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: S Mills QC, M Zarifeh and M Elliott as Counsel Assisting

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NEW BUILDINGS TECHNOLOGIES HEARING

COMMENCING ON 12 MARCH 2012 AT CHRISTCHURCH

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JUSTICE COOPER:

Good morning and welcome to this week's hearings before the Royal Commission which have as their subject matter New Building Technologies and we hope that this will be a chance to look positively into the future.

MR MILLS OPENS

Thank you, Sir. May it please the Commission I appear with Mr Zarifeh and Mr Elliott as counsel assisting.

Well, as Your Honour observed, this hearing does have a distinctly more optimistic tone than a number of the matters that the Commission has had to look at in the past. There are no deaths or injuries, no inquisition of failed buildings or failed systems.

The purpose of this hearing which is set down for three days, but with ability to accommodate a fourth day if that is required, has three principal objectives. The first one as the schedule for the hearings programmes says is to look at the history of seismic design and put future trends it will be looking at in that context of the wider structural engineering design philosophy that has largely underpinned modern structural engineering. Of course as the Commissioners are well aware, this is a concept of controlled damage that in large rare earthquakes aims to preserve lives but fully anticipates that there is likely to be irreparable damage to the buildings themselves. That goal, of course, has been largely achieved in Christchurch, at least in respect of modern buildings, but the resulting level of damage has sharpened a debate which will be dealt with by the first speaker in particular about whether that is an acceptable outcome even for rare events.

The new technologies that will be dealt with in this hearing all aspire to outcomes that will not only preserve life but will enable buildings to be re-occupied and economic activity resumed more quickly after even the most serious of earthquakes. So that broader question of design philosophy is dealt with by two people – the first speaker in particular who is Dr Rajesh Dhakel and also by Professor Nigel Priestley.

The second purpose, obviously enough, is to hear from a number of experts all of whom are working with a range of innovative technologies and they are giving evidence in relation to both different types of materials and also different possible solutions. The first speaker we are going to hear from on that aspect of it will be on base isolation techniques. The second will talk about a range of innovative measures working with reinforced concrete. The next category relates to steel and finally to timber.

Now all of these have a common objective of building structures that have less damage to them and an ability to be reusable more quickly. Some of them, as the Commission will hear, may also have the additional benefit that following an earthquake event whether there has been any critical structural weakness to the building may be more readily able to be identified than with some of the more conventional systems that the Commission has already heard a great deal about in earlier hearings.

The third purpose of this hearing is to look at some aspects of the regulatory environment within which innovation occurs and through which it needs to pass if these new technologies are to enter the main stream. So those at least as I had envisaged this are the three principal objectives of this hearing.

Just a word then about hearing structure. The way this has been arranged is that there will be a principal speaker on each of the identified topics and that will be followed by a commentator, frequently a structural engineer in active practice who will bring to bear, particularly where the principal papers are by academics, their experience of working with these issues in practice.

At the conclusion of each discrete area of the hearings there will be a panel discussion. The purpose of that is to allow the speakers to talk to each other about issues that they may have about each other's contributions but also of course to enable ready engagement with the Commissioners. Generally the members of those panels will be people who have given papers or have been commentators during the course of the day but in one or two cases we've added in some new people specifically for the purpose of those panels.

Just a couple of miscellaneous matters then to touch on. First of all the full CVs for each of the speakers and commentators have been loaded onto the Commission's website so for the purposes of swearing them in I will make only brief reference to what are almost, well in every case, very extensive and impressive curriculum vitae. Secondly, in most cases, the speakers intend to speak from Power Point.

That is all I have got to say by way of background to this, and so I'll now call the first of the speakers and this is Dr Rajesh Dhakel.

MR MILLS CALLS

RAJESH DHAKEL (AFFIRMED)

Q. Just before you start your presentation I will just deal with a few formalities, and first of all can I just get you to state your full name for the record?
A. Rajesh Prasad Dhakal.
Q. You have a PhD in Civil Engineering from the University of Tokyo?
A. Yes I do.
Q. And you are an Associate Professor of Structural and Earthquake Engineering at the University of Canterbury?
A. Yes.
Q. You're a Chartered Professional Engineer?
A. Yes.
Q. You've been extensively involved in evaluating buildings after the recent Canterbury earthquakes?
A. Yes.
Q. For that you received the 2011 Institute of Professional Engineers of New Zealand President's Award?
A. Yes.
Q. You have had extensive involvement in earthquake engineering research in New Zealand?
A. Yes.

