leading to the wall's failure.

Compliance

The Commission will hear evidence that indicates that the building as designed and permitted, did not comply with the standards that were in force in 1985-1988. In particular, in relation to this shear wall (D5-D6) in terms of its slenderness ratio and the degree of reinforcement confinement of that wall.

Evidence will be given that indicates that this may have been as a result of the need to re-design the building following the legal action so that it did not encroach on the right-of-way and an omission to re-calculate any resulting change in the seismic load.

The Christchurch City Council at the time of permitting the plans relied on a designer certificate signed by a principal of the structural engineering firm that designed the building.

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In my submission, this hearing will highlight important issues in the design and permitting of high-rise buildings particularly those that are irregular structures.

Whilst the building was damaged following the September earthquake there does not appear to have been any apparent significant structural damage. Further, it seems unlikely that the structural engineering inspections of the building following the September earthquake would, in the ordinary course, have highlighted potential problems with the structural design, in other words without an in-depth examination of the investigation of the building including an in-depth perusal of structural plans.

Turning to the witnesses that the Commission will hear from.

The first witness will be Adam Thornton. He is a structural engineer and the managing director of Dunning Thornton Consultants in Wellington with over 30 years' experience as a consulting structural engineer working primarily in New Zealand but also in the Asia Pacific region and particular experience in commercial and high-rise buildings, seismic engineering strengthening and refurbishment of heritage marine and earthquake-prone structures and the relocation of heavy structures. Mr Thornton and his firm Dunning Thornton were tasked with an analysis of the failure of the Hotel Grand Chancellor by the Department of Building and Housing in their examination of the building.

Secondly, the Commission will hear from Associate Professor Stefano Pampanin who was one of the expert panel that reviewed the Dunning Thornton report on the building's failure. Mr Pampanin is associate professor in structural design and earthquake engineering and the chair of structures and geotechnical cluster at the Department of Civil and Natural Resources Engineering at the University of Canterbury. He received a Masters in Structural Engineering at University of California at San Diego and a PhD in Earthquake Engineering from the Technical University of Milan and his research and professional activities in Italy and New Zealand have focused on the development and implementation of innovative solutions for the design of low-damage earthquake-resistant systems and the retrofit of existing structures.

Thirdly, the Commission will hear from Mr William Holmes who is an eminent structural engineer from San Francisco, with the firm Rutherford Chekene and the Commission has already heard from him in relation to the Pyne Gould Guinness building. He has been retained by the Royal Commission to review the Department of Building and Housing reports, the consultant report and the expert panels report and he will give evidence about his review of those reports.

The plan then is to follow those three witnesses, the evidence of those three witnesses with a panel discussion involving Messrs Thornton, Pampanin and Holmes and that should take place in the latter part of today.

Tomorrow when the hearing continues the Commission will hear from Stephen Martin who is the general manager of the Hotel Grand Chancellor and then from two structural engineers who carried out inspections of the building following the September earthquake, Garry Haverland from, now from Structex, and Andrew Lind who was then with Powell Fenwick consultants, as to his inspection in September. The Commission will then hear from John Hare who is –

JUSTICE COOPER:

Q. Both of those are inspections were following the September earthquake were they?

A. Yes, yes Sir.

Q. There wasn't one following the Boxing Day earthquake?

A. Yes Sir, my understanding is that Mr Lind certainly did two inspections in September and a further visit following Boxing Day.

Q. I see.

A. He will clarify exactly when Sir, the written material's not entirely clear on that but that is my understanding.

The last two witnesses, John Hare who is a structural engineer who the Commission has already heard from, a director with Holmes Consulting Group and he will give evidence of the design of the Hotel Grand Chancellor by Holmes Consulting Group or its predecessor in 1985 to 1987, and then lastly the Commission will hear from Stephen McCarthy who is the Environmental Policy and Approvals Manager Christchurch City Council who will give evidence of the permitting process between 1985 and 1988 and the current permitting procedures for similar buildings.

So if I can now move to the first witness and call Mr Adam Thornton, he has a power point presentation to give which will explain his report.

MR ZARIFEH CALLS

ADAM WILLIAM THORNTON (AFFIRMED)

Q. Mr Thornton, your full name Adam Thornton?
A. William.
Q. Adam William Thornton?
A. Yeah.
Q. Thank you, and you are the managing director of Dunning Thornton Consultants Structural Engineers in Wellington?
A. I am.
Q. And you heard me in my opening talk briefly about your experience. I think you have a Bachelor of Engineering in Honours from Canterbury University?
A. Correct, yep.
Q. And you are a Fellow of the Institute of Professional Engineers and a Chartered Professional Engineer to name but two –
A. Correct.
Q. – professional affiliations. You were tasked or Dunning Thornton was tasked with examining the failure of the Hotel Grand Chancellor for the Department of Building and Housing?
A. That's correct.
Q. And you have prepared a report on that which the Commission has a copy of and a copy on its website?
A. I have, we have.
Q. And I think you have prepared for today a power point presentation to explain in particular the failure of the building and which will cover the report that you have given to the Department of Building and Housing?
A. Yeah I have.
Q. Can I ask you please to take the floor and present it to us and you've got the slide already up there I think, started so I will let you take over and if there are questions from the Commission during your presentation are you happy to take them as you go?
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A. Yes, very happy to be interrupted. Okay this is just a brief summary of the issues that I'll cover during my presentation, a bit about the description of the building, the nature of the failure, the structural actions involved, a bit about the wall D5 to 6, that's, that means it's on grid D between grids 5 and 6 of the, of the building's grid lines. The seismicity, a bit about the stairs and other damage and then there's some questions that we if you like raised and answered ourselves within the report and some recommendations and then a few issues that have arisen during the review particularly from Mr Holmes.
So this is a plan of where the building is that's arranged in north-south direction and the building at the area within the red square is the actual structure that we're talking about. The green bit lined in green is the adjacent car parking building. They are separated seismically and were built at slightly different times and you can see on the lower plan where it is in relation to the city square and perhaps of interest adj– reasonably adjacent is the CTV building in terms of where it was in the city. Just, the carpark building is the same level as a podium that is in front of the tower coming out to Cashel Street so this little portion here on the southern side of the tower is a podium level. That is part of the main building that we're talking about but it is separate but at the same level as the adjacent carpark. And I –

JUSTICE COOPER:

Q. Can I just ask could you go back to the previous slide, the photograph of the, which I assumed was from Cashel Street, I'm just, I'm not quite following –
A. Oh yes that is –
Q. - the part you're making about the podium?
A. Right well that is, I will explain that a bit -
Q. Yes.

A. - further on in another couple of slides, but yeah, this is a view from the south from Cashel Street and so this portion here that you can see there that is a podium that is part of the structure.
Q. Oh I see.
A. Okay but, and I'll show you where the seismic gap is in a moment if I may.
Q. For some reason I'd assumed that was part of the carpark but that's wrong is it?
A. Yes.
Q. I see.
A. And I do make that clear in the –
Q. Yes.
A. – following slides.
Q. Yes all right thank you.
A. All right so this is a view if you like, a section through the building describing the structure and perhaps some of the irregularities that are within it. Starting from the top is the where the accommodation part of the hotel and it's a framed building and the green portion down here is the lower portion which is a carpark half floor so there are 14 floors and quite a small gap but they are offset from north to south. This yellow portion is the adjacent carpark which is a separate seismic structure so there is a, there is a vertical irregularity occurs at this point where it goes from a framed structure to a shear wall structure. Down in this corner here and you can see it on the photo also looking through the same view is Tattersalls Lane so that's a right-of-way and we'll talk a bit more about that as we go through. And above that there's a feature of the building which we'll talk quite a bit about which are these transfer frames, transfer walls I should say and they, they actually support this portion of the green of, of the lower structure. They don't support the upper, upper part of the cantilever tower but they do support this and this, this blue line here in section is the wall that failed so that is if you like the fulcrum for this transfer beam which cantilevers across the top of

that wall to support through a hanging mechanism which is itself is quite interesting, the lower portion. There is a separation through here from this cantilever bay in the frame structure so that does not in fact load that transfer beam.

JUSTICE COOPER:

Q. And in the photograph on the right-hand side there, that area I was questioning you about before?
A. Yeah.
Q. That's not a carpark?
A. That is part of the car, it is a carpark in function.
Q. Yes.
A. But in structure is part of, of this building. There is a couple of levels at the top which are, which were part of the convention centre.
Q. Right.
A. Which straddled both podiums.
Q. All right, because that looked to me like a carpark?
A. Oh yes, very definitely it is a carpark but it is the same…
Q. It's part of the hotel proper?
A. Yeah. And in fact these two photos here perhaps illustrate that so you can see this line here, that actually is a seismic gap sort of running behind that sign you can see the old telltale two hangers on the, on the canopy down here.
Q. I see.
A. That portion there is if you like an appendage on this main structure whereas this here was the original, slightly older structure which is a carpark to the podium level of this level here and it, on top of that is the convention centre that spreads across both buildings and it is accessed through the hotel lifts. The, I - as I understand the history of the building was, it was, it was a somewhat speculative development. It was originally thought to be used as offices above the carpark. Once the, I think once the licence was granted for a casino in Christchurch it was, it

tried to if you like, the owners tried to obtain that licence. They promoted as such, it wasn't, they weren't successful in that and then they ended up turning the structure into a hotel with a convention centre across the top.

Q. Yes and once again the photograph in the slide now displayed on the left-hand side is the photo taken from Cashel Street is it?

A. It is from the south side.

Q. Yes.

A. And you can see the sort of lurch towards the what is towards the south and towards the east you can see there in the tower. As you, as you can on this photo too with the, this is the lean here. Now just on this photo here this is the, where the vertical irregularity occurs so below that point is the shear wall structure and above that is the, is the frame structure and on this vertical line down here but just at the, where the tower, below the tower is our wall that failed the same wall here.

Q. Yes. Thank you.

A. This, this transfer beam is, is in fact there.

Q. Just below the line of red dots?

A. Correct.

Q. Thank you.

A. There are a number of those beams going through the building from south to north.

Q. Yes.

A. All right so now we're looking at a plan effectively through the lower portion of, of the structure and the lower of these two plans, this one here was the structure as originally conceived and you can see the, that Tattersalls Lane right-of-way noted there and these walls here on the eastern side of the right-of-way. Now the developer attempted to get permission to put structure within the right-of-way and obviously the structure design is to proceed on that basis and this was the design that they came up with. Piling had already started on this side of the site when in fact the legal decision was that he could not found the building in the lane itself although he obviously had rights to the, air rights over the top of the lane and so the structure was then amended and redesigned to this form here. So what you see here is that the, all the structure, all the footprint that touches the ground is on this side of, of the lane. In fact I think that's, well in fact I know that that, this line along here is not actually the side, the edge of the right-of-way, it's somewhere further across about where that line there is but I think from a planning point of view they didn't minimise the cantilever just because of, to suit the planning within the building. If these walls were over here then the wall and the lines of structure above, this is my assumption, would not have suited the architectural planning for the use of the building's upper floor so I guess there was the opportunity to reduce the size of the cantilever slightly from a structural point of view but from a planning point of view where they put it, this line of structure here sort of made sense. I just, and it's worth just talking a bit about irregularity Irregularity can have quite a few forms and a number of them are exhibited in this building. A vertical irregularity is normally a change of stiffness or load path as you come down through a structure, whereas a horizontal irregularity is normally an offset between the centre of mass of the building and its centre of rigidity. The centre of rigidity is the point about which it would rotate in a torsional manner or the centre of where the lateral resisting systems are. So in this building we've had vertical irregularity of course occurring at that change between the upper tower and the lower tower. I would say also that the transfer beams form a vertical irregularity, so you have a change of stiffness in the building because of the transfer beams that occur at the top of the podium and when you look at this plan here where the outline of the tower, if you can follow my mouse, is of this area here, so there is also a bit of a – in terms of the gravity load reaction, where the load is reaction, that because of the cantilever bay, that there is a net cantilever action on the structure that it bears on the ground, so that's often like a third form of vertical irregularity. Then the horizontal irregularity at the lower structure, there is both north south and east west if you like, so in the north south direction you have a wall here where you don't have a corresponding wall at this end, so this puts the centre of rigidity somewhere to this side of this main wall. What that means is that in a rotational mode that the extremities of the building are this end and where this wall are perhaps –

JUSTICE COOPER:

Q. Can I just interrupt, can you just be a little bit more informative in the words you're using because we are creating a record here and whilst we can see now where you're pointing with the mouse, when we come to read the transcript of what you say, 'here' and 'there' may not mean as much as they do now. You have got a north arrow there so you use the points of the compass and we will all follow what you are talking about.

A. Okay, in terms of the outline of the tower, I do have a subsequent slide where I do have that outlined, but –

Q. But you are discussing matters now on the basis of the movements of the arrow so I want to capture this what you are telling us now in a way that will enable us to read it and understand it in a week's time or a month's time.

A. I quite understand. Yes, all right, so I'll just repeat that in the north south direction there is an eccentricity and the centre of rigidity will be to the north of this main central wall, the 'I' shaped shear wall and that can have the effect of increasing the displacements at the extremities that are furthest away from that, that centre of rigidity, so that at the wall D5 to 6 you might expect slightly greater displacements than certainly you would at the centre of rigidity. We also of course have a bigger eccentricity about the east west direction in that first of all the centre of mass will be half way across the building from east side of Tattersalls Lane through to the west side, that is to the seismic joint with the carpark, so it will be approximately half way across there whereas the centre of rigidity will be really the — determined by the locations of these walls which line in a north south direction. There are four walls over here and there are three walls here which are slightly – in fact these two are smaller than these ones so it will be further west than the geometric centre of the walls, somewhere in this region close to the western wing of the 'I' shear wall. So that is giving say a number of eccentricities. There is also eccentricity in the upper tower frame, horizontal eccentricity and I will show you that when we get to that slide. I would just note here that in this very conceptual drawing that the designer did at the point of time when the design was changed, he did show a little return on that wall there, and it's a natural thing for a structural designer to assume that he may need a return at that point because it's quite a short wall. In the event the final design did not contain such a return and one can only assume that he's satisfied himself that it was not required through analysis and design. It's easy to reflect now in retrospect that had there been such a return then the failure may well not have occurred.

Q. When you say it was not provided in the final design, have you seen a plan which lacks it, or are you saying that it was not provided in the building as built?

A. No, the only place where it occurs is on this diagram so that the construction drawings do not contain it. This is not the construction drawing, it is a page from within the calculations, engineering calculations.

Q. So on the plans that received a building permit, that return was not shown?

A. Correct, yes. I guess you can also imagine that this wall was right in the middle of the reception area for the hotel as you entered through the lobby doors off Cashel Street, it was there in front of you, so there's certainly – you might imagine that there was perhaps architectural pressure to minimise the size of that element. Right, now here is a plan that shows if you like the foundation plans, so this is the completed design. We're seeing here a wall which is highlighted, the wall that failed, and just for reference we now have gridlines and we refer to these quite often, so these are the alphabetical gridlines going from west to east, A, B, C, D is the line with our wall on it. E is the extremity on the far side of Tattersalls Lane and then from north to south we have walls on grid 11 on eight, and this wall straddles between five and six, so I'll come back to this, the grids as we go through because a lot of things are referenced from the grids. You can see that there are piles in fact in the Tattersalls Lane so they were used for some of the temporary support while the building was built, but obviously they had permission to do that part of it. A little bit about the foundations, the building was on – founded on piles. In this area of town there was not much – very little evidence at the surface of liquefaction and we've been unable to see any effects or suggestion of foundation failure on this building. Well here's this plan that perhaps I should have shown you before, which is the plan at the ground floor with the extent of the tower highlighted there in yellow. This area to the south of that, so between grids 1 to 5 and effectively between A and C is the area of that small podium which is attached to this main structure that I referred to earlier. And you can see there is actually at the ground floor structure does occupy the space between grid D and approximately 40 percent of the way across to grid E at the ground floor level, so that the right-of-way doesn't start at grid D, it starts part way across. And this then is level 2 and this is relatively typical up through the lower floors through the podium structure. So just describing some of those elements, the grey elements of course are the shear walls all of interest, I've highlighted there in red. The area within the blue box is the area which is effectively cantilevered, and this area here is supported on the line E grid by some transfer beams at the top of the podium, and I'll show you pictures of those as we move through, so this area of slab between effectively grid D and grid E is spanning between those two grids and is supported both at grid D where the shear walls go to ground, and it's supported from above on grid E. The structure, this suspended structure is an in situ flat slab so that there are effectively no beams and that's part of the reason why there aren't cantilevers at each floor, because they wanted to minimise the floor to floor depth within the carpark so you can get efficient structure, get more cars in and no ceiling, so it's an in situ relatively thick concrete, reinforced concrete supported directly on the walls and the columns which you can see a bit more faintly and again there are additional columns out on the podium portion of the structure. I just highlight this point here, I'll show you a photo of that, that there was some I would call secondary structure that was supporting the wall, so there was actually a wall along this line, not a structural wall but a – separating the lane from the foyer and there was obviously some structure in there and that structure was sufficient to break the back of this slab above. So there's a photo looking along that line, you see the shear wall, D5 to 6 there and there is some secondary structure in there that is – so this point has not dropped but when the failure occurred this wall and all this structure that we see here dropped by about 800 millimetres.

Q. The photo numbers, that number was the same as the number in your report –

A. It is.

Q. – to the Department of Building and Housing. Is that generally the case?

A. Yes, yes, most of these graphics are direct cuts from – but there are a few extra and I'll hopefully elaborate on those.

Q. Thank you

A. Yes, so this is along this line here. So this wall when it failed and it failed at ground level, dropped by about eight to 900 millimetres, so it really effectively all this area here went with it, because it is supported by that wall. This wall in a normal sense would support an area which extending from grid E back to, almost to grid C, and from grid 5 to half way between grid 6 and 7, so it's quite a large portion of the tower is supported on this shear wall grid 5 to 6. And here's a plan which is very similar to the previous one you saw, but it is up at levels 9 and 10. Just to be clear that through the podium the floors are numbered, this is nine, this is 10, so north of grid 8 is one floor level and south of grid 8 is another floor level, just because there are ramps within the structure. So they're not full floors, full floors you'd understand it would go eight, 10, 12. Now this is the level, two levels down effectively from the top of the podium and above this level are the transfer beams that I'll show you in a moment, but as this wall at D5 to 6 failed and dropped, load was transferred onto these adjacent columns here which are grids 5 and 6 and on C and B, and they, the tops of those columns were crushed to a certain extent and they failed and dropped by decreasing amounts compared to the shear wall drop. It's very fortunate that they did not collapse utterly those, because that would have brought down a large portion of the building. You can see along adjacent to grid A, there are additional walls there and they are walls that, like parallel walls within the carpark structure, so they mirrored if you like the shear walls in the carpark structure and in the western side of the main tower building, just obviously from planning points of view. One of the extra bits of damage that we observed within the carpark building was some strange horizontal cracks on the walls within the carpark at mid-floor height and it was quite difficult to initially understand what had happened there, but I'll explain a bit more fully as we go on, but essentially this floor level adjacent to the shear wall gave a large kick towards the west as the wall collapsed and the gap between these walls was, and is filled with high density polystyrene, so effectively it applied a pressure to these walls in adjacent buildings to a point where they – some cracking occurred in the walls.

Now we've moved up a level from there and this is the level where the transfer beams that I've described occur and this was not a carparking floor, it is a floor where the hotel offices were and the kitchens were, so obviously walls within these – where these yellow walls are shown on this plan would have been very counter-productive in a carpark floor because you can't drive cars through them or around them. Here you can nestle offices within, between the walls and so this is a place where they've chosen to put this structure that supports the cantilever bay, so you can see again the proportion is cantilevered in the blue box there. These transfer beams or walls that I've highlighted there span across to grid E to support this structure that is hanging below them and it occurs at five places. Four of them, the walls on five, six, eight and 11 are full height so they go, they actually link the slab above them and below them and that's quite significant because in an earthquake the floors want to move transversely relative to each other. We call that drift, now whereas these shear walls, the main shear walls that's their function and we expect that they will do drift. We're not really – the designers I don't believe anticipated that there would be the drift that would occur within these transfer beams. So of particular interest are the ones on grids 5 and 6, because they're the ones that are supported directly at their fulcrum point if you like by the shear wall, the grids 5 to 6, and as I said before I think these walls, these transfer beam walls effectively give you another form of vertical irregularity because they're an interruption to the natural vertical stiffness of the building. Just one note of interest which we'll come back to, but the wall on grid 8 almost failed at that point there and I will explain that, just so you can see where that is in plan.

Q. So that's the area above the arrow, localised damage to grid 8 transfer beam?

A. Correct. Now we've now moved to the plan on the floor immediately above that, so this is the top floor of the podium. It is built as one of the podium floors, it's an in situ slab floor. It's supported on the walls as the floors below it is but it is also, if you like, the base of the frame so you, I have superimposed these here, these brown rectangles are the columns of the seismic frames above.

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Q. You say it is the level immediately above but is labelled level 14. Was this one of those buildings where there is not a level 13?
A. I think that might be correct actually, I think unless there is a, no I think that is correct, but it –
Q. Well a previous, we have been discussing level 12?
A. Yes, yes, well certainly 14 is above level, is directly above level 12 and I don't think there is a 13 at the back, I was just, I'm not quite sure how the numbering goes on the northern part of the building.
So you can see there where the columns are coming down. Now there's a few things of significance to note here. First of all, this is the area, this corner here was supported by this wall. When that wall failed that vertical load had to transfer some, to somewhere else. It tended to transfer to this point and this point a –
Q. That is the intersection of grids 5 and C?
A. 5 and 6 on C, again that'd make clearer on some later slides. There is some transfer also on to the point 7D and for the hanging structure which is on grid E so there is this, all the structure below D and E is hung from a grid E structure, grid E line and there is a, I'll show you some pictures of that line but you can see that these two points here, that is the ends of the transfer beams on 5 and 6 at grid E they dropped because the wall that's supporting them dropped. That transferred load onto this point 8E and that is what initiated the failure in that wall you saw below at the end of that wall on grid E, sorry, grid 8.
Q. Just to make sure my colleagues understand this, but I must say I do not really understand how this structure is supported when it hangs below these beams. What is the mechanism by which they are –
A. I will, that will hopefully become clear to you Sir when I get to that slide.
Q. All right.
A. Yes, so, I just, while we are at this point here, one of the questions it is worth asking as well. This wall here failed but we have other walls in similar situations. Why didn't they fail? Now part of that is that when we consider this line around the frames above so well perhaps just to state

that the seismic frames above the podium level are, that is the frames that resist the earthquake loads are on grids 5 and 11 and A and D. Now in any seismic frame you get a vertical reaction at the end column of that frame so if we consider the frame on grid 5 as under earthquake loadings you get vertical actions in the column at grid 5D and at 5A so you can imagine as the wall, as the frame goes like that, lifts up one side and then the other side, pushes up and down so you get what we call vertical shears or vertical axial loads being developed in those columns. Now the same thing occurs for the frames in the other direction, so the frames on A and D. So the frame like this you have biaxial or concurrent axial actions in the corner columns so that is in the corner of column at 5D, at 5A, at 11A and 11D. So that is putting additional load on top of our wall there at that point, so it's just worth remembering that as we go through and again just remembering those grids D, A, 5 and 11. Now we move to a typical tower plan. This is the only tower plan I have put in that they are really, they can be really very similar until you get to the top portion of the building which is not particularly interesting. I have highlighted there the wall below. Of course the wall is not there, that's something I've drawn in, that's where it is in relation to the frame above and it just illustrates a bit better the seismic frames are the brown ones and you will see the beams running through between them. Now our other frames, because once we get up to here it's a different floor system. There are beams on each of the numerical grids, well on 6 and 7 and 9 and 10 as well as the seismic frames on 5 and 11. These frames support the floor and there are columns that support the beams. They were not intended or not relied upon by the designers to take any of the seismic loadings, only to carry the gravity floor loadings. The floor itself is a pre-cast floor which commonly described as a rib and infill system so there were pre-cast concrete ribs in the order of sort of 200 square. They span between the beams and then between the ribs themselves are at 900 millimetre centres. They have a thin bit of timber laid over them to support about

80 millimetres of concrete, 80 or 90 millimetre of concrete which forms the concrete slab which is poured in insitu. So it's a lighter, much lighter form of flooring than the floor in the car park but it is deeper because of the depth of the beam that is required so within obviously hotel accommodation there is going to be a ceiling, there's going to be air conditioning so they can use the space between the beams and above the ceiling for running those services and of course the lighter floor much reduces both the foundation load and the seismic load. And this here you can perhaps just again illustrate the horizontal irregularity that occurs in this direction so about in east-west direction the centre of rigidity would be effectively in the centre of the seismic frames so that is between, midway between 7 and 9 and probably midway between grid D and A, not between E and A, whereas the centre of mass will be between, approximately centred between E and A so there is an offset there which when you have a horizontal irregularity it will add seismic loads to one side of the building rather than the other so that in fact the seismic loads on grid D would typically be assumed to be higher than the ones on grid A.

Now this, the cantilever bay between D and E in this upper tower section, each floor cantilevers out so the beams on the numerical grids 5, 6, 7, 9, 10 and 11 all cantilever out and support that bay. It is not a simple cantilever because there is a column on the grid A grids, there are columns there and those, if you like, react with the beams to get some what I would call horizontal portal action, not particularly important but it does reduce the size of the cantilever beams. There is also a beam on grid E itself and that forms a frame on that line which again was not intended to be to act as a seismic resisting element but will attract loads as seismic actions occur. The same could be said for the other gravity frames that I have mentioned before although they are not relied upon to carry seismic load they will attract seismic load just from a compatibility they are forced into the same displacement and so the beams and the columns that support them do effectively feel seismic actions. And all round the building in fact in the lower structure as well there are pre, pre-cast concrete elements. They are sort of on the order of 100 millimetres to 150 millimetres thick. They are the cladding of the building, they support the window frames and they are supported off the structure but detailed, ie connected in such a way that they don't take lateral loads that they can, are suspended much like a picture frame but with, on the edge of the building, on the perimeter of the building.

Right we've got some sections now just to help illustrate and I'm looking now at the grid D frame so this is the frame that runs north-south. At the south end it's supported on the wall in question and you can see it here in cross-section. Now this cross-section's really showing just the upper part of the frame, it's not showing the full extent of the podium shear walls. Again our shear wall of interest is shown in red and then you can also see the adjacent shear walls that lie behind it along grid D and you can see in section there the yellow transfer beams. As I explained four of them on grids 5, 6, 8 and 11 are full depth. They go between the floors whereas the one on grid 10 which in fact is over the kitchens does not. Now when, when the wall on – failed and dropped this is a very simple diagram what happened there so between 5 and 6 moved vertically downwards. This frame between 6 and 7 then was put into extreme actions and severe hinging occurred at each end of that frame.

And here's some pictures of that extreme beam hinging. You can see here within these photos where the beams had to go extreme rotation. This is much more than would be the normal demand you would expect to be made of a beam frame. Now we're looking at the frame on line 5, so this is that northern, sorry the southern-most frame facing Cashel Street. This is actually a picture looking reverse of those early photographs so it's looking from the north towards the south inside out if you like. And again we have a frame, the seismic frame is running between grids A and D, it does extent to E because that's the cantilever portion and at the bottom between E and C you can see the transfer beam wall which does not support the columns on grid E above that level but the blue, blue line there is, represents the hanger which I will explain further. This is the, one of the elements that supports that lower bit of structure on, between grids E and D. And then the red line is our shear wall on grid D 5 to 6. And these are other, these brown elements are columns on that line in the lower structure which were again only intended to be as gravity structure. So again just looking at this frame here under seismic actions the column was on grid A and D experience vertical increased axial vertical loads as during the seismic action so while one would go into compression the other one would be in tension and vice versa as the shaking oscillates. And again on the right side of the picture is a simple diagram of what happened when that wall on D 5 to 6 dropped. It really caused that whole frame to lurch and lean towards the east.

Now here's a couple of photos similar to the ones that we saw earlier but they are in fact during the, they were taken during the construction, the original construction of the building and it just, just illustrates there a couple of things. First of all you can see on the eastern side of the building in both photographs it's annotated on the right-hand photograph that the upper columns are not supported by the structure below there is a gap there. You can see our wall, our transfer beam here. This is the one on grid 5 and in this photo here you can see the rather slender looking wall, the one that failed, the wall on grid D5 so that is spanning from ground up to the underside of the first floor. Now I just realised that in fact I didn't finish my explanation on the, why this wall failed and the other ones didn't so I might just go back to that at this point.

JUSTICE COOPER:

Q. So you're going back to a document which has got our number 0046.13 on it, not on the screen as displayed but that's the level 14 diagram?

A. Yes so the question we're asking is why did the wall D5 to 6 fail where the other walls which are in similar locations, ie the walls underneath the corner columns of the frames, these are the columns at 5A, 11A and at

D11, why did they initiate failure in those walls? Well there is a number of reasons for that. The first is that the walls on the eastern side had a much larger contributing area. So the contributing area for the walls on the eastern side effectively goes from grid E as I said to grid C and from 5 to 6 and a half and at the northern end from 11 to 9 and a half, so a very large area of contributing area. Now the equivalent contributing area on the western side is only a fraction of that. It's only half way between A and B and half way between 5 and 6 so it's a tiny little area like this compared to a much larger area there so from a, from a gravity load point of view the amount of floor it's supporting is much less. The second issue is that from a slenderness point of view the walls at the back of the, or the north side of the building because they were going up sort of half a floor height at a time in the ground floor foyer space this, the height of this wall unrestrained that is from the ground floor to first floor is about five metres whereas back here it's only about three and a half metres so that –

JUSTICE COOPER:

Q. So that's the shear wall –

A. The shear walls between 10 and 11.

Q. Yes.

A. Have a much, a considerably less clear height. They also have less load on them. A third thing is that our wall of interest is a very short wall and has no returns on it. You'll remember I said that the designer had doodled in a return at that wall in his concept but it didn't follow through into full design. This wall here does have a full return so as the extreme load comes on that column at 11D this return wall here will share, this return portion of that wall along grid 11 will share that, that very high axial load and will also prevent the end of that column, of that wall from buckling about its weak axis so that's about it's out of plane the thin direction. There is no such column providing or return providing restraint at grid 5D. Now the other issue that is worth considering is that at, and you can say that we have similar actions at grid, at the column, sorry shear wall on 5A but of course that wall as well as having, it does have the extra height but it does have a much lesser intervening area and it is twice the length so it has again more area to resist the axial loads. So that's why when you look at it from back at perhaps in retrospect that if any of the walls are going to fail it is logical it is this wall at the D5-6 is the one that is most vulnerable. There are other reasons that add to that vulnerability that I will explain later on as well.

Q. Was there a reason for not providing a return wall for the shear wall D5–D6?

A. As I said it's total speculation. There may have been pressure from the architectural designer to minimise the effect within the hotel lobby but through the calculations that the designer took I think he satisfied himself, unfortunately incorrectly, but he did satisfy himself that it was okay as it is, or as it was – drawn and built.

Q. So had there been a return wall there, might it have prevented the building being damaged as it was?

A. Yes I think, well if it was adequately done. It helps the wall in two respects – one is that it braces it so it stops that outer plane buckling but also it reduces the high axial stresses on the blade point of the wall. We'll come onto this a bit later on but when you've got, if you like, a relatively small column with a high axial load on it and when it's pushed over by the earthquake all the load is concentrated into one end of the wall and so you can imagine with quite a thin blade shaped wall that the stresses at one end of that wall get extremely high. If you have a return on it those stresses can be spread along the return and that was the advantage that the wall at the other end at E11 had.

Right I'll go back then. So we've seen that and we've seen that. And here's just a couple of other pictures there that illustrate some things we're talking about. In particular here you can see, photo 4 is actually showing the grid 8 frame facade – that's the eastern facade. This is not one of the seismic frames but really what happened on that frame models what happened on the grid D frame and you recall I showed the photos of the extreme hinging that occurred at the ends of those bends.

Q. Just point to grid 8 again for me please.

A. Sorry, this is grid E and grid 8 is this line here so that point did not drop, grid 8 did not drop. Grids 5 and 6 did. What you're actually looking at there are not in fact beams. They are the pre-cast panels but effectively they are mimicking the beams inside. And then when you look at photo 5 you can see some damage so again what happened is that the shear wall on D5-6 that dropped as we have talked about, the columns, the loads spread to the column beside it which was at D4 and those columns got crunched at the underside of level 12 and that dropped, and so that drop - those levels 12-14, relative to the structure. Now in the podium portion of this building that did not drop so you've got this sort of big mauling that's gone on here, damaging the pre-cast panels. Now some early observers they made the assumption that, perhaps you did Sir, that this was a separate structure and that pounding had occurred at that point. That's not what happened at all. It's the fact that there was a vertical displacement of one part of the building and not of the rest and so that caused some deformation in the facade.

Now I am moving on to talk a bit about the transfer beams and the actions there and the failure so really just illustrating what we've already talked about. This shows the two transfer beams on 5 and 6, so 5 is the external southern frame of the tower and 6 is the one parallel to it, but internally by a bay. You can see there and I've used the same annotation - the pink wall is our shear wall of interest, the blue are the tension hangers and I still am coming on to those slides that describe that frame on grid E and to the left at grids you can just see the grids B and C. This is where that extreme damage occurred to the top of these columns as the load transferred from grid D across to grid C and you can see there quite clearly it's a bit like, if you like, a see-saw. This is the point of pivot and it's carrying a big load here so the net effect is there's additional load coming down on this wall from the load hanging below and there's a bit of a net contributing uplift load which is reducing the load on this column, that is the column on grid C. Of course there is also additional column on grid D from the frame above because that column is supporting cantilever frames that cantilever out, the upper structure out towards grid E.

I'll just show you this slide here which is some damage we observed in the structure and I'll explain that a bit further on but as this wall dropped, as this wall failed, this end of the wall dropped and I'll describe it like this, to start with the wall was supported vertically in this manner so using the column on grid D and C was supporting. At the point where this column support was removed there was nothing supporting that wall and it tended to drop and it is resisted through the actions of these slabs and I've got some little drawings that illustrate that. That's just to remind you where we are just talking about, we were just looking at elevations of that, of these frames on 5 and 6 and I'm going to step over now to talk about the hanging wall on line E. I will come back to that other issue, apologies. So this is an elevation of the lower portion of grid E so this is level 14 which is the top of the podium and these are the various floors so this is level 2, 2, 4, 6, 8, 10, 12 and 14. And the yellow lines are the eastern end of the transfer beams and you can see there are the five, sorry, four on grids 5, 6, 8 and 11 which are full height and you're absolutely correct Sir. There is no level 13. It goes from 11 to 14. There is the half height beam over the kitchens and on grid E there are two additional columns there that act as traditional columns so if you like the blue lines are hangers so they are in tension. They are supporting all the floors below. So let's just step back. We've got a floor slab which was ramped and it's spanned from grid D across to grid E so from those shear walls across to the eastern facade. There is a beam along that edge to support that edge of the slab. That beam and the slab edge is supported at each floor by each of these vertical elements, the hangers and the columns. Now beneath each of the walls so the transfer beam walls is a vertical tension hanger. There are a couple of other supports to the floors which act as additional columns so if you consider a bit of load that is at grid 9 it enters this column, it goes all the way down the column to this hanging transfer beam so that's another wall which is at level 2. It's a wall beam not again, sorry, the floor level is another four or five metres below that, so the load comes down into this beam and is then supported by these hangers. Now you recall that grids 5 and 6 in that southeast corner they dropped because they were supported by the wall and so it caused these beams here to deform considerably and you have seen photos of that and you have seen the, above the pictures of the frames the same thing happened. You got, so the purple, these purple marks represent the extreme hingeing that occurred. That in fact put this extra load on this column and on this hanging beam and that's that where we had the extreme damage in the transfer beam on grid 8 and you'll see a photo of that shortly. This is just a picture of one of those hinges on the beam on grid 8 in the car park so what we're seeing here at photo 14 is, that's that column at the, at grid, I'll just give you the grid number, at grid 7, looking towards grid 6.

JUSTICE COOPER:

Q. This legend on that photo says beam 6 to 7?
A. Yes.
Q. Is that right?
A. So the column on the left-hand side of the photo is grid 7.
Q. This is grid 7.
A. Okay so if you look at this, the previous diagram you can see where that photo is taken, it is shown there.

COMMISSIONER FENWICK:

Q. That shows plain round strips, looks like plain round strips, lapped in cover concrete. Was that typical?
A. I think actually that those, it wasn't typical that was just where the beams splayed vertically because of the ramping so there was stirrups. These are the transverse reinforcing that go round the main bars

sometimes when the member is prismatic and not parallel but it's actually sloping that they would tend to lap because of the change of the depth of the member. That's not typical.

Q. But you say it is also not acceptable by current –

A. Well, it certainly is not, not desirable or acceptable to lap within a hinge zone. This was not anticipated to be a seismic frame but of course I guess certainly in retrospect we would say that if there's a possibility that hingeing could occur then you should also not do that sort of practice in this area.

The next photo is actually on a similar line and I would say is far more serious. So this is the elevation of grid 8, so this is sort of and shows the area of damage that I showed you in the plan. So in this elevation you are seeing an elevation of the main I-shaped shear wall in the middle of the building, so it is on grid 8 and it extends from B to D. It's very large, it's the main structural element and you can see the vehicle ramps between grids D and E and at the top, so in the zone where the kitchens and the hotel offices were. So obviously cars aren't required to drive through there and instead is the transfer wall beam and at the end of that, at grid E is the hanger which then supports the lower transfer beam which you saw a couple of photos ago, and you can see I have highlighted there where the damage occurred and you can see that in the photo in the right-hand corner. Now, what is, I do say I have found unusual is that it is difficult to see in a quality of print that you have got but this, this beam has stirrups, that's the confining the main reinforcing and they are lapped and you can see there has been within the report you haven't got it here there's a close-up picture that shows that there's actually been some slippage within those stirrups so you can imagine this element here, this enormous force down in this corner of the beam, at the bottom of the beam adjacent to grid E as the tension hanger is pulling down on it and it's tried to split this beam as it changes its depth, having that return in the beam I guess you could also say is not ideal. Ideally you would take the reinforcing from the hanger right to the top of the wall so then it can distribute its load evenly into the beam. Here it's been pulling on the bottom half of the beam which has tried to pull the beam apart, and it didn't fail but I guess you have to say that it's, if the loads, it got worse that it may have.

JUSTICE COOPER:

Q. So you still have not explained, perhaps you still going to come onto it how the hanger is fixed to the beam? That is my layperson's question. I am not understanding that.

A. Well I'll take you back to this here so we're looking, the previous picture we were just looking at is a section –

Q. Just let me read for the record this we are back to the diagram hanging wall line E which is 46.22 in our numbering system?

A. Yes. And so the previous picture that you, we looked at was looking towards the north along this line between 8 and 7, so it was looking at this, so this is the tension hanger. It's, I mean, it's for all intents and purposes is a concrete column but it's acting in a tension rather than compression and it is holding, it's taking loads from this lower beam. This beam is gathering loads from the two if you like pink columns which are traditional, more traditional columns, they are gathering load from the floors taking them down to that transfer beam and then this beam is spanning from this hanger, the hanger on grid 10 to the hanger on grid 8 and it cantilevers out to support the loads on grid 7. Now so within this hanger there is vertical reinforcing rods. They are buried deep inside this lower beam and the reinforcing rods come up. tThey have staggered the vertical laps within that so which is good practice and then those vertical bars are anchored into the end of the transfer beam on grid 8.

Q. So it is sitting on the transfer beam?

A. Well it is, it's built into the end of it if you like. If I go forward to that picture here again, so it's – look I've shown it -

1100

JUSTICE COOPER:

Q. So we know on 46.24, yes.

A. Back on the transfer being grid 8 picture, that, I've just shown the colour stopping down the side of the beam, but the reinforcing within that tension hanger goes up and is buried into this transfer beam on grid 8.

COMMISSIONER FENWICK:

Q. Mr Thornton, how was that reinforcement anchored? Was it anchored with a bend or was it welded on plates, or what, at that – I assume at that notch out of that beam it was anchored at that point was it in some way?

A. There are – the reinforcing is bent, look I'm just looking at the picture as you are, it's not something I'd looked at particularly. I mean that element itself, there's no sign of distress in terms of the hanger pulling away from the other side of the beam, but I can see from the drawing there that the reinforcing is embedded in the transfer beam. I do have a full set of the drawings here, I can have a slightly closer look at that if you'd like.

Q. But I wouldn't want to interrupt your flow now, perhaps if we can check that detail out later if you like.

A. Yes, I mean just looking at a slightly better quality of that, but there are simply bends of the vertical reinforcing which helps the anchorage of those vertical bars into the beam element.

Q. I've got one further question related to that, sorry, going back to your transfer beam on grid 8 which we were looking at before, that's – it doesn't help giving the number does it, it's number 24.

JUSTICE COOPER:

Q. Well it will help us later if you do.

A. Sorry, which –

COMMISSIONER FENWICK:

Q. It's transfer beam grid 8. The one that shows the failure of the lap stirrups.
A. This one?
Q. That one, yes. You see the stirrups were all on the outside of the beam there. What I'm really asking, would that meet current design concrete standards, or not?
A. Yes I believe so. Are you saying should've the beams sort of more legs?
Q. No, I'm saying should there have been some internal stirrups, not all outside stirrups?
A. I believe that it would meet because it's not a – you certainly wouldn't expect any – your purpose of putting them in the middle is to stop the main bars buckling, so, and to restrain the buckling, but certainly it would be a good practice to distribute the stress through the beam, but I don't believe that the code would have required in this scenario.
Q. Right, I'm not sure about it, I think it may, actually the current latest standard I think may require, depends on the spacing of bars and I suspect the spacing is getting wider.
A. Yes.
Q. Transverse and longitudinal, but I just – I'm not sure about that but I think that may be the case.
A. It's certainly the – it certainly is the case if it is definitely a seismic – if it categorised as a seismic beam.
Q. Sure.
A. I would need to check, I can do that later if you like.
Q. It's just a point we need to – I mean if they had been internal stirrups there, clearly this distress would have been greatly reduced?
A. Yes, because it would have got a more effective –
Q. Good trans –
A. - lap stresses, yes.

Q. So it's something we should need to look for and check for future design, thank you.
A. So I'm just going to come back to this picture I showed you before, just to describe –

JUSTICE COOPER:
Q. This is our number 46.25?
A. Yes, and I'm just going to show you some pictures, which I apologise I'm not very well annotated, I just drew them yesterday, but this is if you like it's just a part of this drawing we're just looking at, the actions on the transfer beam and the column, the shear wall below it and the adjacent columns. So very simply and this is showing some of the loadings on it, so that you have – this is an elevation of the beams on grid 5 within the podium structure. So this is our wall of interest on D5 to 6, these are columns which were taking gravity axial loads. This is our transverse beam and so it's got a load hanging below it, so this is from the hangers, and it's got a big load above it which is coming from the corner of the seismic frames above, and smaller loads over here, and so you've got a big vertical reaction in the foundation down underneath that column, and not a net tension but a reduced vertical load, so in a normal situation that's the loading on that section. Under earthquake conditions a couple of things happen. One is that the load directly above the wall gets larger and I will explain that as we go on. There is also a shear applied, relative shear applied between the two floors so these floors of course we're looking at are 14 and 12. That induces this diagonal yellow arrow which is as I explained before, all the floors want to displace relative to one another under the effects of lateral loads. That induces in this case a diagonal strut which indicates, in fact induces additional vertical load on the wall at the five to six, and that's where a load which I don't believe was anticipated by the designer, original designer. That of course led to the very high loads at the bottom of the wall and that's the bit that failed and when support was taken away from the bottom of the

wall through the failure then two things happened. One is that the loads transferred back to the adjacent columns, these are purple blobs illustrating that, and of course they failed themselves to a certain extent, and as I described before, this transfer beam for a while became supported by a horizontal couple provided by the two floors, the floor at level 14 and at 12, and it's that horizontal couple that formed a kick towards the west that did the damage to the adjacent carpark shear walls that I mentioned earlier. I apologise for not having got more annotation on those but it's just – really just to describe that motion where they're supported on two vertical points and then it lost its fulcrum and so the moment had to be carried for a while by a slab mechanism, a slab couple.