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Q. And for that you received the prestigious Earthquake Commission New Zealand Society of Earthquake Engineers Ivan Skinner Award?
A. Yes.
Q. You're a member of the Management Committee of the New Zealand Society for Earthquake Engineering?
A. Yes.

Q. And your topic today is up there on the screen so I don't need to take that any further. So I'll just sit down then doctor and just let you take the Commission through the issues you want to deal with.

A. Thank you very much. Honourable Commissioners, my professional colleagues, ladies and gentlemen. First of all I'd like to express my sense of gratitude for the privilege of being asked to present in this esteemed forum. What I have to say about what the existing state of seismic design of structures in New Zealand has performed and where, in my opinion, the design philosophy and the focus of the future researchers should go to in future. So hence I have titled my presentation as Performance Based Seismic Design: where to from here?

In order to make my point clear I'm going to refer to some of the observations that we made in the recent Canterbury earthquakes because that is vital in validating what I mean by the performance of current seismic design philosophy and being conscious of the fact that there will be other professional colleagues talking about the detail of new building technologies in the next two or three days I have consciously made an effort to put my reference to those technologies as short as I can.

So, first of all to summarise in my opinion, what did we observe in the recent earthquakes? We did observe severe damage to non-structural components (by non-structural components I include ceilings, parapets, partitions, façades, windows, chimneys, canopies et cetera) than to structures. In buildings that were moderately or minorly damaged the non-structural components in many cases were observed to be severely damaged. When a building completely falls then we cannot distinguish what was the extent of damage to non-structural component before the building collapsed but to those buildings which did suffer some form of damage then we can go inside and have a look at non-structural components and when we did that the observation that I got in general was non-structural components were more severely damaged than the structural components.

Modern buildings, they suffered damage.

Older buildings, severely damaged with some collapse.

And I hope all of you agree with me that ground response was very poor in many parts of the region, the Canterbury region and that included hazards in the form of liquefaction and lateral spreading.

And many buildings were rendered un-useable due to ground or foundation damage, although as a structural entity they didn't perform that badly but because of the foundation or ground damage underneath they couldn't be re-used or re-occupied.

So did we expect this? Before we ask that question we must put what we were exposed to into account. So we have to remember the imposed demand was similar to, especially in September earthquake, and higher than, in February earthquake, the current design level demand. And this is a very general statement because the demand depends on what type of structures are we talking about. Because the structures are identified with their natural period of vibration and for each different structure with different periods of vibration the demand will be different but in general that's the reception that we got.

So when we take that into account then damage to modern buildings, was that expected? And definitely yes, because we designed the demand, designed those modern buildings for a lower demand than they were exposed to and the modern design, even if the demand or the exposed seismic force is less than our design demand we expect damage. So that was not a surprise.

Severe damage and some collapse to older buildings. Did we expect this? Yes. In fact professional engineers have been telling the last few years that, okay, our old buildings which were designed for a very low level of seismic demand were very vulnerable if an earthquake of a reasonable magnitude was to hit the region and perhaps we were very lucky that more didn't collapse and one of the reasons that the engineers, including me, have been giving is because of the shorter duration of the ground motions induced. Looking at several old buildings in the town after the earthquakes they were there standing but very fragmented states, very much deteriorated states and it wouldn't take a scientist to predict that had the ground motions lasted for another 10 seconds they would have completely fallen down into rubble.

Damage to Non-structural Components. My answer is, we never thought about it. It is only very recently that we have started giving due attention to this very important component of our building systems. So we never designed our non-structural component except for very modern building systems for better performance from its non-structural components.

And very poor ground response. I'm a structural engineer. That's why I'm not in a position to comment on it because I don't have enough background information but my question is, did we expect this? Did we know this? If yes, why did we let buildings be built on those. That's the question probably all of our non-geotechnical engineering people might be asking.

So what are the things we need to do differently from here on? Local soil characteristics to be checked thoroughly before buildings are planned to be built in an area. That's probably the question coming from a completely different background 10 years ago, from a developing country and comparing it simply because in a developed country like New Zealand buildings being built even within the last decade in, in the kind of areas where the soil was so soft and so vulnerable, that was, forced me to ask a question to myself, is the local regulation authorities, did they do the verification of the ability of the soil to take on those buildings. Really was that process robust enough.