I've got some photos there of those, what were those purple blobs on a previous slide and these are of the columns at level 10 on lines 5 and 6, and as you can see they – these are the columns that yielded and but did not fail utterly which we are to be thankful for, but it did drop those columns. Those columns lowered by the order of sort of 500 millimetres and here's a close-up of one of the other ones, so in the days immediately after the February earthquake was, these were viewed with great alarm and were the source of the suggestions that the building could topple over and within a quite a short time they were interim strengthening was put into support these, so they're no longer like this, they're bound up with steel and a lot of concrete. Generally you have to say that they performed pretty well because they were certainly never anticipated they would get loads anywhere need as like what happened.

Right, so I'm going to come to a few, just describing some of the mechanisms and the derivation of the loads on wall, particularly the axial vertical load actions on a wall at D 5–6. So on the left-hand side there we've got a picture which shows in conceptually that frame on six and five and I haven't got the right number of upper floors but this is the, the black lines represent the seismic frame above the podium, the yellow is the cantilever frame between grids D and E. These, the red rectangle is our transfer wall beam between levels 12 and 14. The vertical purple lines are the columns below the podium which are not considered, were not considered to be part of the seismic frame because the seismic loads in that portion of the building are resisted by the shear walls. This grey line represents our shear wall on D and that you can see where the failure occurred at the bottom of that and the green is the hanger action pulling down on those transfer, transfer beams. So, and if you like we should consider seismic action from shaking from the west towards the east so the building has been shunted towards the east. In the upper frames that induces bending moments in the perimeter frame so in particular the frame on grid line 5 and that induces shears at each end of the beam so, and in the internal beams, at the internal columns I should say like on grid 4 and 3 those shears cancel out so there's no net increase in the vertical load due to those, the seismic actions whereas at the end column of the frame and this is the end column the column here at 5, 5D there is a net vertical load and that accumulates as you go down the building so you add up all these little vertical arrows, they are seismic loads that increase the normal gravity loads which have been carried by that, that column. So at this point here at the, that column at the top of the podium we have the normal gravity loads which as we've said are, are large because of the large cantilever. That means it takes most of this, the bay between 4 and, sorry, between C and D as well as, as well as between D and E and we also have what we call the seismic over-strength shears so these are the shears that are derived through bending moments in the frame that arise during the seismic action.

JUSTICE COOPER:

Q. Where's E sorry?

A. This is E, this is D.

Q. Thank you.

A. Right so you can see there the vertical earthquake actions and there's also the normal vertical reactions. Then when we get down to the transfer beam there is, there are the induced, the gravity shears that are coming from the hanging elements and there's also those induced seismic shears which is, I tried to show you in that previous slide with the diagonal strutting action that occurs between these two floors which induces additional vertical load in this, so there's quite a summation of loads that are going into this, into this wall at this point. Now on the left-hand side I've got a table here which shows, these are just the vertical loads. At 5D and 6D so these are the two columns that are sitting above the wall that runs between, on grid D between 5 and 6 so, at 5 this is the seismic corner column, 5D that's the right, not the corner column of the building but the corner column of the seismic frame on the southern façade and this is the one directly to the north of that, it's an internal column.

Q. You're referring to the row 6D now?

A. Yes. Yes, row 6D so column 6D. And so the first column shows the gravity loads that's dead and live load from the structure in a normal situation and the third – look I'll skip down to the fourth line which calls seismic over-strength beam shear so those are the shears that are introduced from the seismic action in the frames above, we just talked about before. The fifth line, displacement induced seismic from transfer beams. They are if you like the loads coming from this diagonal action within the transfer beam and in going back to the third line is the loads induced by vertical earthquake. So earthquake of course has horizontal actions and can have vertical actions. No doubt you've heard in some of the seismic sessions previously there was quite large vertical actions as well. It is quite difficult to determine exactly what the effect of that vertical action, vertical accelerations are on a structure like this. It is dependent on the response of the building to the ground motion. The fact that we have a cantilever a large cantilever section which can respond perhaps at a slightly longer period than an normal structure sitting on the ground certainly gives the suggestion that the, that it could have been quite a significant contributor to the axial load on those walls. However it is difficult to be precise about what the effect of that is. So we've put in a range from, of vertical acceleration actions ranging between .5 g and 1.5 g for, so we can consider the different sort of actions on that and that's, those are those two ranges of numbers in the third horizontal column headed "Range of Vertical Earthquake loads VE".

So, and then in the column that's headed D plus 1.3L plus E so that is dead loads plus 1.3 live loads plus E that was the combination that the code at the time required you to take when considering the axial loads on the wall so it's taken say the dead load and the live load and the earthquake induced loads. They could range by those numbers there for each end of the wall and that sums to a range between 34 and 45 meganewtons or 30–45,000 kilonewtons. The other combination that designers are required to look at is the factored gravity dead and live load so it's putting a, when design, when structural designers look at loads that are carried by an element they normally apply a factor of safety, effectively relating to the relative risk of the, of the loading and so dead and live loads have a factor, you can see there 1.4 and 1.7. When you are combining those with the dead and live with earthquake loads the factors on the dead and live are reduced so that's why the previous line is dead plus 1.3 live plus the E component, the earthquake component. So this range of between 33 and 45 meganewtons is what we assess as the sort of range of vertical axial loads that could have been experienced on this wall element. I would just point out one issue here and that is when we look at this line headed "Seismic Over-strength Beam Shears" you'll see that the number at one end of the wall is a lot bigger than the other end and that of course is because at Grid 5D that's the corner column so that is getting bi-axial actions, actions from both frames working at the same time adding up to a large number. Now that could equally be minus as well as plus so as the seismic loads, if you like, oscillate it goes from tension to compression. I've shown it as compression because that's the worst case on our wall. At Grid 6 those loads are much smaller because it's an internal seismic column. Now that in itself, because you're applying a much higher load at one end of the wall than the other, it does introduce what we call a "moment of flexure" in plane in the wall at the top of the wall at the podium because that's where those loads are being applied but because that wall is linked into the structure as a whole that moment would be dissipated down the depth and so we don't believe that there's much of that residual moment at the bottom of the wall.

COMMISSIONER CARTER:

Q. Given that the actual live load on the building at the time of the earthquake would be actually a proportion of the live load in the required calculation, what sort of sensitivity is there in the answer to varying the live load component?

A. Not great because the live load is in a structure like this particularly with some of the heavy elements. It's not a large portion. It is virtually dominated by the gravity load that's self weight. A hotel does have quite an extensive fit-out, as you know, so there's quite a lot of weight from the partitions and the baths and things like that. It typically doesn't have a lot of people load, people and luggage so it's probably slightly taking 1.3 live in this situation is probably maybe slightly higher, maybe double actually what was there but that wouldn't make a great difference to the end result.

Right so this is just a page which summarises those and if we look at the bottom row of numbers, F.1.4, if you like the possible maximum loads are for the axial load is 33-45 meganewtons. The seismic in-plane moment it could be 10-15 meganewton metres. There's actually a typo there in the report – the "M" is not shown – and the shear which is the horizontal shear in-plane of the wall can be in the order of 1.5–2 meganewtons.

Q. Sorry, where's the typo?
A. If you're reading off the spreadsheet that's correct and the original report there was a typo there which I've corrected. Now, just by comparison in the original calculations the design actions that were arrived by the designers are those numbers below and you'll see that the moment in shear is perhaps not that different relatively speaking but the axial loads could be a lot larger than what they had derived and the reason for that really is because they did not take the seismic induced axial actions into account. They didn't take any vertical earthquake actions into account and we'll discuss when we talk about what the code required them to do. I think it's probably fair to say that the code did not require them to account for those. The code did require them to account for the over-strength beam shears and that was not done and the induced loads from the transfer beams was also not included. The code is not specific about something about something like that but it should have been considered because of the actions from the compatibility. So the original designer did underestimate the axial loads.

COMMISSIONER FENWICK:
Q. Can you go back to that slide please. Where did your bending over-strength of approximately two come from. Can you explain that to me please?
A. Sure, that, well maybe when we look at the bending, the interaction that bending, axial bending, interaction diagrams might be appropriate.
Q. You'll handle it later okay. Thank you.
A. Right, so now we're just going to look at the wall itself so what we're looking now at is that elevation of the wall – D5-6. It is looking from the west towards the east so Cashel Street is to the right of that elevation and you can see a number of things on this elevation of wall D5-6. First of all that the high floor-to-floor span which gives it quite high slenderness between the ground floor and the first floor – level 2 as it's called here. You can also see at the top, highlighted in red, the transfer

beams on Grids 5 and 6 which sit on top of the wall at that point and below you can see the foundation structure. And this diagram also shows the vertical and horizontal reinforcing and that's also shown in this box down here. The upper drawing of that shows the reinforcing, both the vertical main reinforcing and the transverse reinforcing and you can see there –

JUSTICE COOPER:

Q. I am having trouble orientating myself here with these two blue shaded areas. I understood the wall is travelling between grids 5 and 6?

A. It does. Well let's talk about those blue zones there are zones of confinement as we've assessed them as to be required from the code, the contemporary code which was NZS310 1982 and you can see that in the bottom. If you look at the red box, the bottom of those two plans is suggesting quite a lot of what we call transverse reinforcement so the stirrups going through the concrete section there and the length of that blue box is about the length of that so that's what that's trying to show. The code did not require that transverse reinforcing over the full length of the wall but at each end, the end zone of the wall. In fact what was provided was what's shown in the upper direction which is quite a lot less, as you can see.

Q. So if the confinement requirement had occupied the whole of the areas shown in the box, the right-hand side, the left-hand side which has just got two lines of blue, would be completely shaded blue, right?

A. Yes it would, yes and these zones in the red box are vertical, if it's not clear, vertical sections down through the wall. It's only looking at about half the wall. It's only looking at the right-hand end of the wall.

1130

Q. Yes, well that helps, thank you.

A. And then in this picture in the top right-hand side, base of wall details, a couple of thing's significant. First of all we've shaded in purple there the zone of the brittle failure. That's where it actually failed in that zone there. Of interest there is that is the vertical bar which is shown as a vertical line, in the middle of that drawing it says 13 D16s at 300 each face, so that means it's 16 diameter bars, there were 13 of them, they were at 300 millimetre centres on each side of the wall. They are lapped at the bottom of the wall there and this is known to be a zone where the wall will yield, if any yielding is to occur and normally lapping is not encouraged, certainly for the main bars. The main bars you will see on the bigger elevation are lapped above the first floor, whereas the internal bars, the main bars being four D24 bars at each end of the wall. That's about an inch diameter. They are lapped above the level 2 and that's as required by the code. The secondary reinforcement is lapped within the, what we call the web of the wall, the middle part of the wall. In this case I think that's probably unwise because effectively it's all reinforcing, it's acting as main reinforcing under the actions that resulted on the wall, and we've attempted to sort of show the failure mode in those two drawings that we've done on the – in the middle there, cross-section showing failure mode. We think that the top of the laps, the lap bars of these middle bars probably had an influence of where crushing started with the very high axial loads and that developed into a transverse, so that's across the wall sort of diagonal failure and so the wall effectively slid off itself, sheared off and dropped and you can see that in the photos that follow. I'll show you those. Now here is – this is a picture immediately after and you can see it appears, what you're looking at there is the south end of the wall. At the top of the wall is a bit of a reaction like – for I think probably happened after this failure at the bottom because this kick towards the west because of a failure at the top of the wall, and here's a picture on the left-hand picture is showing the northern end of the wall, and here's a picture looking down on the middle section, and you can see there the ends of the lap reinforcing.

So this bar that is drooping down beside that end of the lap bar is the bar that lapped with that reinforcing on the – that's the eastern side of the wall so a portion of the wall is crushed and it slid down on that angle towards the west as we attempted to show on this diagram here, so the picture that you are – showing you is this photo 10, is looking down on there.

Q. We are going to have an adjournment Mr Thornton at a logical time.

A. Yes, well I think I'll just talk, I've got more to talk about this wall – but it'll sort of take quite a lot of time I think, so I'll just finish talking about the detailing if I may and the confinement, and then maybe if that suits you it might be a time to stop. I'm just bringing you back to this, because one of the questions I guess we ask ourselves is if this reinforcing, confinement reinforcing had have been in, would that have stopped the wall from failing. We've come to the conclusion that on its own it probably would not have and I'll elaborate a bit more after the recess but it's to do with the slenderness and the available ductility within the wall even, really the very extremely high axial loads that resulted on this wall.

COMMISSION ADJOURNS: 11.36 AM

COMMISSION RESUMES: 11.53 AM

JUSTICE COOPER:

Q. Continue.

A. So that is where we finished seeing the shape of the failure at the bottom of the wall and you can just see here there is very little in the way of transverse reinforcing going through that wall. One of the ways that you assess how much or the requirement for transverse reinforcing does depend on assessing the in-plane moment in the wall and the demand for main reinforcing so you recall that at the end of these walls is like four one-inch diameter rods, D24 bars, which is quite a small number of rods, a small area of reinforcing and looking at it in comparison to some of the other walls it's a bit, looks a bit counter-intuitive because some of those other walls that we looked at have much larger amounts of main reinforcing and consequently more transverse reinforcing. The reason why I think the designer ended up in that position was that he assessed, if you like, with the axial load that there was only a small demand for flexural reinforcing so that small demand for main reinforcing therefore he did not need to put much confinement on it. He missed the point that the axial loads were very high and he should have been thinking of it as a column rather than as a wall and I will explain a bit more about that as we go through but I think just worth making that point because when if I had showed you the reinforcing drawing for one of those other walls, for example the one down at grid 11 it had a lot more confining reinforcing in it even though the loads on it were a lot less. I, just out of interest I included a couple of photos there from other buildings in Christchurch that had like similar wall failures, so and I think when we look at the recommendations and dealing with shear walls it is so that the, these ones undoubtedly a bit different to what happened in some respects but it does have some similarities which perhaps suggests that there are the way that we designers have been designing our shear walls needs to be improved.

Now I am going to come on to a little bit now about the actions within the wall and diagrams on this page are what we call interaction diagrams. It shows the capacity of a column both in terms of axial load, that is the vertical compression or tension load on it, and the moment, that is the flexural actions on the wall. Now on the left-hand side there are two diagrams. If you look at the bottom one first it is the major axis so this is the in-plane actions on the wall so that is when the, if you like, in a north-south direction loading that will induce in-plane loadings on this wall and the, you can see that with zero moments the wall can take an axial load of about 60 meganewtons. Now remember that the axial load we were talking about was about 45 meganewtons as the maximum that we assessed might be taken. The original designers had assessed a vertical load of around the 15-20 meganewtons which put them in this bottom part of the graph there. There is something reasonably significant about that and that is that because in that scenario there as you increase your axial load then your moment capacity is increasing. So at 10,000 kilonewtons, 10 meganewtons, you have got a moment capacity of sort of 25 kilonewton metres whereas if you double the axial load the flexural capacity has increased, and normally for engineers, designers, you know, that is the sort of safe area to operate in because then if we get increased axial load then actually improves the flexural strength of the wall whereas under the high earthquake-induced axial actions we were up here, the upper part of the graph and when you get to there of course it is generally considered sort of less safe area to work in because as the axial load is increased then the moment capacity is decreased. So that is the major axis.

In the minor axis it is, the moment capacity is much less and is not particularly important and in fact my own view is that in this particular considering this wall the moment the flexural actions are not so important. It is really about thinking about the axial load that is on the wall and the displacements that were imposed upon it. if we look at the picture on the right-hand side that shows, it is an interpretation of a combination of a bi-axial interaction diagram for that wall and it is expressed in terms of eccentricity in each direction so the moment is deduced by the, the moment capacity is deduced from the eccentricity and we have got three numbers there for different axial loads so if we look, it is worth looking at the horizontal axis which is giving the eccentricity or in-plane loading. Now when we get up to say the higher load that we have given there which is 28 meganewtons so that is not the highest load that we assess but it is the higher end, that is assuming an eccentricity of getting up to one and a half metres. Now our wall was only five metres long, so what that means is that a good half of the wall, more than half of the wall is in really high axial loads so that the axial load be it 28, 45 whatever is not sitting squarely on the wall, it is effectively displaced towards one end of the wall so it is getting very high compressive strains towards one end of the wall. I just come back if I can to Professor Fenwick's question about over-strength and I – there are a lot of factors in this, obviously in this wall and determining in a true sense to work out what an over-strength capacity is we work out what the, what our computer analysis tells us what the in-plane feature is and then we, for the actual axial load we apply we see what the moment capacity is at that point and that gives us our relative over-strength. Now in this case it would be a very large number because the computer analysis was not telling us that the axial loads was all that high so I mean I, to be honest, we have taken a number of about 2 as an amalgam of possibilities but as I said I do not think it is desperately relevant because it is really more about the extent of the axial load and the displacements that were imposed on the wall.

COMMISSIONER FENWICK:

Q. I take it then your factor of 2 does not apply to the axial loads induced by the moment resisting frame above the floors?

A. No, not at all, no for those we have taken the normal to work out our VOE's, our over-strength shears, we have taken normal over-strength factors as applied to being flexure and we have reduced them cumulative as we come down the building as we are directed to by the –

Q. Based on probable strengths?

A. Yes.

Q. And probable strain distributions?

A. Yes, the actual detail of that I am not quite sure to be honest at this point but it is done in accordance with the code so dividing by 5 so that takes you to your probable level and with a straight hardening effect on top of that, effectively 1.4 if you like.

1203

COMMISSIONER FENWICK:

Q. The code would have been based on upper characteristic strengths. You're unlikely to have upper characteristic strengths. I mean that's what we're doing, design upper characteristic strengths, but the building itself would not, you would not expect wall reinforcement to have an upper characteristic strength would you, you would expect it to have an average strength?
A. I guess that's fair. I mean there was a reduction of factor applied, but that's more a probability of the number of beams that are likely to be yielding at any one time.
Q. You've selected the number of beams yielding, again from the code approach for designer columns I assume?
A. Yes. Yes, so I mean in the over-strength component, the flexural over-strength component on the wall below is a reasonable portion but it's, again it's one of the components along with the gravity loads, the shears induced by the transverse beams and the – in my view the probably seismic vertical earthquake actions as well.
Q. Sorry, I thought it was a high component.
A. It's up to –
Q. Ten thousand, about, well 10 meganewtons.
A. Subtract a bit at the other end so it's maybe eight meganewtons, is that fair.
Q. Thank you.
A. Now we're looking at a method of looking at the, if you like, the brittleness or the ductility or the robustness of the wall and this is termed a moment curvature graph and it's – on the vertical axis it has the moment capacity, and on the horizontal axis it has the curvature. In simple terms that's the sort of the rate as of a – that a member bends, well literally the curvature of a hinge, which in this case would be at the bottom of the wall but in a frame it's typically at the curvature that you get in the end of a beam, or in the bottom of a shear wall. Now ductile members are not brittle elements, would have characteristics a bit like steel so you have a yield portion and then a post yield, or plastic section

where the member will keep on deflecting without gaining or losing load capacity, and if you look at the bottom two lines on that graph, so the – what's that, a sort of a teal blue and a light brown, those are showing if you like desirable characteristics, desirable elastic and then plastic performance and as you get up to the next line too the light blue line at – starting at about a moment of 20,000 Newton metres. That too is a desirable. As you – now these numbers as we go up, sorry these lines, graph lines as we go up the page are increasing axial load applied to the wall. Now as you get up to the top ones and it gets up over the – towards 40,000 there, they are showing very little of that desirable plateau if you like. That's indicating a very brittle type member because once you get past its peak strength if you keep pushing it, like pushing extra deflection which induces more curvature then its load capacity drops off and you have a brittle failure, so you know a brick wall for example will have this sort of characteristic. It can take its strength up to a point and then it fails abruptly and loses its capacity. Now a modern structure shouldn't. Steel structures we expect to follow a curve like showing at the bottom where it takes a load up to what it yields and then it keeps on extending without breaking, without ultimate rupture and we try to achieve the same thing with our reinforced concrete design and that's part of the reason why we confine the reinforcing, confine the concrete and the main reinforcing with the transverse reinforcing, which effectively makes the concrete act in a ductile manner. Now –

JUSTICE COOPER:

Q. The second to bottom of these lines, I just can't quite read the notation. Is it capital EC equals .002 –

A. Yeah.

Q. And at the other end -

A. Yeah. That's a measurement of strain.

Q. Right, and is there one at the far end, is that – is it the same (overtalking 12:08:49)?

A. 004, yeah.

Q. Is that EC as well?

A. Yes it is, yes.

Q. Okay, thank you.

A. I mean strain is a little bit different, that's right, that's a unitless thing which is a measure of the stress over the elasticity of the structure. Right, so this is again shown for the major axis and that even at the higher loads, even in the in-plane is shown to be quite a – potentially a brittle element. Remembering of course that the – where the design, original designer thought he was, was down sort of half way up the graph where he would, at those axial loads you would get a much more ductile performance from the wall.

COMMISSIONER FENWICK:

Q. Mr Thornton, can you perhaps enlighten me, how do you – to calculate the deformation or the deflection the wall can stand, you would need a effective plastic hinge length which would be very variable in the two directions. In-plane it would be very different from out of plane, can you give me an idea of what you believe the plastic hinge length is so I can assess what the physical displacement capability would be?

A. We looked at it in – again the available information available to practitioners is a bit – could be improved in that area, particularly something like a wall where, a shear wall where it's not forming the same deflected shape as a column where we have guidance on the hinge length, in fact you can derive from for the various documents and standards and guidances as to the length of the hinge, and then you can take the elastic portion between the hinges. Here we – it's not clear that quite what happens at the first level because in a shear wall all the yielding, the rotation is occurring at the bottom of the wall whereas it was straining it at the first wall, so – I'm not sure (overtalking 12:11;22).