JUSTICE COOPER:

Q. Dr Dhakel, I'm sorry to interrupt you. I think you're being prompted by these slides on the screen but one has been missed out. I'm not sure if you recall that and I suspect you may not because you're going off the screen. Can we just for a moment go to one in this same series which has the suffix 1.4 and the materials we have is another slide and I just want to know whether you have consciously left out –

A. I can see the reason because there is one slide, probably it was before this.

Q. Yes.

A. It will come two, three slides later.

Q. Okay. You've altered the order?

A. Yes.

Q. That's fine, that's fine.

A. That's the only change that has been made.

Q. Okay well that's a matter for you. I was just checking.

A. Yeah, I felt that probably that order would make my message clearer than what it was earlier.

Q. That's fine.

PROFESSOR DHAKEL CONTINUES:

A. So, so the non-structural components need to be systematically designed for better seismic resistance from here on. That's the thing that we must do differently.

Stricter regulations are to be put on retrofit of older buildings if we do not want to see the catastrophe being repeated or the damage, the collapse being repeated, probably not in Christchurch because not many buildings of that type are, are remaining, but in other cities. And we have to communicate better. That's one thing which occurred in the mind of many technical experts involved in the process, that we have to communicate better among ourselves because the communication between architects, structural engineers and geotechnical engineers – probably a better communication between them could have helped them understand each other's expectations and limitations in a better way because there might be several cases where the architects would want to have a specific configuration of the building but in structural point of build that will demand a more robust analysis when you design those buildings.

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So of course there are ways out to meet each other's expectation but we need to be aware of those. So we have to communicate better and the most important thing is with the public. Because what we observe is actually the public sentiment was that when we say a building has been designed for earthquake resistance they understood that it, the buildings were earthquake proof which is indeed not the case. We cannot get an earthquake proof building.

Now the other thing is we have to adjust design target according to expectation. If we expect our buildings to remain intact after a big shaking like what we had in February then we have to design accordingly. We cannot sow berry seeds and expects cherries to grown out of it.

Now so in order to do these things differently what are the research needs to enable us to do these things differently then. In my opinion we have to do work, more work which is already ongoing and we have already achieved a lot in these aspects in the development and advancement of low damage building technologies and these technologies do relate to reinforced concrete, steel and timber buildings, all of these kinds, and we have to also work which is probably is a little bit of infant stage at this stage in time, we have to work more on damage resistant design for non-structural components, especially partitions, ceilings, façades, parapets, chimneys, stuff like that.

And if we really want to minimise the loss or if we want to see the loss being minimised then our designs should directly aim to achieve that so loss minimise seismic design approach with means to control the loss, not only to structural damage but also due to contents, downtime and injuries that need to be worked on in future. And we need to have reliable ground assessment methods to see whether a ground in particular is up for it if we want to build a new building on it, and if the ground is found to be weak we need to have mitigation strategies to improve the ground performance in a likely earthquake.

So at this point I would like to change the topic slightly and look at Performance versus Loss. It is an enigma to me because these two are different, have different meanings, but so far we are usually referring to them as I said and which is exactly has been brought to forefront from the observations from recent earthquakes.

So if we are complaining about the damage that we have got. I mean by we, it is the engineers, the public, the authorities, if we are complaining about the damage of these, of the buildings and structures that we have got in the recent earthquake then basically dare I say that the modern design expects or attracts them as not in that level of earthquake, in a lot lower level of ground shaking than that. In many cases when we design ductile buildings we use force-reduction factor to reduce or the ductility factor to reduce the elastic demand on structures and that comes very close down to in many cases serviceability limits they demand. So even in a very moderate shaking we expect the structures to go into inelastic phase. So the ground shakings generated by these earthquakes were not smaller in any way, not than, what we design for. So obviously because that is expected so that we don't have anything to complain about because what we have if we want to complain we have to rightly complain about the performance criteria that we've put in our, in our design. So are we complaining about loss? And if so let's look at what my take on that is.

The current design cares about life-safety but not any other forms of loss, not monetary, economic consequence. So if loss, if that hurts or if that matters of course it does to the economy to the owners, to different stakeholders of the property then the design process should explicitly aim to control loss. That's what I, I think we should do. So let's look at what's the objective of seismic in general throughout the world in all seismic design philosophies or seismic design codes that have been, have been used in the world.

In general in summary I can dissect the performance requirement down to three categories, discrete categories: the immediate occupancy which loosely says no disruption of service after small and frequently occurring earthquake. Of course that is a subjective term and the quantification of that actually differs within the realms in different design codes. So the meaning is if a frequently occurring earthquake occurs we don't want our building to be closed so of course if some minor damage occurs the building doesn't need to be closed.