Q. Can I summarise that perhaps, you don't know what the effective plastic hinge length would be?

A. Not clearly no.

Q. Okay, it's an area really which we need a bit more research isn't it?

A. I believe we do, yes, and certainly I think we practitioners need more guidance in that area.

The second, the next one shows the – that the same type of graph but for the out of plane. Now as saw from those photos I showed you just before, the actual mode of failure. Our interpretation is that it's a transverse failure. Now the wall failed for a number of reasons as was described earlier. It had high axial load, it had a very high floor to floor, so its slenderness as required by the code was exceeded, so that means its height to its transverse width was exceeded. So that means it had a propensity to buckle out of plain. Perhaps the fact that the lap was there concentrating the stresses, a whole lot of things, but it looked to us that it was on the point of wanting to buckle. It had very high compressive stresses and from the axial load, and then we've got a horizontal displacement imposed on it by the structure. When you analysing a building like this you assume that the walls on the outer plane are not, are not really contributing the seismic resistance. Their, their stiffness is so small compared to the other walls that are running perpendicular to them so they if you like go along for the ride much as the seismic frames do but they have to withstand their, the displacements that are imposed upon it by the structure and so that's relative when we look at this, this graph here because it's showing that really once we get over 20,000 axial load which at the sort of low level of axial load assessment then it develops a propensity for brittleness ie non-ductile behaviour. And you can see that by the shapes of those graphs that are tending to taper off. There's really no, no yield. So what, what we –

JUSTICE COOPER:

Q. Did you say 20,000?

A. Yes which is the, which is that red, 22.

Q. 22,000.

A. Yep. What we did do and this is partly answer that question you asked Mr Fenwick in terms of the, the, what we looked at is what was the expected displacement at the, at the level 2 and it's a very small number and we did look at what the, what we expected the, the elastic and the deflection to be, the capacity of the hinge at the bottom and it's really almost, we're talking millimetres before you get to a point where you're pushing it beyond, effectively beyond the crest of these so that it is and that's where we came to the conclusion that a brittle failure was relatively inevitable once we got the very high loads and you impose even quite small displacements on the wall on a traverse direction. We did, just to slow it down (inaudible 12:15:35) but we did look at, we tried to look at what is the, what is the effect of adding more confinement reinforcing to see whether, whether that would actually would have prevented it, the failure from occurring, ie so if the designer had put in the amount of confinement reinforcement which we, we believe the code directed them to do would it still have failed. This I think the software that we're using to do this I think is giving slightly misleading argument, results here because you can certainly argue that the first portion of those lines should all be concurrent. The different lines represent higher, we've tried to model the effect of the confinement by allowing greater strains within the concrete, whereas in reality I think the, certainly in the elastic portion the line should be concurrent. Anyway I think why I have put it up is because I believe it shows that even with if you had added more confinement that it wouldn't necessarily have stopped the failure. At the end of the day the wall was just too skinny and its short direction so that, particularly when you start, when you've lost your cover concrete the amount of core concrete that's left is not sufficient to act in a ductile manner. I think this is something that Professor Pampanin will talk a bit more about and, and because I think in academic circles the thought is that we need areas, we need to limit the amount of axial load on columns and walls and so perhaps that's some area for direction to the code writers in terms of where we're, where we're heading.

I am going to talk a little bit about seismicity now and I'm sure you've probably seen graphs like this before and I think Professor Pampanin will probably talk more at length but we need to sort of look at where our building fits in this. I've got here on one page four graphs, they are showing on the left-hand side in September both the acceleration spectra and the displacements spectra and on the right-hand side the displacement spectra again for, sorry across the top we've got, sorry for February we've got the acceleration and displacement spectra on the right-hand side. So acceleration spectra gives you a graph showing the horizontal acceleration in this case in relation to the period or the natural frequency of, of the building and so if we look at this, that the red line which is the current code line if you like 11-70 what that shows is that as a building gets more flexible the load it experiences due to the response of it, of the building reduces as the period increases. The two dotted red lines represent slightly older versions of code and the bottom one is the code that was in place at the time of, sorry I beg your pardon the upper one is the, is the code which was in place when this building was designed. So you can actually see that in fact for the, the –

JUSTICE COOPER:

Q. Just explain that to me again, this building was –

A. Designed in '85, '86.

Q. So I'm just a bit confused by the, by the arrows. The NZS11-70.5 2004 is that the dotted –

A. No that –

Q. – black line is it?

A. No if you're looking at a top, I'm looking at the top left graph it's the –

Q. Yes.

A. – it's the red line just, you can see that that's the, it's pointing to this one. It's the -

Q. Oh it's -

A. (inaudible 12:20:53)

Q. It's the continuous line?

A. Yes but it's, yes continuous red line.

Q. Right okay, right now I understand thanks.

A. What that shows is that the buildings with a period of less than 1.5 seconds the current code, this is for Christchurch, is an increase on the older code levels but once you get beyond 1.5 in fact the older codes were at a higher requirement.

Q. Mhm.

A. Our building is you can see on the graph below that where the period is sort of, a range of period is shown sort of somewhere between round about a bit over two and a half seconds is the initial period. As the building softens and various effects in the foundation actions come into account then the building softens and has a, can have a longer period so we're interested in the range of sort of between two and a half to maybe four seconds, that's certainly in terms of code requirement. The current code requirement is less than what it was when the building was designed. Again still looking at the, at that top left, the acceleration spectra which is roughly sort of giving you a measure of the force that's, the horizontal force that's applied to the building compared to its period the, the erratic lines if you like are the records from four sites around Christchurch CBD and in the period we're interested in it shows quite a peak but also the peak is quite variable so on a couple of them the, there's a black dotted line and a brick red dash dot line which are quite low at two to three seconds whereas the, the green line and the blue line which are at two of the other sites are a lot higher. That does give perhaps – suggests some uncertainty and again Professor Pampanin I think will talk to that, but if you – there's a grey line in the middle which I'm pointing at down there which says the mean of those four records, that's suggesting that there is a peak which is quite a bit higher than what the code required in September. Now if we look at that at the 4203 line, the contemporary code with a broken red line with the longer dashes, that's what our building was designed for but that's with a ductility factor applied, so yielding could have been assumed to have occurred way down here. That's because that red line has an implied ductility and it's somewhere between three and four. By ductility I mean that it's – it should be able to take displacements, maybe three to four times the displacement that occurred when the building's first started to yield. So if you take a point say one-third of the current code line and then look at the peaks, even up to that grey line, the mean line, it means that there would have been a severe ductility demand on the wall in September. Now observations from the engineers, who you will hear tomorrow, quite clearly – are quite clear that there was no such demand made on the building. It did not experience that bigger earthquake in September. I think that this something that has been discussed, another four, and again professor Pampanin will perhaps talk to that, but I say I think the one view is that certainly the records are perhaps not reliable in this – that September earthquake in this area, and partly – the range that you get for the different recording stations is, maybe –

JUSTICE COOPER:

Q. In the inset on that first diagram, the upper left-hand quarter of the page, EQ7WTB, is that a measuring station that ceased to exist, because it doesn't – that's its one and only appearance on this page.

A. Yes, you're right.

Q. And that seems to me probably the closest to the -

A. Yes, I don't think that's

Q. - subject site.

A. We might defer that question to Professor Pampanin because he prepared this.

Q. He can explain that for us, thank you.

A. All right, so again if we look at the displacement spectra for September, the – well this is – if you like the same information but presented in a different way, and it's showing not the acceleration but the expected displacement of the building at its effective height which is not the top of the building but a point which sometimes is referred to as effective centre of mass. It's – in simple terms its typically two-thirds of the way up the building, but it can vary and in this building it would have varied because of the vertical irregularity.

COMMISSIONER FENWICK:

Q. You're referring to the centre of mass in the first mode?

A. Effectively yes. And a point of interest which I know is of interest to Professor Fenwick, is that these graphs are typically shown as coming up to a point at around three seconds and then showing horizontal, as a horizontal extension, so that what's that – so effectively that's suggesting that once you get over three seconds the displacement is the same. Now if we look at the actual records on this case as it's clear, that once we got beyond about three seconds they – the displacements appeared to reduce and that's –

Q. Mr Thornton, I think you're referring to a question I put to you and the question was, 'Does that comply with the code of the time?' I know the general perception given the many other codes in the world, is that the equal displacement concept applies from three seconds on, and that's the concept in 1170.5, our Earthquake Action Standard, but I was unaware of any condition in the current, in the then current code, the 1976 or 84 code that said the displacement stopped at three seconds. In fact, you know assuming the displacement is a function of the period, there's no limit given on it and it climbs on up, so I'm just wondering can you advise me in that code does your diagram comply with that code or is it someone else's interpretation of what they think it should be?

A. I think the latter is the case and again that's something which Professor Pampanin's probably better placed to answer than I can, but I suspect that it's really applying retrospectively the sort of what's done now, to what perhaps could have been done, because the displacement spectra were far less (overtalking 12:29:24).

Q. Yes, how do you feel about matching bits from one code with bits from another code?

A. Well in terms of the design sense that – it can be flawed, of course here we're just trying to illustrate an issue rather than – so perhaps nothing too much is depending on it I think. I mean whether that's – I say the actual record suggests that in this case that in fact displacements tend to drop off but I understand that there are cases where it has been shown to increase, so, and I'm – but again I will leave that to Professor Pampanin to elaborate on.

Q. I suppose it might be significant if you were trying to work out whether a building as designed, like at the code at the time, you'd want to represent what the code required accurately, for that purpose wouldn't you?

A. Yes. And I think, but if we look if you like by happy chance in the area that of period that we're interested in, simply the mean of the actual is something similar to the area, and is assuming a displacement of around 500 to 600 millimetres. Now again that would have imposed a significant ductility demand on our structure that was not evident, so I mean it is quite apparent that the September earthquake did not exert – demand a response or involve a response from the building that matched these spectra.

Q. I think if we may perhaps I can question you about that a bit later on.

A. Sure, now if we look at the February ones, the notation is similar. We can see there a couple of interesting things. One is that the records from the different recording stations around the city are a lot more if you like concurrent, they tend to be more consistent, particularly when you look at the displacement spectra that they – and so that certainly one reference of mine it's suggested that, suggested that the records there's a reason for thinking that they are more reliable and what is also of interest there is that as the building, as our building softens, then the –

I'm looking at the displacement spectra then a higher, both a higher acceleration demand and a higher displacement demand is placed upon it, so as the building softens the loads get larger and the displacements get larger. I mean this, and here is a case again referring to Professor Fenwick's question, where if we take those as being true then the determination of having a horizontal displacement after three seconds is – an equal displacement after three seconds is patently not correct, and I think certainly from practitioners, certainly from my point of view it's – it was a surprise to see these blimps down around three seconds, both in September and in February, but when we look at that again at displacement spectra for February it is clear to see that there was a quite a displacement demand placed on the building. This is in the principle direction which was primarily east-west in the February event and that, so that does put induced transverse displacements on our little wall which we know from the curvature diagrams it was not able to withstand. All right, I just have some line conversions of those but I will move through those. (inaudible 12:34:20) some information we had some –

JUSTICE COOPER:

Q. Can I just note we are going on to the document number 46.46, thank you.

A. These are if you like some computer model generated pictures of the structure, both effectively as we modelled it and as it and its affected shape. You will see there in fact we, in the upper structure the cantilever bay is not modelled and I think that is quite typical, certainly the original designers did not model that or, and the gravity structure unless it is carrying vertical seismic actions from above is not modelled, and you can see there a front view showing the walls, the frames above. Here is after the failure of the wall and the vertical displacement of those frames so this is allowing the frame members to take the loads once the wall is removed and it assumed the shape reasonably like what we see on site and here we can see the, this from a different angle.

COMMISSIONER FENWICK:

Q. Mr Thornton, you are saying that this shows deformed shape?
A. In a very –
Q. That was a result of an elastic analysis –
A. Elastic analysis only.
Q. – without the cantilever portions of the structure in place?
A. Correct, yes.
Q. So how does it, that surely missing out those cantilevered proportions which is giving you quite an effect, if you have got a cantilevered load on one side of the building that is going to pull that down that must have surely a big effect on the shape that it is going to deform into, even elastically let alone elastically when you get into elastic deformation?
A. It has some effect. The actual amount of it is I guess debatable. It could have no effect if the gravity and the seismic, if the cantilever system and the seismic system are not mixed so that if the gravity system effectively is resisted without inducing column shears but only as a see-saw action, up and down motion, then I do not agree. I think it is should, unless you take column shortening into effect to increasing lateral displacement. But you are correct in that in this case there are certainly the in-frames do have an effect on the seismic frames and so resultant out of balance moment at those columns on grid D will introduce some shear through the tower section which will add to displacement in one direction.
Q. Thank you, so this then is just ignoring those actions entirely?
A. It is. This was done to effectively to look at displacements in a relatively crude form I grant you but also to look at the strengths of the, from the strength demands, member demands in that sort of deformed shape which is the shape that it has ended up, that it ended up in.
Q. Surely eccentric loads would induce moments and demands and strength on their own, so how can excluding them, can you explain to

me how excluding those actions can give you the strengths and demands of the structure?

A. Well in the east, we were primarily looking in the north-south direction which are not affected by those –

Q. I think even in the north-south direction the cantilever actions on one side would induce moments in the columns and when you apply the action the other way then you get a bi-axial moment in the columns and actions wouldn't you? I cannot quite see how you can separate those two actions out?

A. In terms of the axial loads in the columns I think that is, this was of almost through statics by the effectively from the member capacity or the reinforcing in the beams.

Q. Thank you for your explanation.

A. And there is a little diagram here which is in the report which does attempt to, if you look at, to look at the displacements of the building both of the upper frame section and the wall, the lower podium walls making some allowance for obviously the relative stiffnesses but also the damping that you might achieve from the upper structure and in the upper structure we did have potentially a lot of damping there from both a lot of partition walls but also the perimeter pre-cast panels which were, there was a lot of them and they were all quite tightly fitted but well designed so they did not, they were able to move. They had quite a lot of sealant before them and there are some reports of broken sealant following the September event so it is possible that some, it is possible there was some softening occurred in the September event, not structural damage but softening of the, if you like, reduction of damping that occurred in the first couple of cycles. To get to this, these deflected shapes they are, they do start from elastic base, elastic deflection base but then they, there are assumptions of different ductilities as applied to the tower section and to the podium section. The podium section and the frame section are designed for actually slightly for different yield points so effectively there was an over-strength capacity applied, not a full over strength but a design margin applied to the shear walls so that yielding would clearly occur in the frames well before any yielding in the walls occurred. This is flexural yielding we are talking about. So you can see there that the, you have got elastic deflection by blue line and then different, a higher ductility applied to the displacements of the upper tower frames. There is some increase in the, in the podium resulting from two things of the deflections. One is the effectively the over-strength of the frames but also if you like the increased elastic actions from the components of story shear that are applied to the bottom stories only.

So I'm going to move away from the, that a bit on to the stairs but I guess the stairs now are, it is if you like, it is relevant to that, the deformation story, displacement story. The – so this section, this picture titled "Stair Sections" is an elevation in two parts so the bit on the right effectively sits on top of the bit on the left so you've got on the left goes from ground level up to level 14 and then the bit on the right goes from 14 up to the top of the building and it shows the stairs in the, in both directions but I've only coloured the ones in one direction and that's the direction if you like the diagonal through the stair that, that lengthened due to the displacement, the resulting displacement once the wall failed. As that, as the wall failed the building lurched towards, particularly towards the east and that imposed on the tower structure a permanent, a quite major displacement towards the east. It has the effect of if you like lengthening the diagonal between, across the frames in the lower, the lower point of any floor at the western side to a higher point on the eastern side. And it is these stairs all the ones that are coloured red which collapsed and fell down through the stairwell. They didn't get all the way down to the ground level, they, as you can see there is about, it took about three levels of the, of the podium level stairs to collect and withstand the bombardment from the stairs above if you like. And initially there was at least one stair I think left at the top of the building, although I understand that may have fallen out in a later, later aftershock.

JUSTICE COOPER:

Q. How was the highest of the intact stair flights able to resist the weight of the looks like four flights above?
A. Sorry, I don't understand?
Q. If you look at the left-hand side?
A. Yep.
Q. You've got stair flights debris and then the green shading indicates intact stair flights doesn't it?
A. It does. Yeah.
Q. So I'm just wondering how was that upper most of the intact stair flights able to withstand the weight of the collapsed stairs above?
A. Well it, by working very hard, I'll show you a picture that shows –
Q. That shows (inaudible 12:46:50)
A. (inaudible 12:56:51) side of that and it is, but I, I imagine that what happened that there was if you like a progressive collapse.
Q. Yes.
A. And the ones above it if you like slowed the, absorbed some of the energy from the falling stairs. One of the other factors is, that is perhaps of interest is that if you look at the lower stairs up to level 11 that they are a single flight of steps going up from each level so there's no mid-height landing. Once you get into the upper structure there's a mid-height landing. Now that was a, if you like, an access and egress requirement that was introduced to the codes at about the time that this building was being designed and built so I'm assuming that's a fact you know the lower stuff was consented and was half built and then the codes changed and they were required to put landings in, in the, in the upper level. So that had the effect of lengthening those stairs a bit but also if you look at the lower stairs you will see that there's a shear wall which is one end of the I wall, the central I wall is beside grid D there

and I suspect that that wall helped to, because if you think about that stair that's going from 11 to 14 it spans, it would fall down and hit that wall, one end of it would hit that wall adjacent to grid D so I suspect that that wall and the longer stairs mean to a certain extent that sort of that maybe their descent was slowed as they sort of got wedged up against that wall.

COMMISSIONER CARTER:

Q. The diagram shows only all those red walls as collapsed, what about the walls that are not coloured running in the other direction, did they, did they not suffer?

A. No they didn't because, well they, they are the short form, short line of the diagonal, if, very crudely if you look at my fingers the, as the earthquake was moving it does this. Now it ended up doing that so the ones that are going across there the ones that fell down –

Q. Yes.

A. – and the shorter ones didn't lose their seating.

Q. So that applies to one oscillation of the earthquake, the reverse oscillation wasn't sufficient to -

A. (inaudible 12:49:21)

Q. bring those other stairs down?

A. I'll, I'll – the work that we've done on this suggests to us that certainly with the number of oscillations that we had that the stair, none of the stairs would have collapsed if it wasn't for the wall collapse.

Q. Yes.

A. So it was the wall collapse that induced the additional displacement towards the east.

Q. Thank you.

A. And the others didn't. Now if we move on to the next slide you'll see some, some of the detailing of the stair and this is of the, in fact one of the lower flights but it is similar, the connection details are the same. Now you'll see that they, it's a pre-cast element of concrete which means it's made off site, it's craned into position and each flight of stairs has sticking out at the top and the bottom a steel, it's a little steel beam that sits in a pocket and you can see the details of the pockets there on the right-hand side. Now the interest thing about those is that in all these cases they have some ability to get longer, to slide out and that is probably the order of 70 to 80 millimetres and any one end the stair can lengthen but it has no built in capacity to shorten because, well maybe 20 millimetres in one case if you look at the top right fixing you'll see there's some polystyrene there at the end of the steel and there's a 20 ml gap in the stair so that's allowing for a 20 millimetre shortening. Now what actually happened is that the ones that shortened it blew away, these steel beams pushed out some of the concrete at the seating point and I'll show you a picture of that. This I guess I assume you're going to talk quite a lot more about stairs when you come to the Forsyth Barr discussions but, so this is a detail that's used quite a lot by designers who designed this building in a number of buildings and certainly nowadays if you look at detail down the bottom of this drawing where I'm pointing beside detail 2 normally the, the gap between the stair and the floor below is taking it at the bottom of the first riser so a horizontal line is drawn through there and the bottom stair just simply sits on top of the landing. Now that gives plenty of capacity for it to slide and probably up to about 300 millimetres movement or so and many designers do it that way and have done it that way for quite a long time. I think I'm right in saying this detail is driven by the desire to have a clean line on the underside of the stair because if, if it is supported on a landing at this point here the landing steps down. I can draw you a picture if you like but the stair landing comes and sits, the bottom of the stair flight sits on the stair landing so the underside of the, of the stair landing has a step on it which maybe to an architect is seen as not desirable. Certainly I don't think anybody's doing stairs like this anymore, but, so what happened is that, I say it had capacity to move outwards and these stairs, they were strong enough to take a shortening ie the stairs could be compressed without failing. The method of failure was for the landing to be damaged, and you will see that in some photos. You will hear, you would have seen on the Forsyth Barr report that failure also in there occurred at the mid-height landing. There was a much longer and heavier stair than these ones. I will show you those photos here. So here's a photo, the top right photo, photo 23 in fact shows damage that has occurred to a landing as the stair shortens. So this is obviously a stair that survived, it was on the shorter – on the diagonal that got shorter and it did some damage. Now it's a fair argument that if, even if the shear wall had not failed and the movements had carried on for a longer duration, then that repeated action may well have caused damage such as the stair would have collapsed. The photo below that, 24, does show that bottom surviving stair, well higher surviving stair I should say on the side that collapsed and you can see there's a lot of load on top of it, and the side, you can see the debris at the bottom. You can see the debris in the bottom of the stairwell above that point where the stairs ended up, those that fell down. And there's a picture of which are not annotated, on the left shows a view of the shaft with the stairs that have fallen down to it, quite remarkably there's very little damage to the adjoining walls and you can see there what appears to have been where the point of the stair connection has ripped down through the landing, so there's a number of mechanisms that went on there.

JUSTICE COOPER:

Q. What's the red tape?

A. That's a fire hose that I think someone may have used to exit the building immediately afterwards. When we looked at the displacements without accounting for the failure of the wall and I would have to say without accounting for any additional displacement from the eccentricity issue that Mr Fenwick has mentioned, then we don't believe that the stairs would have failed just from running out of room, but as I say, as it had gone on for a longer duration then the repeated movements may well have worked their way through the landings and here’s a photo which shows this mechanism happening. I didn't take this photo but I understand it is one of the top surviving one and which may have fallen down in the interim. You can see there that the stair has dropped if you like vertically, almost a tread, and is the steel just about to fall out the bottom of the landing.

Q. Thank you.

A. And here’s one in progress which is partly pulled out and some more damage there.

So I've got a few sort of text slides now, really as prompters. So the first were of questions that if you like that were asked through the expert panel and were responded to in the report. Some of those I think I've addressed these already, so the response to the September event was quite low and I think perhaps Professor Pampanin can address that as well but there's general acceptance that the spectra that were developed for the September event with any response of the building did not match what the spectra has shown in the September event. Did the building comply with contemporary codes? I made the comment in the report that generally the building appeared to have been well designed and well detailed. There was a critical vulnerability in this wall. It was too slender, it didn't have adequate ductility and the vertical, potential vertical loads on it were underestimated quite severely. Would it have failed in a code event? So that question really is did it fail just because we had a particularly big earthquake in February and I think the answer is that it could have failed. It's by no means certain but in any particular event it will, the shaking may, the predominant shaking may be in one principal direction. We needed for it to fail it did need concurrent actions resulting in very high axial impression loads in the south east corner. Obviously that is what occurred. You could have code level earthquakes where that didn't occur but I think the answer is yes it could have failed in a code event. Could have failed in the 1170 event? Well I think the answer is really the same because in fact 1170 in particular prescribed a lower event because of the long period of this building. What was its percentage NBS, new building standard? It's a difficult one to assess actually because it is quite hard to determine exactly at what percentage of code loading would induce that wall to fail and again it is dependent on the direction so we took a bit of a stab and it's not much more than that because it withstood the September earthquake, you know, and when you look at those actual loads on it, then maybe it would have survived a 70 percent level, code level, but there's not much science in that I have to say, so it is difficult in the circumstances to determine that. And was the stair collapse dependent on wall failure? Well I think in the event that we had, yes it was, but a code level event could have resulted in a stair collapse because of the detailing in particular, the inability to take a shortening within the stairs at the landing connections.

JUSTICE COOPER ADDRESSES MR ZARIFEH

COMMISSION ADJOURNS: 1.03 PM

COMMISSION RESUMES: 1.53 PM

JUSTICE COOPER:

Q. Yes Mr Thornton? I think you were on the sheet headed 'Recommendations'.

A. Yes, so these were some of the recommendations that were included in the report and I think relatively straightforward. Certainly design rigour for irregularity we think not that needs some attention. The current code does place some limitation, does draw attention to irregularity but I think from a practitioner point of view having more guidelines and perhaps prescription about methods of analysis and dealing with different forms of irregularity is something that could require, could do with some clarification.

COMMISSIONER FENWICK:

Q. So if I understand you rightly, it is just purely on changing the, when you are referring to that you are referring to changes in the standard design approaches more detailed in the codes in fact?

A. Yes, yes, it may require code change but also if you like practice change and in education perhaps, and I think also –

Q. Sorry, is that, will that also perhaps include a more rigorous checking process for certain structures?

A. Well it is a much wider question and of course there are consent, the government has signalled consenting changes and amendments to the Building Act which are going to affect those practices anyway. My firm belief is that any reasonably complex structure should have a high level of peer review and I think that is, I mean we have moved, the country has moved away from having expertise, (inaudible 13:55:57) speaking in territorial authorities and perhaps some engineers have believed our own expertise a bit too much. I think it is always a good to have rigorous peer review and so –

Q. If I was to summarise then, it would be a change to the building standards, increased education of these problems, and would it also include additional requirements for peer review? More rigorous peer review?

A. That is a very good summary.

JUSTICE COOPER:

Q. Mr Thornton, the signal from the government you are referring to is what?

A. Well I think the term is risk-base consenting so that for very simple, if I could summarise, I think a fairly simple residential dwellings where issues of course have arisen from weathertightness they are seeking to remove, I guess, liability from TA's and you are going to do by requiring designers and builders to certify their own work to a greater degree.

Then there is a middle range of complexity where the, which will still have more a conventional consenting process and then in the more, let's say in the commercial area which would apply certainly to the buildings like Hotel Grand Chancellor that TA's should be able to rely on the experts so the designers, architects, and engineers who are designing these because they are the experts in that field. Now I guess in one sense professionals would agree with that but I think if you do that and remove the requirements for peer review then you have a, you end up with a problem so it is –

Q. These –

A. – the process needs to become much more robust, excuse me, in terms of both the industry and perhaps requirement for peer review.

Q. Is this, are you referring to a reform that is underway or something that is merely being spoken about?

A. It is underway. Part of it has been signalled in fact came into play this month which is the first stage about residential risk-base consenting and then there are, I think, amendments 3 and 4 which have been tabled but –

Q. To the Building Act?

A. Correct.

Q. All right. Thank you.

A. Second item we had is design rigour for shear walls, particularly flexural shear walls. Shear walls have, the term shear wall can cover all forms of, well quite a range of walls. There are some walls which are what we would call squat which is quite low and very long. They tend to fail in shear, sliding if you like. There are walls that we have here in the Grand Chancellor which attract large flexural actions so they tend to yield in flexural bending before they yield in shear. And then we have this, a variety of that is if you like very highly loaded shear walls, in fact this wall really acted more like a column than it did a shear wall. Certainly its function was because of the very high axial load so that is something that I think again there needs to be signals to the design industry, possibly some changes to the codes. I think Professor Pampanin will talk about the suggestion of limiting the maximum axial stresses on both short walls and columns. Those types of things, but also I think I showed you those slides of other failures and these types of failures have been evident I think in Chile and other recent earthquakes so the design industry and building industry needs to get their heads around that changes need to be made in that area so it will be, requires research, education, changes to codes and to design practice.

The stair separation is a relatively obvious one and the department has already signalled to local authorities that they need to address that issue. I do not think we need to say too much more about that. Having a, allowing for a large potential movement perhaps higher than what your analysis would show is probably an important first step. I would say myself though that inherently for me as a designer you can never guarantee that a stairwell will be a safe place because it is linking two, you know, two floor diaphragms which will want to move separately, differentially and so some movement has to occur. Now logically no matter how well you design the detailing and the separation there is potential for something to go wrong so my, I agree the aim should be to make sure a stairwell is a very safe place and can provide access afterwards but I think it is also important that part of education the public at large is people should not rush to the stairwell during an earthquake because it is a safe place. You should always, the message should be wait until the shaking has stopped before someone checks out the stairwell and then use it for a safe egress. Look, I think others would disagree with that view but I think just from a practical point of view when you are joining a stairway between two floors which are under, undergo drift that caution is the preferred way to go.

The fourth item I have got there is floor depth walls. It is not a very typical situation though these are the transfer beams we are talking about where it is effectively like a shear wall part way up a building and it is obviously not a desirable thing in a seismic area but it is, it is worth noting in design practice that it is an undesirable mechanism. It is, when people want to, it is normally used when people want to do a load transfer so that you might have a structure where, a tower structure over a podium where there might be, you know, a large public spaces below, a cinema or foyer where the architect wants a clear uninterrupted space and will ask the engineer to do a transfer structure to transfer the gravity loads to a wider grid spacing and in that situation sometimes engineers will come up with a solution of being effectively a dead beam that is, goes from floor to floor so that does require extreme caution in that situation.

And look the next one is sort of on a similar theme. It's really about design rigour for displacement induced action, so it can cover a number of things. That actually covers stairs but also covers secondary structure frames that are designed to, not designed for the earthquake actions, not designed to be relied upon but must take the compatibility deflections that will result from the earthquake actions and it can be things like stairways, fire escapes. It can also be what's determined non-structural frames, non-gravity frames, seismic frames, pre-cast panels, all sorts of things. So I think if I could generalise, designers have had a reasonable appreciation of that over the last 10 or 15 years particularly in pre-cast panels on the perimeter of the buildings, there is a good understanding that they need to be separated from the structure in some wa. But certainly between, between the 80s, mid 80s mid 90s there were some gravity frames which were inappropriately detailed for the amount of displacement they were going to go to, not in this building but in other buildings.

And in frames supported on cantilevers again this a pretty bespoke type problem to this building but it is, when you've got large cantilevers the actions that can occur to the lateral displacement can be unexpected to the unwary designer so it's again some sort of signal that there can be an issue there.

And the fourth one which I've, I've put in italics because it's not in my report but is an issue that has come up in discussions both with Professor Fenwick and with Mr Holmes and I think we'll talk a bit more about that during the perhaps in the panel discussion and the next slide. I will talk a little bit further about that if I may.

So that, I've just some headings here that I guess we will discuss perhaps during, some of these we might discuss a bit further in the panel discussion this afternoon but they are issues raised in the June review of our report. One is the importance of consideration of bi-directional loading. I personally don't have too many qualms that that is reasonably well covered in the New Zealand codes and reasonably well understood by the modern designer. Obviously in this case when this building was designed that wasn't taken into account but I don't think that's a systemic issue. On the performance of the, of the other walls I think I've deal with that the other reasons why the other walls in the building performed better or did not fail. The likelihood of stair failure I think I've also addressed that. P-Delta effects. That's an issue where deflections, lateral deflection is, gets to such an extent that if you like the eccentricity of the load starts adding to the horizontal forces on the structure. That is, it is well signalled and I think reasonably well addressed in the code. There are certainly questions of, of how to probably allow for that in modelling. That's something Professor Pampanin may address. We don't think it was a particular issue in this building really because of the shortness in duration and the relatively small displacements prior to the wall collapse. Vertical acceleration effects I think is something that's certainly worth quite a bit of discussion in various forum and maybe we as designers need to know a bit more about when it's likely to occur and what we have to design for. Under the contemporary code for this building, the 84 code there was no requirement to consider the effects of vertical acceleration on the primary frame. There was a requirement to consider it on cantilevers so it's always difficult looking back but I suspect that if, that I would have followed what the designer had done in terms of not considering vertical, the effects of vertical acceleration effects when, at the time the building was designed. It is a bit, perhaps a bit different now. I guess the, to perhaps counter that is the, on the, again on our wall D5 to 6 it had a large contributing area and quite a bit of that area was cantilever so that it was a cantilever type load so there is I guess there's the likelihood that excitation of that load could add to the, to the vertical loading on that column. The other thing that perhaps is worth saying about vertical acceleration and this I think has become clear I think from seeing the preliminary report on the CTV and that is, which is, which has looked at quite, in quite detail the, the timing of the, of the horizontal motion and the vertical motion and of course the horizontal motion particularly for a building like our one has quite a long period whereas the vertical motion is a very short period and a rapid fire so you can imagine that while the building is in extremis in one direction while it's out there at a high level of displacement it could be feeling perhaps a number of violent cycles of vertical acceleration and in simple terms you might almost feel like a pile driver while the building is at extreme displacement. So as I say I certainly think that is you know where vertical accelerations are deemed to be likely then there more direction both through standards and design guidance would be a useful thing. The current code 1170.5 is still reasonably reticent. It does require vertical consideration to be taken into account for parts of buildings, again that's cantilevers and individual portions. My interpretation of it though is that it does not require vertical acceleration to be considered on the building as a whole unless you are undertaking an elastic time history analysis. The second to last one there is the, is the speed of loading question and that is an issue that you know we had a very violent event here in February. It was very short and sharp, a few cycles but very, very rapid but high accelerations and vertical accelerations contributing. A lot of the failures or yielding that is demonstrated in buildings all round the city seemed to be not what, they look very similar to the results we've seen from university testing over the years which tend to show numbers of yield lines at hinge positions whereas a lot of the ones we've seen tend to have been a single, a single point of failure and the suggestion has been that the speed of loading has, is an issue related to that and so maybe that's something that is something for the researchers to consider.

COMMISSIONER FENWICK:

Q. There are a number of papers on that topic and they do exactly what you say. They change the deflection characteristics of reinforced concrete structures tend to be more brittle, cracks wider spaced and so on so yes?

A. Yeah.

Q. It's a nice point, research, more research required, perhaps a bit of literature researched to assess I suspect and perhaps we need to start incorporating that you know in the standard so it's a good point.

A. Yes

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A. Yes, but then how to translate into a standard or design practice is the follow on from those papers. Just ratcheting is a device which has come up through the review and that's a loosely I guess I could describe it as the propensity for the, for a structure under psychic loading to move predominantly in one direction rather than to be, to move about a mean position if you like and you could consider that it – ratcheting might occur in a horizontal cantilever if under the seismic loading the beam will yield, so that each time there's a downwards motion the top reinforcing of the beam will yield, the beam will drop a bit and that will be repeated during the cycle, so although it's withstanding a load, it's going to keep to move down, so that's a simple way of looking at ratcheting. In the case of the Hotel Grand Chancellor, it's been suggested because of the large cantilever over the lane that the building had a propensity to if you like, for the yielding to be greater as it moves to the east than when it moves to the west and this is part of the issue that what

Professor Fenwick has brought up, so that when you consider that joint on the grid line D, that's on the edge of the cantilever. The cantilever moments from the beams, this is in the upper tower frames, will cause the frames to yield more effectively when it moves to the east than to the west and on the return cycle, if you like, the gaps aren't being closed or the yielding doesn't occur to the same extent because the demand on that reinforcing is lessened by the cantilever effect and so it could move to – in one direction, that is the east. I think it's a very real issue, we don't believe that it had any significant effect in this case because of the short duration. It's a time but the more cycles you have the more propensity it would have to move in that direction, but it is certainly something that perhaps practitioners haven't thought about much and I don't know that the code does either, so it's certainly an issue to perhaps, worthy of discussion and dissemination. Finally I just make a little comment that I perhaps should have made earlier when we were talking about the way we'd analyse the building and we didn't undertake, we did not undertake an elastic time history analysis on this structure. For a number of reasons, one is that I guess we had the advantage perhaps to compare to the PGC and the CTV in that the building hadn't collapsed and we could very clearly see where the damage was, and so we were able to concentrate our if you like examination and analysis on those elements that obviously did fail and in fact it relatively soon became apparent what the issues were and we could effectively work out a lot of the actions on the wall effectively from statics, that is the summation of the gravity loads and the evaluation of the likely over-strength shears resulting in additional axial actions on those critical walls and columns, and I think the – on that basis we had discussions with the department on it, that we felt it didn't justify doing for an elastic time history on that, on this structure. What it perhaps would have given us if it had been done is perhaps a better understanding of the likely deflections, but in this case it would not have affected the summation of why the wall failed. It also of course is extremely complex structure, you can imagine with all those eccentricities to model that with great confidence would have been a very major and time consuming process so I don't feel the report if you like is lacking anything because of that but it's a question that may be raised and I think that's it.

JUSTICE COOPER:

Q. All right, thank you very much. Now there may be some questions, I'm not sure if you'll need – we may need these slides at some stage, but Associate Professor Fenwick.

COMMISSIONER FENWICK:

Q. Thank you Mr Thornton, that was a very nice clear demonstration of the diagrams in particular were very nice and clear and your description of what happened. The Commission as you know is required to have a representative sample of buildings, look at them and say why they failed and why they didn't fail, so we can get an idea of what needs to be changed in the design process and codes and checking and so on. So we perhaps have to dig a wee bit deeper than you have, so I really want to dig a little bit more into this to try and gather more information so we can pick up features which might have contributed to this collapse and then we can perhaps transfer those to other buildings which are either new or perhaps indicate where we should concentrate our efforts on retrofits, so a little bit different I think from the aims of which the DBH had in setting this out judging by this report and the PGC report. So we'd like to make use of your expertise to dig into these, if you perhaps –

A. Sure.

Q. – go along with that. So I've got a number of points through here and a number of these you've answered as you've gone through and some you have answered, and I would like to dig a little bit further into those answers. I've held back from going through at the time because it was easier to do it in sequence. On, well it's actually in BUICAS0003.15, page 15, in the first bullet point on that page which hopefully will come up in a few seconds.

JUSTICE COOPER:

Q. I should make it plain, this is your report, this is an extract from your report -

A. Okay.

Q. – to the department.

COMMISSIONER FENWICK:

Q. Sorry I should have pointed that out.

A. Page 15, yes.

Q. On page 15 you said the maximum possible displacement, average displacement and you've explained that the average was the – you took the response spectrum, the full response spectrum was in the direction of interest, you added the values up and divided by four so you've got an average response spectrum and you're using that analysis, you said the maximum possible displacement 700 millimetres, it's in the September event, and in February it was 1,500 millimetres average displacement, and those displacements as you have indicated are first mode displacements or something close to first mode displacements at about 70 percent of the height of the building of what you've called the centre of mass, which I might call the effective centre of dynamic centre of mass. So you've then gone on to say that the response to the September earthquake did not match what was expected by the – from the spectra. Now in the – I wonder if you could just highlight why you felt it didn't match that, can you – you said that the engineers' report who inspected that reported back, but can you just highlight why they didn't think it had gone as far as they would have indicated it, or expected it to go.

A Sure, because when a building displaces, undergoes displacement, there's designers in ideal world understand how far the building is likely to displace in an ultimate event, the code event. They don't design the building to get to that point without any damage or elastically, really for on the basis of economics, so this is an event that maybe occurs once every 500 years for a typical building, what we call an importance level 2 building and so designers use available ductility so that when an earthquake occurs up to a certain level of earthquakes, a small level earthquake, the building will remain elastic ie undamaged. Once it gets beyond that it will go into the post-elastic or plastic region. The aim being for the building to be able to withstand the maximum displacements that effectively the earthquake shaking demands of the response of the structure without collapse but not without damage and once you've passed that elastic position then obviously damage will occur and that's when you call upon your insurers. Now when you pass that elastic point and you move into the elastic damage occurs in the structure and that is evident to someone observing the structure really by the way, in a concrete structure it would be the extent of cracking, the widths of cracking and the distribution of cracking. Now that was looked at by the engineers that you will hear from tomorrow.

COMMISSIONER FENWICK:

Q. Did they rip off the lining so they could measure the crack widths?

A. In a few locations they did, um, there wasn't an extensive amount of that.

Q. And they measured –

A. But you would also perhaps expect other damage to, you know, to the partition walls and to the façade pre-cast panels so that and there, I think in one of the reports there is a picture of some very minor, minor cracking. And even, even when you look now at some of the frames that the cracking is not what you'd expect to see with the ductility three or four had been achieved in some of the frames.

Q. I should talk to those engineers because I've tried in the lab and found it extremely hard to do unless I've got strain measurements but I would just, just wonder whether you have considered it was in the north/south direction predominantly wasn't it?

A. That's my understanding.

Q. Yes.

A. For the September event.

Q. And in the north/south event you have the beams along the outside, the main perimeter sort of beams which were resisting the lateral load alongside them were pre-cast, pre-tensioned units in the floor?

A. Correct.

Q. And the floor was tied into these units, the units were fairly shallow and the beams were fairly, fairly deep but the other thing is of course the plastic hinges, you haven't mentioned this, but the plastic hinges were detailed to be away from the column face weren't they?

A. That is correct, in that north/south direction.

Q. Yes.

A. Not in the east/west direction.

Q. And in the east/west as well. Now that –

A. No.

JUSTICE COOPER:

Q. No, no you don’t agree with that?

A. Not in the east/west only in the north/south direction as the offset hinges.

COMMISSIONER FENWICK:

Q. I will have to look at the drawings again. I interpret them in both directions, okay.

A. I think – that's my interpretation.

Q. North, north/south you say they were remote?

A. Yes an offset hinge so that the yielding doesn't occur at the face of the column but at effectively a beam depth away from the…

Q. Right okay. Now when those plastic hinges form they generate tension on the top surface and of course you've got the pre-stressed units going past it so these pre-stressed units will act as a spring won't they to tend to close up the existing cracks?

A. Ah –

Q. Don't you think?

A. So you're saying that, well the – maybe are you suggesting that the diaphragm reinforcing the mesh and starters will perform elastically and will close up the gap again after?

Q. Yes I mean the floor reinforcing, the floor itself consists of reinforcing mesh which of course would yield if you pulled it too far but because the crack is not at the end of the member you would get some as you stretch the beam you will also start to stretch the floor and the floor is tied into the beam so the floor of course is acting as a natural spring because of the pre-stress of it so wouldn't it tend to close up, I won't say it completely close up but wouldn't it tend to reduce the size of the cracks don't you think, I mean…?

A. I'm, I'm struggling to see why it would be different from an offset hinge than it would for one at the base of the column.

Q. If the hinge is at the base of the column then you only have a very short length of pre-cast unit tied in behind the thing and so you've only got a very short shear transfer strength, length over which to transfer your shear force. If you've offset it say you've got something like a metre in length you can transfer this tension force by shear (inaudible 14:27:59) into the beam so you've got actually got effective spring acting there if you're going, going to the column face of course the length, the strength of your spring is very much greatly reduced?

A. Yeah I'm not…

Q. Perhaps, perhaps we should just place – did you consider the influence of the pre-stress in the floor on the performance of those beams and the effect it might have had on the cracked bits?

A. Not specifically. One of the things is that of course is those ribs are at 900 centres and so if you consider TBM action then you have got a bit of displacement from the beam across to that first, first rib. What, what we did do is in the, I mean we only looked at it in the post-February event and we did pull up the carpets to see what sort of cracking there was, what sort of floor dilation, what sort of, what sort of growing and there was, it was extremely minimal. We did, we could find some cracking around, at the beam hinge area but it certainly wasn't great I mean and that also, that sort of led us to part of the, our, our conclusion that the you know there weren't great ductility demands made on the, on the structure in either of the earthquakes. Obviously when in this February earthquake there was that extreme damage in those bays where the vertical displacement occurred. I mean the, I guess the view that you're suggesting and it's worth some debate in more academic circles I guess but of course we're all designers, now we're very aware of the potential effects of dilation where the, where you overcome, you make the floor and its reinforcing yield and you're stretching it.
Q. Well thanks, I think the answer is you didn't consider the possible effect?
A. Yes.
Q. And that's something perhaps we need to look into –
A. Yeah.
Q. – to see if it does have an effect.
A. Yep.
Q. And to something to allow for, and of course we have done that to a limited extent through testing in the last 10 years. In the February earthquake your table on the side you had it before, sorry on page 15, that's 0003.17 can I have that one?
A. Is it 16?
Q. It's the one with the – that's it, the diagram, if we just concentrate on the figure 7.
A. Oh yep.

WITNESS REFERRED TO DIAGRAM FIGURE 7

Q. Now the predicted displacement from the table we had up before from the February earthquake said at the centre of mass the deflection, predicted deflection was 1050 millimetres. And in this diagram you can see and in the table on the previous page that's 116, we don't probably need to look at it at the moment, you have reduced that 1500 millimetres to 500 millimetres? I beg your pardon 1050 millimetres displacement at the top and that's been reduced to 500 millimetres at the displacement to the top. So could you sort of indicate to me please how you came to be able to reduce the predicted spectral displacement which is for a first mode I assume of 1500 millimetres, I beg your pardon, 1050 millimetres to 500 millimetres at the effective centre of mass, effective dynamic centre of mass or centre of mass is what you called?

A. Well the 1050 comes from the, the spectral, the spectra which, so that's –

Q. That's the first, first mode predicted, first mode valued?

A. Yeah. And that's -

Q. And I agree –

A. – that's a peak one. Now I guess it's a question of what, what we've suggested is that the building softened to if you like over the hump so whether it actually, and on the, at four seconds it is, it is down round 600 if you look at the spectra.

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Q. So how did you reduce your theoretical value? Did you say well I don't believe it so, I mean, how did you indicate, how did you halve it?

A. Well by saying –

Q. What was the logic for saying this?

A. – that at the effective period, that was the maximum possible displacement if you were combined you know at that period of whatever it is, 3.2 seconds. You need to refer to the spectra as well, displacement spectra.

Q. I have got the spectra in front of me.

A. Yes, so it was possible according to the spectra to get a displacement of 1050.
Q. Right, so how did you come up with 500 millimetres?
A. Well because we are saying that that was not the response of the building because it has softened.
Q. Normally when a building softens its displacement increases, doesn't it? Isn't that the usual logic?
A. Well it depends which displacement spectra you are looking at but this is suggesting here that after three seconds the displacement reduces.
Q. So on your spectral value then you would say that you are actually assuming the period was about four seconds?
A. Yes.
Q. But you have got no general, I mean this is just your feeling what it would be?
A. Well it was, I mean it was, it was looking a bit at, one of my colleagues did some of this analysis and looked at the likely effects of some foundation softening or all those things that can be looked at but also looking at also at the level of damage that occurred. Now to get that level of –
Q. Level of damage –
A. – displacement –
Q. – in this February earthquake?
A. Yes. Yes, even if you go and look at it now obviously there is that extreme damage relating to the collapse of the wall on D5 to 6 but in the upper frames, if you take that damage away the damage is quite small, the apparent damage. It is certainly I mean this, to get to that sort of displacement would have required a, you know, ductility demand 6 or 8 or something, now it is I think it was reasonably apparent that that, that there was no such demand made on the structure.
Q. Now how did you allow for P-Delta action within this structure? I mean this appreciable displacement on there, did you include any allowance for P-Delta actions?

A. No we did not but it is, so we, in the again prior to the failure mechanism and at the displacements we were looking at they are not particularly significant particularly in terms of the failure mechanism and so I guess, you know, we were particularly, our brief was to see you know the cause of the failure and to be able to understand that quite clearly and we identified if you like where one bit of the building did not meet the code requirements. I can't hand on heart say that no other part of the building didn't as well because to go through every part of the building and check whether it complied would be a very long detailed process so I guess we as I had made the point earlier we did have the advantage of being able to hone into the points of obvious distress and failure.

Q. Just let us have a look at that figure, at the top there assuming you have halved the deflection at the centre of mass and I think there are logical reasons for doing that, I have probably may have alternative ones which might consider later on for doing that rather than yours but if you look at the top of the tower you have got a deflection up there of around about a, just over 1000 millimetres, if we look at the top of the shear wall the deflection is about 50 millimetres so the average drift between those two heights, the top of the tower looks about 68 metres high from what you have got there, it looks about 24 metres high at the bottom there, just reading off the graph, so over that height you have got a total drift between the top and the bottom of the frame part of about a metre which if you work out the angle to the vertical of average angle vertical of the columns it is .23%. Sorry, 2.3% is the gradient so it is about 23 millimetres per 1000, now the axial load roughly at the bottom of the tower very crudely is round about 79,000 kilonewtons. That force is being supported, do you agree, by the columns -

A. Yes.

Q. (inaudible 14:39:35)

A. Yes.

Q. The columns are at your figure an average inclination of roundabout this 2.3%. Now when it is deforming of course the inclination will vary over the half cycle between zero and back upright to 2.3% as maximum so perhaps we are looking at possible P-Delta effects we ought to consider somewhere about half so I've said right well let's half it and take 1.3% drift, .13% drift, that means that the equivalent lateral force due to the inclination of the pre-stress force due to the (inaudible 14:40:17) inclination of the gravity load of the columns would be equal to 1.3% of the weight of the building so the question I have really got for you is if you agree with those figures there are about the right order you have stated elsewhere that the gravity load, the base shear strength for the walls was 4.8% of the weight of the structure, elsewhere you have indicated it may be higher we will come back to that in a minute so if that was 4.8% of the weight of the structure we are now reducing that by P-Delta actions to 3.2% weight of the structure. Would you have considered that to be a negligible effect?

A. No, no, if your numbers are correct, no.

Q. So do you think P-Delta actions could they have had an influence on this or are you trying to say the structure would have actually failed long before it reached the (inaudible 14:41:20) in the top?

A. I think that that effect has in this case, any P-Delta effect would not have had any significant effect on the loads on wall D 5-6.

Q. Thank you. It is not an action you felt you needed to explore at any rate so –

A. I think that –

Q. – you are not too sure what effects it presumably –

A. Yeah –

Q. – you can't be too sure what effects it would have had or...?

A. Yeah, no, that's right, as I say we were concerned with what were the effects on wall 5 to 6 and what caused the failure and by adding initial displacement sure it adds greater demand on those frames effectively –

Q. You see you realise I am trying to bury a bit deeper of other actions –

A. Yes, yes.

Q. – which may or may not have contributed so we can transfer them to our knowledge because I mean back in 1984 we calculated P-Delta actions differently than what we do now. What I had done was more or less what we do now.

A. Yes and I am, look I can't answer the question whether they but I think it is probably quite likely that the original designers did take account of P-Delta effects on those upper frame but that again that wasn't so much interest to us.

Q. In section 10 in your report we can get this from BUICAS 161.0003.29...

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Q. Section 10 is near the bottom of the page and you state there the dependable design strength I think comes out in application to the Council. I sort of saw, and you've quoted it here, as .048G which is the lateral force co-efficient, 4.8% of gradual load acting sideways and you've got a different figure for the walls - .09G which again is the design figure but you've said that the probable strengths are .8% and 10% of the weight of the building. Now is the 8% and 10% are they for the frame and the walls or are they just in the range of .8, 8% to 10% or can you tell me what they are and how you made those assessments?

A. I think, look, I think it does correspond to the frame and podium.

Q. Okay, I mean I'm quite intrigued because I sort of did a little hand analysis on it, a bit crude, plus or minus 20%, and I got the lateral force drifting to the west as five and a half percent and the lateral force drifting to the east as 3.2% as a dependable lateral strength. I took out the code torsional actions from that to give me an equivalent base shear strength so I was quite intrigued when you were coming out with a figure which was very much higher and I just wondered, you know, how you'd actually arrived at that figure. I was quite pleased when I got that because when I added the two up and divided by two I get nearly the 4.8% so I thought that looks about right, just someone's forgot about the ratcheting effect which, tendency to move to one side that goes in this particular structure.

A. Yes well on that latter point I did a pretty rough analysis too and in terms of the shear that might be induced from the out of balance cantilevering action I wondered what effect that has through the tower frame and at the base of the thing it potentially is sort of 20% towards the east if you like so it gives you, in terms of the available strength, it lessens, as we've talked about before, the strength in an easterly motion compared to the west.

Q. Sorry I'm having problems hearing you.

JUSTICE COOPER:

Yes I am too.

A. Sorry, in terms of the effect of the potential ratcheting effect from the cantilever between grids D and E, yes, it does have an effect on the shear through the tower potentially. In terms of the problem, well I don't know how you worked out your numbers but we were, I guess we do assume that perhaps all the reinforcing is at a higher than the minimum yield strength that is specified for the reinforcing for example.

COMMISSIONER FENWICK:

Q. And the sort of typical value, I mean the design strength is based on a lower characteristic -

A. Yeah.

Q. – strength of steel and the average steel is stronger?

A. Yes.

Q. Does that account, that double –

A. – Well that can account for a reasonable amount of it.

Q. How much would you think it would account for? The typical test results we've got now you add on 8% and that's the difference between the mean and the lower characteristic for typical current reinforcement?
A. Is it as low as that. I would have thought a bit higher but I bow to your knowledge on that but look as to these numbers, what we came up with I'm not sure how robust they are. As you know it is extremely complex and structure so there's a lot of ways to analyse it and a lot of assumptions and engineering judgements to make to get to any particular level but I think we're in the same order of magnitude.
Q. I don't think 3.2 and .8 or .55 and 1 are in the same but I agree it's a complex problem. You have to make a few assumptions in deriving those forces. You can't recall the calculations you did to come up with those numbers?
A. I didn't do them all myself but I mean I could do more examination to that if you think it's important for where you're going.
Q. That could be valuable to do that if you could look back and find out how we came to that –
A. Sure.
Q. – It would be interesting to, you know, know why the strength was so much higher than the design strength and again quite where my calculations have gone wrong if that's what the case is.
A. So just so I'm clear there. You're saying what numbers?
Q. Well I'm allowing for the ratcheting effect, tendency effect there which adds a thousand kilonewtons one way and subtracts a thousand kilonewtons the other way and the figures I've come up with are drifting to the west. It's 5.5% of WT and drifting to the east is 3.2% of WT and that is to the moment resisting frame part of the structure, not the base of the whole structure.
A. Okay whereas we've said .8 and you've said an average of .4.
Q. The report indicates 4.8% for the tower part, for the moment resisting part, and 6% for the wall part?
A. Yeah. So if I could take your average which is about –

Q. About 4.8%.
A. And we've said 8%?
Q. Yes. So on page 16 of your report, that's .0003.16 you've indicated that the ductility range was of the order of, well I've said 2–4, well the values quoted in the report are ductility range is 2.3 and 3.3 for the September and for the February earthquakes.
A. Sorry, where are you looking there?
Q. On that table. (ref: last two boxes). The last two boxes you've got (inaudible: 14:51:36) displacement okay and you've got 3.3 there down at the bottom?
A. Yep.
Q. Now did you consider using the inelastic spectrum to see what the demand would be in terms of those sort of ductilities, if you had that sort of ductility you could look back and see what the sort of order of displacement would be. Did you consider doing that?
A. Sorry, doing some inelastic analysis do you mean or...?
Q. Well you didn't have to do the inelastic analysis. It was all on our website and done for you. We ran our response spectrum for elastic and inelastic analyses for different ductilities levels. So if we could look at BUICAS 161.0038.7 and you'll see the elastic and inelastic spectra that are on our website for the February earthquake which will give you an idea of the sort of deflection you could get, (inaudible: 14:52:57) Could we turn that round.
A. Are these the ones Professor Carr did?
Q. Yes.

(slide view rotated)

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COMMISSIONER FENWICK:

Q. Can we rotate it round – thanks. Now if you look at the red corresponds to a displacement ductility of two, and the green, a displacement ductility of four and the blue which looks like black is a displacement ductility of six. You can see looking at that knowing that the range of periods is really in the range of 2.5 probably to 3.5 or something like that, the actual displacement ductility is almost what you assumed isn't it, it's about 500 millimetres, you didn't need to go to any of these explanations or possible other accounts do you think just to – working back though, I mean this is where the equal displacement approach does not work in this particular case, but did you consider looking at these to assess what (overtalking 14:54:44)?

A. Well we didn't until you drew our attention to that paper, but when – we, on the February earthquakes, the results that we got we felt did match those reasonably well but not for the September ones.

Q. So using the inelastic spectra can I draw the conclusion that it would match your results?

A. It would for the February event.

Q. And but the fact it didn't in September we have discussed that as well.

A. Yes.

Q. And I think there may be other possibly issues (overtalking 14:55:25)?

A. Yes, and I think Professor Pampanin may be keen to shed some light on his views on that.

Q. Now you calculated the axial forces from the tar which were imposed on the wall and you correctly identified, of course you've got when the earthquake comes in one direction it can throw the load onto the corner columns and this additional way of doing that is to calculate the what you think are the highest moment capacities of the potential plastic damage and work back through statics to calculate what axial load they can induce due to the seismic actions and add it onto the gravity actions. Now that process of course was in the standard, though not quite, certainly for columns, it really didn't say you'd used it on extra columns and walls but I think that's fair enough to say well we could take that, I think most designers would say we'll take that process and use it to calculate the loading as you have done. The question I have got for you is, we used a different process then, we changed the criteria between what was in 1984 and what we use in 2006, so my question is how would that have influenced the axial loads we kept from these seismic VOE moments, capacity moments. Again I'm –

A. I don't think there is much change in that between what the codes are then and now, it's still – you were then required to consider bi-axial concurrency and do now. The modification factors for the number of storeys things may have changed a bit, but ...

Q. No, they're the same, no the point I was getting at, the way you calculate the over-strength moment has changed and I wondered what significance that had for these loads. In 1984 you were told that for design for strength or over-strength you can include the reinforcement in two slab thicknesses on each side. Now we say, oh you've got to go back and you've got to include the action of the floor on quite a wide region on each, a very much wider region than we had before and I'm just wondering how much influence that had on the performance? From the point of view I mean if we've got existing structure which we've designed to an old standard, do we need to look at it now in terms of what we know now happens with this floor with the wider section, wider floor interaction, particularly the pre-stress has a major effect on it than the floor.

A. No we haven't considered the difference between the two codes in that regard.

Q. So you just did this according to the –

A. The 84 code.

Q. 84 code. So in terms of our current one then I'd say it's probably a bit on the low side, yeah. Except I think you mentioned in your presentation you assumed over-strength or the reinforcement was another characteristic strength which in fact probably in analysis of existing buildings it's probably more reasonable to take it as a mean strength, average strength wasn't it?

A. I don't think that was the impression I meant to give, we did it I guess in terms of the code of the day which was – it was just to take the yield and apply the, like the over-strength factors in, (inaudible 14:59:40), usual stuff.

Q. I think in your presentation you said yes we need more work on vertical excitation and yeah –

A. Yes, one other point about vertical excitation which perhaps I meant to say this morning, in a typical situation for a, if you like a non-cantilever situation or an internal column you have, your gravity case which has – the loads are factored up by the you know 1.4 dead or 1.2 dead as it is for the current code and project for live, and when you're considering the earthquake case you take off those factors so there's a sort of a cushion there to accept those. I think upon reflection when you look at the end column of a frame where that you have the high seismic moments all induced axial actions from over-strength shears, they're already adding to the load so it's if you don't have that buffer from the gravity load factor so that I suspect it's probably something for a bit of research but to consider an element of vertical acceleration on end columns or frames I would have thought is a good place to start.

Q. Yes. We currently have a factor in there for earthquake actions which you said .85 times theoretical crushing strength or something which gives you another .85 and a .85 takes it down to about .72 FC dashed on average and probably we should be dropping that figure and that would probably help cover it wouldn't it, do you think, I mean that sort of thing?

A. I guess that's one approach of doing it.

Q. Yes I agree, there are cases where the gravity load doesn’t give you a –

A. Yes, it doesn’t give you a session.

Q. A margin, yes. Now you used a modal response spectrum and equivalent static analyses to check this out, or just modal response spectrum analysis or –

A. Both.

Q. Both, okay. Can we just go through this ratcheting effect?

A. Sure.

Q. So if we can have BUICAS161.0038.14? So that's illustrating how we – do you want to, would you like to describe that to us or do you want me to go, like me to go through it? I think we both agree on that but it's ...
A. Well this particular slide here is, well the bits to it, but was there a particular bit you wanted to go through or the more the –
Q. Well it's just really the basis of the response spectrum analysis where you take a single degree of freedom structure which is the top middle diagram, simple lollipop structure, you play the earthquake ground motion at it, you would assume normally 5% discuss damping or other damping and you calculate the response of that structure to that motion. Now if it's an elastically responding structure then it will have a load deflection characteristic such as the graph on the right where it's the, the load or lateral force, equivalent lateral force acting is on the vertical Y-axis and the displacement is on the horizontal axis and the assumption which is wound into a lot of criteria in the code is that the inelastic structure, ductile structure which is shown dotted will deform approximately the same displacement as the elastic structure and of course the Grand Chancellor we've seen in the February earthquake that that wasn't the case but normally it is reasonably the case, that's based on the assumption. Then one can also look at the, what happens with time from this sort of structure and I've shown below the diagram on the right-hand side lower half, the sort of elastic response you get on one of these structures in the diagram towards the middle of the page and below it is the type of response you'd get from a inelastic structure and the peak displacements occur at different times, different places but they're roughly the same and because that elastic structure has equal displacement, equal stiffness and equal strength in both directions it tends to vibrate round about the, the Y-axis. Do you agree with that description or would you like to qualify?
A. Yes I do, yes.
Q. Can we go to the next one which will presumably be number, page 15 on the same series 0038.15?

WITNESS REFERRED TO DIAGRAMS

Q. This shows a plan of the Grand Chancellor building north is to the left and east is straight up and the outside bay of the Grand Chancellor building was as very clearly demonstrated in the sketch was cantilevered out. One or two things to notice about this the major part of the lateral resistance system was taken by the frame on, I can't – there we are there, on A and on D and D's a slightly kinked one but that's where the lateral resistance occurs. If we can go on to the next slide, so that load hangs off the building, we go on to, it will be 16 won't it, I think we have, that's 15, so go to 16. That shows the building and you can see the, on the left-hand side you can see the outside bays is supported by cantilever action with the main frame because there's a connection missing right at the bottom of the modal (inaudible 15:06:47) just above the walls. The axial load acting on those cantilevers induces a bending moment about the, for the rest of the structure so for the, the three columns which you sort of see there, sorry the four columns at the bottom you can see that's where the lateral resistance really comes from, from those and you can see that the, that line the gravity of load is eccentric to the centre so the whole structure is going to try to lean over to the right to the east in effect due to that gravity load. And if we look at the moment gradient because each cantilever comes in at a later moment that's equivalent to applying a lateral force at the top of the structure of 1000 kilonewtons and I think you've come up with the same number?

A. Yes I don't know that I actually agree with your points of reaction shown at the bottom there because it may be it could have happened on the first two columns rather than right across and it is at each individual level you have a moment in the cantilever beam, the external, on the external face of the column and some of that is resisted by the internal beam adjacent to it and the outer balance is resisted by the, a combination, based on the stiffness of the columns and the beams.

Q. Would you agree though that it's a free body standing up and if I take moments about the centre of that structure at the bottom there must be an eccentric load, acting on it it's going to induce a moment. Doesn't matter how that moment is carried, carried by one column, more columns, all of them, that moment must be resisted for statics. Would that, would you agree with that?
A. Oh absolutely yeah, yeah.
Q. So I mean whether the load is high on the two outside columns or two inside columns –
A. Yeah, yeah.
Q. – I don't think changes the moment equilibrium or the lateral force calculation does it?
A. Yes, I mean it doesn't, it doesn't add any base shear.
Q. No, no it doesn't add any base shear but it does give a tendency for it to move sideways?
A. Yeah.
Q. That tendency for it to move sideways of course is resisted by the frame the moments in the frame?
A. Yeah.
Q. So I mean it's still tending to tip over that way?
A. Yeah, yeah, no and that's what I went through this morning.
Q. If I applied earthquake loads and the direction it's tending to tip it's got to have lower strength than if I apply earthquake moments in the terms in which it's trying to upright it. Do you agree with that?
A. I do yes.
Q. Okay. So on that basis then if it goes inelastic because it's very much weaker in the east direction than the west direction and I quoted my figures before 3.5% lateral which we may disagree on but 5.5 but we both agree there's a difference between the two values it's going to, if there's any elastic it's going to move over to the east rather than to the west it's going to tend to drift towards the east?
A. Over a period of time yeah. Yeah.

Q. The period of intense ground motion for this just looking at the earthquake record now if it’s about 5% lateral force up there when the acceleration reaches .1 g you would agree that’s capable of producing inelastic deformation in the structure?
A. Yeah.
Q. Ground acceleration gets to .1 g, lateral strength .5% g. Do you think that would be capable of giving, inducing inelastic deformation, yes or do you think?
A. Ah, well once you’ve exceeded the, whatever the yield was which we’ve said is around –
Q. Right.
A. – 4% whatever it was, 4 to 5%.
Q. So if I look at the four earthquake records and I say well, over what duration, what was the time between when I first reached a peak acceleration of .1 g to when I last reached a peak acceleration .1 g and I’ve done this for the four records you used in your average for your analysis. I found the average period was just over 10 seconds?
A. In order to do what? Sorry, to…?
Q. Between when if you looked at the earthquake –
A. Yeah sure.
Q. – base, ground acceleration from the time when the first peak reached or exceeded .1 g, 10% of a g laterally?
A. Yeah.
Q. Which we’ve agreed could be potentially enough to cause yielding to occur?
A. Oh that's fine yeah.
Q. To when the last one occurred it was 10 seconds. Now that wasn't the end of the motion, the motion went on but it was less than .1 g. Now in that time you’ve got time for three, two, three, sorry at least three more possible cycles inelastic displacement haven't you? Well at least two and a half or three?
A. Yeah.

Q. So would you agree that ratcheting is likely to occur?
A. Oh yes but it may not be enormous in terms of its displacement. I'm not disagreeing with you but terms of the mag–, the other thing in this particular event it was at some time during that 10 seconds the wall failed and the response obviously changed.
Q. Yes. Yes.
A. Quite dramatically.
Q. So okay, so it could've failed before the structure ratcheted on the other hand it could have ratcheted it over some way. The question I'm asking is do you think that ratcheting had any significance on the wall?
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A. I don't think so because I think the dominant action on the wall was the axial load and the displacements at that effectively the level 2 level, the first floor above the thing, now I don't think the ratcheting would have a great effect on the wall podium structure so –
Q. Can we deal, can we just go back and consider you know each little item separately?
A. Sure.
Q. If it ratchets the rotations would have increased in all the plastic hinges. The rotations would have increased in all the plastic hinges. The rotations would have increased the moments would have increased, they would have strained hardened and got higher so the axial forces would have –
A. Well those are the ones we have allowed for anyway because we have allowed for the over-strength shears.
Q. No I am saying what happened in practice. If you just had a ductility of 3 you would have very small inelastic rotations, the ductility 3 displacements in structure are quite small but if it ratchets so you got higher rotations you'd have higher strains in your reinforcement, don't you think this, don't you consider this might have increased the moments which were resisted by the structure inducing axial loads –
A. In the, in the scenario you have described yes I agree –

Q. Now –
A. – but, but of course we did assume that the, in terms of our derivation of the axial loads that there was some strain hardening occurring through the over-strength, to get the over-strength shears but what you are saying is that perhaps the ratcheting sped up the process to get to that –
Q. Okay.
A. – to get the higher axial loads driving and I think that is certainly feasible that that is what happened.
Q. So, I'm sorry, you're saying it didn't have an effect because in your calculation you had used high over-strength moments to start with and I am saying the over-strength moments would have climbed, probably not climbed to the values you used, so I am trying to bury down to what happened in other structures you see so –
A. Yes.
Q. – so you agree it might have increased the load but probably you have over-allowed for it already so –
A. Yes, in terms of the detailed, as you said, digging down to the sequence of what actually of the build up of those stresses what you describe is a good scenario.
Q. Right, thanks. Okay, now the ratcheting effect would have increased the displacements on the structure?
A. Towards the east?
Q. Yes.
A. Yes.
Q. And this would have increased the displacements acting on the critical wall?
A. I don't think so, not to any great extent, not at their level where we are considering it.
Q. The fact that you know when it ratchets giving it pushing, pushing the whole structure like this?
A. Yeah, when we looked at the, I mean there is very little evidence of major, of any significant yielding or movement of the walls at the ground

level so again it doesn't appear that there was great ductility demand on the lower walls so I guess what I am suggesting to you is that it may be that while you had the ratcheting going up on the top and increasing potentially increasing the deflection of the tower frames that may be that did not, maybe it didn't translate into deflection within the podium structure.

Q. Just a final thing then is moving that way, we know that the strength, I think we both agree the strength to deform in the eastward deflection is relatively low compared to the strength required to deform it in the westward direction so if we are looking at the shaking we might expect higher shears to be induced in the lower part of the structure due to deflections towards the west, ground shaking pushing it towards the west. Do you agree?

A. Relative to the west, yes, that would be higher towards the east.

Q. Yes.

A. Yes, agree.

Q. Can we have a look at BUICAS 161.0038.13? This is a picture we have seen earlier on. Now that shows the, I think the structure deformed to the east which is more or less what ratcheting would indicate it would start to deform to the east, I think we agree on that, but that wall has failed on a diagonal and it is displaced, is it displaced in the direction you would expect it to be given that the structure tending to move to the east? Can you see the picture of your failure there which is looking from the south to the north so the east would be on the side, the right-hand side, would you expect given the way the structure is deforming it to fall that way or the other way?

A. Well –

Q. Is that an expected solution? Is that expected?

A. Well I think the –

Q. Perhaps I should ask the second, get you to consider the second point as well. I think we agreed before that because the tower had a strength which was weaker moving to the east than moving to the west when we shake it, the tower part that multi-storey frame part, would resist a higher based shear when we potentially when we try to move it to the west, shaking equally in both directions but the structure will have a tendency to go one way but it resists automatically a higher load and it goes to the west so what would be the influence of higher load on that? What I am getting is, that is a diagonal crack in there, are we sure it is a crushing failure or is there something else coming in there? Can you just sort of consider it? I mean first of all is the direction of failure what you would expect or is there some other possible, do you think there may be some other possible explanation for that failure mode?

A. Well it is a good question and I think I mean, we have, I must say we have looked to explain what we have seen rather than to think about what is the most likely but I think we see in the report that the you know before the wall yielded in the, if you like the post-elastic demand could be to when the wall is, when the motion is going the other way so the wall is actually going into tension that there is very little reinforcing in that wall and so if you have got a combination of moment either in-plane or out of plane and relatively low axial loads so take away from it then there is potential, there was potential for if you like tension cracking. Now that would, if that is going to occur that is likely to occur at the top of the laps so one scenario is that the potentially there were cracks formed at the top of the lap, whether they did or not, when if you talk a thing about stress raises when that side of the wall so the east side of the wall is extreme compression then you have if you like compression potentially stress raises of from the reinforcing pushing up showing a discontinuity of stress if you like because that is where the reinforcing is stopping so there was potential for higher concrete stresses arising at the top of the lap and something has happened there whether it is crushing, whether it is perhaps related to a tension crack but once anything starts it is easy, very, in my mind, very easy to see that failure occurring because there was no transverse reinforcing going through there to stop that sliding of failure.

Q. Can I just bring you back, it is the lateral, the force out of plane thinking about there, it's gone on a diagonal yet the displacement at the top is the other way. What I am truly getting at is could it be a shear failure? Once you got very high axial compressions on a member the shear strength drops. Initially when you've got a wall or a column with axial load on that increases the shear strength because -

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A. Yeah.

Q. – the compression force (inaudible: 15:23:15) comply but when we start looking at this wall we've got very very high axial loads, as you've shown, on that wall quite dramatically. Now I'm just questioning it. Was it necessarily a crushing failure or was there some other mechanism. I agree that the bars were lapped and that might have acted as a small trigger to give you a crack at that point?

A. I think we said in our report that it had the appearance of a diagonal sort of shear failure. The shears on it in the out of plane direction are quite low so it would have to be a combination of high axial loads perhaps, a tendency to be wanting to buckle because of the slenderness. So there's a number of factors which led to it – the high compression loads, the lack of confinement, perhaps almost pushing it towards buckling and when it failed I agree it looks very much like a shear type failure with nothing to resist it. Exactly how you get to that point I think it's quite difficult to be precise in saying that was definitely there, the answer, but I guess that's where we're coming from.

JUSTICE COOPER ADDRESSES THE COMMISSION

WITNESS STOOD DOWN

COMMISSION ADJOURNS: 3.25 PM

COURT RESUMES: 3.45 PM

MR ZARIFEH CALLS:

STEFANO PAMPANIN (SWORN)

Q. Your full name is Stefano Pampanin?
A. Yes.
Q. And you're an Associate Professor in Structural Design and Earthquake Engineering at the University of Canterbury?
A. Yes.
Q. You were a member of the expert panel for the Department of Building and Housing and in particular reviewed the report from Adam Thornton that we've just heard about on behalf of the Department of Building and Housing.
A. (no audible answer 15:46:17)
Q. What I want you to do and what I understand you're going to do is you're going to speak to us briefly on that role and confirm the expert panel's view it took of the consultants' report and just highlight a number of matters that have arisen and I think we've had reference to your expertise already and highlight those matters and amplify on them, correct?
A. (no audible answer 15:46:48) (adjustment to microphone)

JUSTICE COOPER:

Q. Now I think Mr Zarifeh put to you, and you've agreed with him that you're an Associate Professor, but you're a Professor aren't you?
A. Say again.
Q. Mr Zarifeh said to you, and you agreed with him, that you are an Associate Professor. I thought you were a Professor?
A. It's a good terminology. In New Zealand the term is Associate Professor which was formerly a Reader so the title that you're addressed with typically overseas would be Professor is the title that you address on.

Q. My apologies Mr Zarifeh.

EXAMINATION CONTINUES: MR ZARIFEH

Q. And I think you've got a number of slides that you want to refer to.

A. I do and I think it would be the easier way to basically not only going through the discussion but possibly also facilitate what is going to be later on the panel discussion in a way. First of all these slides were meant to be helping or presentation of approximately half an hour depends on the Commissioner. If we do not have the time I'm very happy to go through quickly or skip many of them and then we can refer later on tomorrow to some discussion.

Q. Perhaps if we keep this portion of the evidence to the Hotel Grand Chancellor into a summary in brief form and we can come back to explore other issues if the Commission wants to or during the panel discussion tomorrow.

A. Yes, it is basically all in relation to the Grand Chancellor in the way that if I go back to the terms of references of the expert panel, Department of Building Housing expert panel, it is important to highlight that the expert panel not only were looking at reviewing the four consultants' report on the four buildings in discussion but also was to try to give a little bit of, have an understanding itself to give a judgement prior to provide recommendation on the context in which those four buildings were (inaudible 15:49:47) and specifically I have been assigned an additional task within the expert panel which was to prepare what is called a contextual report so it does make reference to the Grand Chancellor in the sense that the wall system, reinforced concrete wall system, of the Grand Chancellor can easily be identified as lessons learned or be prompting lessons learned in modification to the cause which is what basically the Royal Commission is going to aim at and in that sense I am happy to simply show what the recommendation (inaudible 15:50:27) There was an interim report, as you know, issued in late September but there also is going to be a more detailed and expanded final report and even without referring to the CTV building the recommendation which the expert panel has come out on the other buildings itself have been expanded and elaborated upon a little bit.

JUSTICE COOOPER:

Q. So when is the Department of Building and Housing report going to be completed?

A. Ah, I should leave the chair of the Department of Building and Housing or the expert panel to refer to that but I can say that has been completed. It is finalised and is now not yet out of public domain just because it's out for discussion from third parties, possible affected parties, so it is going to be released as soon as possible. It has been completed anyway.

Q. Well we plan to deal with as a separate hearing topic necessary changes to the existing codes later in the year and we really want this hearing to be focused on the Grand Chancellor. If there are other matters that are needed to explain why in the expert panel's view Mr Thornton's report was accurate and accept the, explained that the reasons for the failure of the Grand Chancellor building by all means go into those related matters but otherwise we want to keep this hearing confined if we can because we have so much to do and we are going to look at the wider picture later on.

A. It will be done and we can then discuss tomorrow morning in case you are interested in going deeper. So first of all the expert panel has reviewed, in the process we are starting from the beginning as you are well aware of, the process was not a peer review done at the end of the consultant report being submitted, it was happening in due course which allowed for the modification to be done, further investigation to be done or agreement of no need of further investigation or more complex analysis to be done. Going through the expert panel recommendation you're going to find that basically the report is fully supporting the same findings that the consultant Adam Thornton and collaborators have provided. There are some issues which are important to highlight further and they were in the expert panel not in the section of the Grand Chancellor but in the wider section and I try to go through these sort of discussion that has occurred, remembering that in the expert panel also the consultants like Adam Thornton were part of and so the discussion had a very good wide breadth. These slides are I think of interest because they were prepared actually in April and they were part of April/May, was part of a (inaudible 15:53:56) engineers in the country were without yet having enough information from the consultants' reports at all, actually Department of Building Housing expert panel has not been created basically as yet. We had sort of a closed door presentation to engineers in the country and when we were discussing about the Grand Chancellor, the major issues which were quite obvious from the beginning – irregularity and elevation – set back creating troubles to the wall inside that failed as well as to the columns just underneath were discussed in qualitative terms already at the time of May and April so in a way from a conceptual point of view it is important to know that moving forward it would be useful to agree at least that some vulnerability of these sort of buildings needs to be highlighted and identified and I say vulnerability and I am not referring as yet to the input motion which means the earthquake motion because when we talk about risk we need to typically consider the vulnerability of the building completely separated by the possibility of having a big earthquake or a small earthquake and a building is vulnerable regardless of the big earthquake or the small earthquake. Obviously the bigger the earthquake the more the risk of collapse is going to happen, but if we look at the Grand Chancellor it was clear from the beginning there's going to be clear – has been obviously more clarified and forwarded by the consultant's report, there were issue which in a way should not have been there for many reasons and obviously it's easier to say that later on it would be important to understand that if there are critical vulnerabilities we need to highlight them. Something which has not been mentioned but I think is important, it is just a parenthesis is that the Grand Chancellor was strengthened right away few days after by having the wall system and not only that being jacketed. Had that not happened right away, few aftershocks including actually the June aftershock could have caused a possibly a collapse of the building, which means that in a way the Grand Chancellor managed to shore redundancy up to a certain extent, but it was in a very precarious status. This means that in the understanding of the vulnerability of this type of building it has been mentioned the more than once and I like to reiterate it because it was really emphasised also by the expert panel a different type of earthquake maybe not as strong as February but a little bit longer than February, could have actually caused more troubles for the same building and this is what a discussion on ratcheting could have been taking place and even in the situation of the wall being designed according to a code, so even in the situation of the wall being designed probably according to the new code, a longer earthquake could have highlighted, exacerbated the vulnerability being such a regular type of configuration. That's something that is very, I think important that we are stressing up. This is the type of strengthening that has been taken place and in a way is a cracked ankle or a broken ankle of the building which likely enough did not cause the collapse of the building because of other legs of the building being place, and that was the redundancy which was a positive in a way feature of the design and the ankle being straightaway fixed, allowed the building to survive bigger earthquakes. I want to use photos live to just highlight the fact that the expert panel confirmed the, or supported the findings from the consultant's report and the two slides are really taken from the consultant's report but also from the expert panel report ensuring that the level 14 where basically we see how a building, the regularity is not yet in the cantilever, not only the cantilever but it is on the fact that there is a building that for many reasons was started with walls for seven storeys, let's say 14, but seven storeys, and then walls are becoming frames. Just to keep in the context in the wall we don't have yet any regulation about how to design a system in a simple manner which is made of a hybrid solution where the walls are becoming frames and for example United States are prompting maybe some suggestion from – Bill Holmes later on there is a lot of –

JUSTICE COOPER:

Q. Just pause for a moment please. You've explained that this slide that's now displayed isn't in the materials specific to your evidence –
A. It is, it is.
Q. But it is I think –
A. And it is, it is, very much I said, sorry. I'm showing two slides which are going to – they've basically summarised the confirmation of the expert panel. These slides is actually in the interim report of the expert panel.
Q. Yes, that's what I'm saying.
A. Yes that's what I'm saying the same.
Q. Just pause and let me finish.
A. Yes.
Q. It's not in the set of slides that you've handed up today but it is in the expert panel report at page 35 or by our code BUIVAR111117.37 which appears to me to be that plan?
A. Correct. Basically I –
Q. Although it doesn't have the heading 'Global Behaviour." All right?
A. Yes.
Q. Is that – am I right.
A. That's correct and I have added only from the version which you have printed, I had some issues with the computer so I added two slides which are these and the next one, these are the only modification for what you have in front of your hand.
Q. Well let's just find that other one now which is headed (overtalking 16:00:01) –

A. The other one comes, is not in the expert panel but comes from the consultant's report.
Q. Right.
A. And again Local Behaviour has been added but is one of the slides from Adam Thornton's presentation.
Q. Yes. So he's probably, in fact I recognise that from one we've seen earlier today where it was, I'll just get the number – 46. Is that the suffix is it?

MR ZARIFEH ADVISES THE COMMISSION SUFFIX IS 0003.46

Q. So that's – appendix B, page 12. Thank you.
A. So I would like to use two slides to show that's basically the fuller understanding of the performance of the building can be summarised by looking at first of all the global behaviour and secondly it look at mechanism which ended up being the failure and the two things are very important because from the global behaviour point of view to be honest the Grand Chancellor is a one-off so we don't see, we don't have in town anything it seemed after this, with such an irregular configuration walls becoming frames and such a recess or a set-back so we are not able to extrapolate this information to likely enough to other type of buildings which might have been designed with the same type of irregularity, but the importance is to try to really understand that a code does not explain to an engineer and does not really suggest that by step how the load path, so the way of distributing the forces from the – you know we say from the top to the foundation but it starts everything from the foundation and the way of bringing in stresses into the building is quite a complicated one and there's nothing typical we say as good or as bad as a devil advocates, like an earthquake, to find the worst, the load path scenario. For how obvious this looks like, this prompted the expert panel I think to start discussing something that also Commissioner Professor Fenwick mentioned in the past, the need to

discuss about redundancy so to do, have a redundancy explain, expressed and explicited which is something that typically is not happening, and one of the discussion in the expert panel was to suggest that a design feature report which is already a tool in place during a building consent could actually become mandatory in that it could not only be itself a mandatory way of presenting the project but also should always include a full explanation almost written by hand by engineer in what the load path, what the assumption are. By doing that and then introducing a peer review anticipating something which is can be a panel discussion, but introducing the information, a peer review process then it will be much higher for a peer review to capture problems, mistakes of issues if the fuller assumptions of the load path has been discussed, so the Grand Chancellor itself again luckily enough in a way does not have other brother from a structural point of view, similar to the same structure in town, nor does it for what we are aware of to the same extent in New Zealand, but there are issue with this recess, a set-back, we did find the set-back's problem and every single time that we had a set-back there were many, many clear issue of local (inaudible 16:04:10) mechanism. Something that we can note right away and that could have been immediately highlighted in the discussion of redundancy, the Grand Chancellor itself, if we look at the plan view, the original plan has mentioned very clearly, was to have these wall, D5-6 at grade E, plus a little bit of when you came back the possibility of adding a little bit of a fin if you wish and the –

Q. Is that what we – what Mr Thornton called a return wall?

A. A return wall exactly.

Q. Yes.

A. So if we look at just the quick calculation, north is on the west, sorry north is left here, east is up, so the building itself was actually quite regular, quite a square, and then suddenly from 29 metres in the east/west direction and if I remember properly 33 metres length in the north/south, the 29 metres drop, 24 metres drop to 19 if I remember properly, 24 to 19. So suddenly you're creating a weaker east/west direction itself and this is very important because unfortunately the February earthquake, and I have slides on that Sir, was very harsh or harder in the east/west direction and September was not, so how do we account that into design? Fundamental we need to know that in a design typically people do not consider, should but do not consider in an easy way how the combination of the loading occurs from the earthquake direction and is something that is not easy to account for so the future we need to have some further recommendation but just forgetting about what we know about the Grand Chancellor people could immediately see that for example in the east/west we basically have a big spine in the centre and this big spine doesn't give us a – not any more nowadays a good feeling. There is something on the north side, there's a wall on the north side but the big spine in the centre is calling for lack of redundancy in the east/west direction so there are example that I would be able to show of buildings which suffered almost a full collapse because they were relying upon only one wall in one direction. Is that legal in New Zealand? Absolutely yes. Is that legal around the world? Yes but is that the best way of doing it? Probably not and we can discuss about what would be another way of asking for a little bit of a redundancy. Because of the shorter distance in the east/west direction by moving the original design wall E5-6 into D5-6 basically the engineer found himself in having a 25 almost 30% of increase in axial load due to the lateral way in east/west direction so is quite a big number just because of normal let's say architectural modification. Second I think that vulnerability would appear so that in the east/west direction apart from the cantilever which is clearly a problem there is major, a major spine not nothing else so why in the south/north is a little more robust but walls of 400 millimetres for a height that is actually seven story, 14 story half way but is actually seven story today doesn't feel right so apart from the slenderness ratio it's unusual of total high three high, five metres and so forth of the first story divided by the thickness what is concerning is that nowadays engineers will not be very happy to design a 400 millimetre wall for seven story high and probably the discussion here was that it could be possible that the engineer was so thinking about the philosophy it's speculation was relying upon a lot this big I-shape wall in the centre thinking that the other walls would have just helped as balancing walls. Very important because if you want to go for a modification on the code we have to think about how can we in a way steer the design philosophy into the right direction more than into the more dangerous one. Now something else which obviously has been discussed if bi-directional loading and I have to admit that for how much it could have been in the code the fact that a column in the corner is meant to be taking the full compression in two direction so is also not a standard practice internationally to re-account fully for that so the code is something but then the standard practice is some that is, has to be rely upon which means that again possibly peer review a design feature which is going to have a tick off sort of a list like any pilot for an airplane would do without being embarrassed in doing it, I think would be quite an easy way of being sure so that the first the ABC fundamentally critically issue been addressed. A column of one metre by one metre sitting basically on a 400 millimetre wall doesn't feel right. There is something that an engineer today would prompt thinking the floor in between probably 12 and 14 level should have been so massively rigid to make the whole seven story below become a strong basement and probably that was the assumption to have a seven story strong, strong basement on the top of which building up a frame but as we understand it not work so there should be recommendation on that. For what I know on the United States, Will Holmes please correct me if I'm wrong, the way of dealing with such a irregular structure, the irregular being only the change of system from walls to frames is to reinforce to run (inaudible 4:10:18) so is quite a massive request for this sort of a building but from a risk-base point of view probably is the one that would be recommended. I don't think that (inaudible 4:10:30) would have spotted the problem, the problem could have been spotted in much easier way by having a design feature report and a peer review done. If we go to the local mechanism what we need to be really aware is that recommendation of the expert panel it is written clearly that we need more understanding and research on behaviour of walls in the bi-directional loading. The reason being is that is really complicated enough to deal with one directional loading and in the past decades the codes had mostly dealt with (inaudible 4:11:07) assumption of the earthquake and the earthquake is coming and we think it is going to hit in one direction independently from the other directions why the three dimensional behaviour is obviously more complicated. The reason for which the codes honestly are simply written in such a way is that laboratory testing, laboratory testing, laboratory facility only very recently are capable of testing under such a complicated testing protocol so willing to be honest and humble you understand that we know what we don't know particularly we said we don't know enough about what we don't know but in this case there is a clear need to investigate wall of any sort and their behaviour and real earthquake protocol which is scientifically is not available and it is prompting some more information. The discussion that Adam Thornton and the Commissioner Professor Fenwick had on this wall confirmed that to me we have a clear and unique sort of a step-by-step procedure to assess what the fatal mechanism would be in such a complicated way and what I can say though is that conservatism was in place in the 1984 code. That's a high level I'm going to go to the slides we have seen that Sir, the higher level of design acceleration for tall buildings which was buildings over 1.5 seconds had been dropped and we can argue why and why not I would like to think in an idealistic way that in 1984 people were concerned about a tall building and the complexity of that and then in the later on then the computer came over and helped people gaining the apparent confidence on the possibility of representing the behaviour of a structure and that conservatism has been dropped in the 90s. So the lessons learned would be that something else option B would be that maybe the 80s people were aware of the amplification effects at higher periods coming from soft soil condition. I don't know the answer maybe in the history of New Zealand we might, we might know where it comes from but the discussion is that basically the simple term is, "Had this building, this wall just one wall been designed with a little bit more conservatism which is not going for a 400 but go for a 700 for example millimetre completely dropping the higher axial load regardless of the calculation just because it felt right the 700 millimetre wall for seven story high, 700–800, then we may be discussing in different terms in terms of performance. Where are we going for how much we know what we don't know and would be a lot of research required to understand a three dimensional behaviour of walls. Simple, simple recommendation how to improve these detailing can save a lot of failure mechanism in the future. Very simple recommendation. Adam Thornton explained already and we agreed in the panel that obviously this doesn't make any sense but to be honest a little bit of a confinement such a confinement would ring the bell nowadays not in the 80s but right today it would ring a bell. We not be that happy to see a wall of that sort having such a small confinement as a gut feeling itself but even if that wall had confinement for all the compression area the analysis I'm sure that the thickness was the problem and the thickness was the problem because the axial load ratio as well as the slenderness ratio are a problem. Both of them axial load ratio is the axial load divided by the capacity of forming decompression of the wall itself and the slenderness ratio is the height divided by the thickness just by changing that parameter thickness is going to save the element and the lessons so that unfortunately is academic really but to share are from recent earthquake in overseas which did occur before New Zealand were in the eighties are the same type of approach happened quite well designed walls became slender and slender and slender and the earthquake in Chile 2010 had a dramatic failure of slender walls just because humankind decided to go to the limit and over and beyond what was controllable so it is reasonably simple to go back to a little bit of a different approach and just to give you some insight to what has happened overseas in the same time which is a part of the recommendation of the expert panel L'Aquila earthquake which is the Italian earthquake in 2009 –

Q. Sorry, can you just spell that for us for the benefit of the –

A. In English people pronounce la kweela but is laquila in Italian so it is L'Aquila, did happen in 2009, sorry 2009, correct Sir, that earthquake had a lot of similarities with New Zealand in that there was quite a high vertical acceleration and it the earthquake was not that strong but was underneath the city, there were approximately 300 victims, a lot of issues very similar to what unfortunately happen in New Zealand did happen but obviously when you are coming back from such a reconnaissance effort and you are trying to describe what overseas countries have learned the standard approach is it is not going to happen over here, so as an engineer our lessons that we have learned from February is we don't need to know in a way how many faults are underneath the city. We need to be able to have this vulnerability which is there regardless of the intensity of earthquake and minimise as much as possible and that is something we can do. If we design more robust buildings then they are going to be robust even in the case of not known or stronger than design type of earthquake and this is something that in the past has been done and obviously the construction boom changed a bit so one of the recommendation from the panel has been to obviously revise the way of designing walls and in changing not only the confinement in the compression area which is typically what we call the edge region which is the (inaudible 16:17:39) region but taking up the recommendation using the Italian earthquake, after the Italian earthquakes, recommendation coming after three major earthquakes in 10 years, so unfortunately Italy suffer three major earthquakes in three different areas in 10 years so a lot has been learned and overall, now the recommendation is that every single vertical bar will be hooked one to the other from each side of the building for the whole width of the building so what we call boundary elements are very well refined but then also in the core it will never be allowed wherever such a lap splicer and each single rebar are going to be hooked and the thickness going to be much larger. Now re-analysing the Grand Chancellor with this sort of a more recent implementation we could definitely I expect a lower probability of troubles, not zero but lower probability of trouble and the cost of that is going to be so negligible that it would be hard to understand why it could not be implemented. I will be trying here to really go through but to remind that the expert panel in deliberation did use information request and I prepared with research associate of mine, Dr (inaudible 16:18:59) a contextual report part of which is containing those famous graphs which I have been so much criticised and I am very happy that I have been because we were unhappy in having to prepare them. What does it mean? I would like to take opportunity to demystify completely what these graphs are about and just mention it, it is a qualitative approximately crude way of trying to mix different histories of design and different type of records on different soils and different type of response so we are –

Q. Could you just pause for a moment?

A. Yes.

Q. We, just so this makes sense in the future, I am just going to read into the record what our number is now that we have got this loaded on to our system so it is BUICAS 461.0048.14 shaking intensity 22 February 2011 stronger component east-west. Thank you.

A. So for much again it will appear rhetoric and also the interim report from the Royal Commission has very well highlighted these issues. Seismicity is going to be something to be honest we will never be able to address perfectly, not even close to perfectly, no way, and it is going to take probably centuries of trying to understand it but what we do know is that in a simplified way Professor Pompori of University of Canterbury mentioned for decades that earthquakes do not read the code so this beautiful mathematic model that we are using in any code around the world needs to be understood as being a mathematical model representing thousands of earthquakes possibly happening the same time, sorry, on the same site for thousands of years or tens of thousands of years, the average which might fit this mathematic curve. If engineers which means if the Royal Commission was able to remind people of that then we will never face the issue of designing for spot on the minimum code requirements. Engineers will never try to use a computer model, try to be on 2% just above the mathematic equation because by being below means that he will be basically are going to be illegal. It is such an obvious matter but from an engineer point of view we would know academic engineers or generally speaking engineers, we would need to know that the uncertainty of the motion is so big that we will never be able to control but good design and good detailing again looking at the past can actually make a building surviving and be more resilient. So what this is about is to try to show that in 84 for example there was quite a good and I honestly I don't know and we didn't go back to revamp why, why not, there was a quite good conservative approach for taller buildings. It could have just been that, the perception of above one second is 10 storey high, higher, and 10 storey higher and higher, it could have simply been they did it 10 storey and higher for a 1980 type of design was complicated enough but it could have possibly been that there was an understanding of the soil condition which has naturally, it has been shown around the world in different earthquakes amplifying in the longer period the demand on buildings so something like that for a match nowadays it would be prior to the February earthquake would have been fought by different commercially-driven obviously entities it would be quite nice to recognise if we don't know enough and either we raise the bar understanding the certainty or we require designers and owners to accept that we can't design for minimum standard because that is not acceptable anymore. So you have been asking very good question which is there was a fifth record over here. That fifth record was a station that we were very happy to be able to use in September. In February the same record station was not available because the data recorder were not reliable enough so GNS did not put over. Unfortunately I re-say unfortunately that was really the middle of quite a circle of records which as we have seen have quite a big dispersion so having something closer by to some buildings would have helped but unfortunately that record was not available.

JUSTICE COOPER:

Q. That would have been the site closest to this building wouldn't it?

A. Yes, not only to this.

Q. And the CTV probably?

A. Yes.

Q. Yes.

A. Exactly. So going to the main issue of the intensity and this is something that typically a code does not refer to, this is February recorded with a stronger component east-west, principal direction but February had a weaker but still strong component in the north-south direction. This is very important to know because the Grand Chancellor was weaker in the east-west. It is important to know that the February was stronger east-west but also important to know that north-south was not negligible, was still very strong. As soon as we go to February, sorry, September though and there was a very good comment in the interim report by Bill Holmes saying, "Could you please use the same scale" for how much have been using the same approach in any other occasions we had the troubles in trying to use the Boxing Day records at that time which was so tiny compared to the February and so we were adjusting scales all the time but if we look now at principal components September versus February. September that's the same scale now is north-south so in a way the Grand Chancellor could have taken because it was going through the big spine wall in the centre and we are very much aware that this is a little bit of a mathematical algorithm in a way to try to express what is possibly the response, repeat of what it is of an elastic responding system which was not the case or is not the case in most cases, we (inaudible 16:25:16) to the system is basically like a single degree (inaudible 16:25:19) system meaning there is a big mass concentrated at a big height. Such an assumption we know, and it was a discussion with the expert panel, can't we use the (inaudible 16:25:30) like with rigour for anything like the Grand Chancellor where there's not a clear mass concentrated in one point but basically there are two masses. These will be at what we will be calling a two degree of freedom system. Two degree of two masses located in the centre of this effective high from the frame on the top and one in the wall would have had it. In academia we are looking at creating spectra for something like that but it's not there yet so there are ways probably in the next 20 years' time generation of course is going to be helping people to make a simplified design assumption but at this stage they are not yet there but the most important thing are – September weaker generally speaking but, more importantly, we did run no leaner (inaudible 16:26:21) of some buildings in town and there's a reference that we printed and published before the February earthquake unfortunately saying, "Was that the big one?" and we used, for example, the Centennial building as an example to show that the earthquake in September was not providing the energy to go over the elastic behaviour and enter into the brittle behaviour. There was not enough, not only duration but really energy content in that type of earthquake. We did run a shaking table test on a building which was a typical pre 1970 building at University of Canterbury using the September earthquake and just to give you, and it is reported so it's in the public domain, a pre 1970 building not dissimilar to some of the buildings that we have been seeing unfortunately before me very poorly under the February earthquake, the September earthquake moving these shaking tables from instrumentation with a building on the top did not show cracking on the building at all. As soon as we changed the earthquake with a lower acceleration but a longer duration and we see later what an Alpine Fault event would be, lower acceleration again, longer duration and absolutely no bumps. There was no need to have these bumps. That building was prone to collapse and that's what we wrote saying that also are what we call a near field earthquake, an earthquake nearby, which we could not have a clue of being underneath the city but the GNS had reported that somewhere around the Christchurch area there could have been a magnitude 7 earthquake hidden somewhere so we did run some analysis on real buildings showing that an Alpine Fault (inaudible 16:28:07) down so weakened, longer duration, as well as a big pool velocity, what we call a velocity pool earthquake, could have collapsed buildings which did not collapse in September and this is what February actually is showing. February is close by, has a very high velocity, is really coming at what we call a 'velocity fling', it's a punch and this sort of a punch does not allow the structure to dissipate the energy during this way. It's well known, even in, not yet in code provisions, not yet explicitly but is well known in literature at least and now not 20 years ago so in the understanding of why February was so harder just to give you an idea the same shaking table test that the University of Canterbury could not run the February earthquake. The table was not able to reproduce that earthquake because for how much it was not so bigger in some area, the velocity was too high and this is something that's in the code we are not addressing, typically every code in the world is showing acceleration only so we need to address this issue of effect of something closer by which is an issue for Christchurch but is an issue for Wellington clearly.

JUSTICE COOPER:

Q. I might just ask you to explain that a little more. As I understand it, you're now drawing a distinction between acceleration and velocity. Is that right?

A. Absolutely.

Q. So could you explain that to me?

A. If we take (inaudible 16:29:39) a fallacy if we want or a myth, (inaudible 16:29:45) the understanding (inaudible 16:29:49) was to try to give inertia forces to a building by taking a mass and multiply by acceleration. It is the easiest way that an engineer can deal with earthquakes. Then clearly there was an understanding that displacement, a more or equally important issue that we have to account for but, to be honest, displacement are typically in the code so far in the whole world as check, not as a design of first parameter to deal with so they are coming too late and too late you have seen the discussion that has happened. The too late means that depending on the tool that you are using for explanation of displacement you can have quite a big variety. What has been shown since the 1994 earthquake in Northridge in California, and then in 1995 in Kobe in Japan, in 1999, so we're talking about a few years ago now, in Turkey, Istanbul, is that earthquake with approximate proximity to the epicentre being close by to the building for less than 10 kilometres, the Wellington type of seismicity, and now Christchurch but before Christchurch would have been far away from that, I call near-field, the type of acceleration, velocity and displacement that the ground is shaking close by to the epicentre was recorded in Darfield to be absolutely extremely high and so being closer by to the fault is causing what we call a 'velocity fling' which is basically, really we call it a big punch which doesn't give the time for the structure to go slowly back and forth and work as it was designed to do but it basically goes only one direction very quickly and fastly and this is something that a high speed sampling typically in laboratory we're not able to reproduce. There are few type of instrumentation that can reproduce that big shaking table and they have to be big enough to test a big building. We don't have them in the southern hemisphere. We have in Japan after the Kobe earthquake '95, the biggest shaking table in the world which is capable of shaking a full-scale six storey building but only Japan can do it which, by definition, means that before that happened and that was 2005 the know-how of human-kind in earthquake engineering is still so far behind of understanding how we face a big punch from an earthquake.

Q. So is it too simplistic to say that the velocity is the speed with which the earthquake arrives at the building or the city that's being affected by it from the epicentre of the earthquake?

A. It's not the speed at which it arrives, it's the speed of the shaking of the ground to be much faster close by to the epicentre than further away. As soon as it goes slightly further away the speed is dropping quite significantly. It's very similar unfortunately for Christchurch. This earthquake was very similar to the disaster that happened in L'Aquila which was something underneath, big vertical acceleration close by so big velocity. As soon as you go far away, not a big deal.

Q. I understand what you say about not being able to reproduce the high velocity earthquakes in the laboratory but having experienced the earthquake on the 22nd of February, presumably it's possible to calculate its velocity –

A. Yes

Q. – after the event?

A. Absolutely.

Q. Has that been done and reported on with respect to the 22nd of February earthquake?

A. We look at that, we didn't prepare it because it could (inaudible 16:33:45) help and the numbers were quite high and substantially high compared with what has occurred overseas and just to give you an example, I don't have the numbers, I don't recall but I give you a range. A record which can come from the Alpine Fault would have a speed of the ground shaking of approximately, let's say, 40-50 centimetre per second. A near field, something a velocity pool can go to 1.5 metre per second. There have been records of 22.5. The record of February, and even June was a very very high speed ones because it was very close by to the city could have give this sort of effect. We do have in the last

10 years enough information in literature to suggest designer how to account for that and typically the way of accounting for it is to not rely upon the capacity of the structure to move or dissipate as it goes to use these plastic hinges back and forth but to diminish that capacity of dropping the damping typically that we are using in the formation of the plastic hinges so there are conservative ways of introducing immediately in the code requirements which, for example, are obviously very useful for Wellington of not accounting as much as we typically do for the ductile behaviour which is reducing the forces – we think is reducing the forces to much higher extended actually does not happen, and this has been happening for the Grand Chancellor very properly, but also for other type of buildings that we have been looking at as high, so the two graphs are hiding completely further information which is the speed at which so the damping and the two structure could have used to reduce the elastic responding behaviour.

Q. I've had an idea about attenuation with distance from source.

A. Yes.

Q. Like I suppose I've been thinking of the thing being attenuated as being the accelerations but you're really now saying what's attenuated is the velocity or are you saying it's both?

A. Is both, and the region where we typically created is attenuation relationship, is typically far away from dare I say the explosion because that area is what we call, an engineer in terms is a sort of a complicated enough to – is a disturbed area we will say, is a very complicated disturbed area so we typically tend to go a little bit further away. From thereon we can have a nice mathematical expression, but there are now many records in the world are showing that close by to the epicentre you can have really significant troubles and now Christchurch has become for that, not only Wellington in a way and this can be dealt with, is not really a major issue.

Q. So does the rate of attenuation affect acceleration and velocity differently or is there a clear relationship between the attenuation of both phenomena?
A. What I will say the most important part is that once the velocity – the velocity is a characteristic course of the type of rupture of the earthquake, let's be very honest. Not all of them are going to provide the type of earthquake, we call – there is a forward directivity type, anyway, it's a forward mechanism that can trigger it or not but once you are in the very close area the most important thing is to know that if we get the records, let me go back and we are going to do that, we haven't done it but if we are looking at this event and we were plotting –
Q. This is the 22nd of February?
A. 22nd February earthquake, you see the peak ground acceleration here would have a very high peak ground velocity close by dropping quite significantly but still being a very high level in the Christchurch city, in the CBD because it's basically within five to 10 kilometres and dropping consistently, significantly far away.
Q. So there'd be a similar pattern if you were able to reproduce on this diagram the velocities, they would reduce in a similar way to the accelerations as one moved further west from the source?
A. Yes. What we have been – and that's the international community have been observing, is that if we could we would be dropping not only almost completely but dropping the too high a roll given to peak ground acceleration alone, as the main design parameter because we could provide examples of very high peak ground acceleration records, September, which are not really strong enough as a shaking earthquake, or lower peak ground acceleration records which are providing higher velocity and / or higher displacement and the structure will be subjected to more damage than – all the way round. This is something that requires a sort of a merged effort internationally, things are happening and it's just a matter of – it's going to be a matter of in a way slowly changing again the – simply, demystify if you wish a reduced

the roll of this mathematical equation, remembering where it comes from and what are the assumption behind, just reminding to people what's behind these graphs. There's going to be sufficient enough for an engineer to take quite a good judgement on it and that is something that is when the expert panel is discussing, seismicit once more. I don't want to go there but the secondary component is showing the same train, September versus February, the secondary component in February for how weaker is when compared to the principle and its north now instead of east west, is still a strong one and this makes quite a difference between again February and September, the single two components of plastic velocity in the Grand Chancellor going back to our objective, had been definitely asking the Grand Chancellor to do something for which would have been hard to design in the 1980s and even probably today. Something that is coming up from the Grand Chancellor, primarily but not only is the issue of vertical acceleration and the issue again is something where we have to be honest. There is a big grey area in the international community for how much we are able to plot what a vertical acceleration spectra looks like. The actual effects of vertical acceleration are very far away to be well known. What we do understand or at least what I've been trying to explain even to myself, is what I fail to being myself a structure, and there is a perception that vertical acceleration has a higher frequency which is true. You can see here that some of the records are really coming out with a very high frequency on the top of a (inaudible 16:41:23). At the same time if you look at these two records the frequency is not as high as we think and these is the ground vertical acceleration. My example of how we need to think at, sorry, effects of vertical acceleration is prompted by the fact that everyone of us may have felt one of the aftershock for example in February, which clearly pushed us up, and it was not an acceleration, but it was the movement, the displacement and the velocity of the ground coming to us. I did feel the ground coming to me for a substantial amount, which is not an acceleration, is a sustain acceleration which gives a displacement or a velocity, a displacement for what were the velocity rate. Now in the code, because we don't know enough about velocity displacement, certainly we don't know enough about velocity and displacement in the vertical component, but example that I'm providing is that the type of feeling that probably Grand Chancellor had was two type of things. The cantilever was probably feeling the inertia so the building itself was structural which was more likely like ours with an extension being the cantilever, so the cantilever probably force hammering against the wall could be explained using one of the spectra, but the main core could have been subjected to this big push up movement of the ground which basically is compressing the legs of the building and my example is typically thinking, sorry for the simplicism of that, but I found it very easily to explain. If we think about going down the slope and getting a bump, that bump is the same way of the ground to move for a substantial displacement at the high speed. We going through a bump, we are going to right away feel our leg being compressed before the waves are able to move up our body and push us up so the inertia is not happening instanteously, we are not able to use a vertical acceleration to explain why our legs are burning if we go to a bump, but we do feel the compression, so the explanation that we have to go through and that's what research is required for, is to move more into the ground vertical motion as a displacement in velocity to understand that first the column has been squashed and then the waves have been propagating up in the building, and this could have been given a humungous amount of axial load in the columns of the Grand Chancellor. Do we have a theory behind, we don't and this is something that always we require quite a lot of information and what we can state, in a way that there is an international interest in understanding and learning from the Christchurch earthquake because a lot of people are finding lessons from what we could call the biggest open air laboratory ever in such a short space. There are information and data that are going to require the massive effort and I saw in the Commission report not only the DBH recommendation about doing more, the question from people could be what if, what if and what can we do in the interim, we will not have an answer in six months, what can we do in the interim and in the interim is using the good old conservative way of designing which probably is going to give us some robustness in many situation and again, having just the thickness of the wall of the Grand Chancellor been bigger the vertical acceleration effects would not have caused, if they had would not have caused excessive troubles. They would have been limited. The other technical suggestion from the expert panel that Adam Thornton referred to is that all of it, all of this can be in a way summarised in trying to put a cap understanding uncertainty of many parameters and the uncertainty of the earthquake and the uncertainty of the response of, three dimensional response to the building putting a cap of the maximum allowed axial load on a structure element being a wall of a column can itself be a starting point. Not allowing all the time to go to the limits it can be a starting point. So on that I would be able to stop over here what I had here I can flick through is simply example of other building in town which have been recognised to be in the same category because we have been doing a lot of work behind the scene to come out with damage correlated to reinforced concrete building and one of the unfortunate if you look at the for example the slide summary is that regardless of the age, 50, 60, 70, 80, 90 approximately 50 or more percent of reinforced concrete multi-storey building were either yellow card or red card. It’s not a great news. If we go to wall and I am going there because it is like the Grand Chancellor concrete walls defined by or distinguished by age, pre-70, 80, 90, and (inaudible 16:46:47) stay here in the 1890s the number of yellow tag which means quite a good damage and the red tag which means possibly demolition is definitely higher than what people are expecting and we have been looking at those buildings to understand what type of mechanism they went through. I will be happy to basically stop over here by saying that by looking at many buildings we found similarities in the sense that many of them were failing in a brittle shear compression and I am highlighting that because Commissioner Professor Fenwick mentioned that things are really complicated. We are talking about three dimensional behaviour. We are talking about high compression interfering with shear not only compression alone which would be self complicated enough and we do not have yet the theory to, or experimental evidences to look at that. At mostly though a lot of these buildings fail because the demand was very high but because the redundancy or the confining detailing again not very expensive were not good enough and I show slides over here to show that this is something did happen in a 1980 type of building, huge compression happening can be avoided. This is something a buckling compression, huge compression failure in the edge of a wall which could be designed for against. This is something that we have found in other type of building which again in a way is not a major deal, can be dealt with, and if you go to 1990 building did behave quite nicely. I don't have it, I don't want to show it here but I have quite a nice video that would be interesting tomorrow if you are interested to show what is behind such a wall. Such a wall in a post-earthquake reconnaissance would show very minor cracks and that is something that has been mentioned over and over. What we see is a residual crack is only the tip of the iceberg when compared to the real crack happening during the earthquake and this is very much true for element like a wall which has advantage of a bi-axial load, big level of axial load all the time closing back the cracks. This means that when we are approaching wall assessment post-earthquake and it did happen obviously in a few buildings that we are aware of there could be a, there should be a higher concern about what we really see and what has happened during the mechanism. I think I am finishing with this slide which is very similar to the Grand Chancellor or east-west direction being one big wall carrying the whole burden of taking lateral resistance. This building did not have redundancy enough. That wall did not behave again for shear compression in a failed very poorly or inadequately and that could have caused the collapse of the building itself and because there was not other type of mechanism so –

Q. Is that building in Park Terrace?

A. Yes. Yes. It is, and the wall issue of lapping horizontal reinforcement we should not have been dealt with as well as L-shape, there was some L-shape walls in the Grand Chancellor. In a way they were not activated because there was a weakest link happening earlier but they did not like the combination of shear and high compression although they were actually designed with a very, very well detailed confinement so a lot needs to be dealt with and I am finishing off here. There are evidences from other earthquakes which I will be happy to show tomorrow as well as some discussion about what we would be able to do in the near future or what we need to do to do some more testing or experimental research on the real protocol that this portion of buildings are subjected for which are tried at three dimensional, they are not two dimensional. Thanks for your time.

Q. Are you able to come back tomorrow are you?

A. Yes.

Q. I think we will do we will hold any questions that we have for you until the panel session tomorrow which will be after we have heard from Mr Holmes. Is that all right?

A. Thank you.

WITNESS STOOD DOWN

COMMISSION ADJOURNS: 4.52 PM