The second in my term when I summarise is probably reusability. The damage is moderate to strong earthquakes. Again that is a subjective term, different design codes, quantify that and in different measures and the whole idea is for the building to be reusable after repair. So damage is expected but to a controlled extent.

And the final one is life-safety or the collapse prevention even large and rare earthquakes so the damage can be irreparable but we have to make sure that the building doesn't collapse and kill people. So the building may be, may need to be demolished after, after that level of ground shaking. so that's the three very broad criteria that have been used and not all design codes use those three criteria. Mind you in New

Zealand we do not three criteria we use only two. The first one corresponds to the first criteria shown there and the second one is thought to cover for the next two. We do not explicitly require the buildings to be checked for collapse prevention in the maximum critical event, so but in general there are different forms lying there in different design codes.

So how has the ongoing ductility based capacity design performed? The kind of design that we have been using so far, how have they performed? Immediate occupancy in minor earthquakes, we have achieved it. But minor damage needing repairs without disruption is required and it does not violate the code requirement because the code doesn't say that it doesn't need to be damaged. It just says that it needs to be immediately occupiable.

Reusability in moderate to strong earthquake, yeah, we achieved that using the modern capacity design principle but moderate to severe damage will result which again doesn't violate the design criteria and in some cases minor injury and disruption of service which is of course what we call as downtime. In order to repair we have to close the building for a while.

And the third one life-safety. In very strong earthquakes with big confidence I can say with modern capacity design if it is implemented in its truest form it will achieve the life-safety and in, not only in Canterbury earthquakes in many earthquakes world-wide if we look at the report and if we look at the observation then what we have found is most of the casualties if not all had resulted from the buildings, the failure of the parts or whole of buildings from, which were designed or built long time ago, at least a couple of decades ago and that's when our buildings were not as robust or built, design philosophy was not as robust as it is now. So in general capacity design apart from some exceptions if implemented properly it achieves the life-safety criteria.

Where do you stand now then? We achieved all design objectives we wanted from our seismic design method, yet the financial loss due to the three Ds – damage, downtime and death – could not be avoided. The total loss in some recent earthquakes was reported to be in 10s of billions. In 1989 Loma Prieta it was $11 billion, 1994 Northridge $17 to $26 billion depending on which report we look at, Darfield earthquake more than $NZ6 billion, and Christchurch earthquake more than $NZ20 billion.

Is this what we wanted? And my answer is, "No". So where have we gone wrong? We have gone wrong in setting the performance objectives of our structures in the design philosophy. So in my opinion where to in future then?

In future we have to move loss optimisation seismic design. What is it? It is the design criteria, a design principle which has two performance objectives. The first one is life-safety which cannot be compromised at any cost we want to retain that. The monetary factor is secondary. And after that we do not care about immediate occupancy. We do not care about reusability. What we care about is minimisation of earthquake induced loss.

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If that is minimised automatically the other criteria that we have been using in our current design will be automatically taken care of. So the design criteria for this future oriented loss optimisers in seismic design, this will simply be expected loss is less than acceptable loss. Who decides the acceptable loss? Probably currently the client. The client tells us up to what level of loss and what level of ground motion can be tolerated. Can he or she accept what in future, the 1170.5 20 years down the track might have a table saying that, okay, for residential buildings this is the tolerable loss level. For commercial building this is the tolerable loss level. In a frequently occurring earthquake, in a design basis earthquake and in a maximum credible earthquake and, similarly, for emergency facilities these are the losses which can be accepted down the track but what we need in order to apply this is we need a methodology that can help us calculate or estimate the expected loss and then we compare against the acceptable loss specified by the client or in the code, right?

So the design philosophy or the design chart might not look any different from what we have been using now, the current performance based design philosophy has a chart like that or serviceability limit state and for design basis earthquake, DBE, it has got different building or importance categories and different performance level. It says immediate occupancy, re-usability or collapse prevention and life safety, all those things are presented in a way, and in the same format we can have in future a design chart like this which says on the column side, “Frequently occurring earthquake,” “Design basis earthquake,” and “Maximum considered earthquake,” for these different levels of demand. Now if we design our building or divide our building based on the importance category in three or four discrete bands, residential buildings, commercial and office buildings, emergency facilities and people, if you people want we can have one or two more categories there but more research is needed. These are just to sell the concept. And then we say, okay, from the three DDDs – damage which will incur loss in the form of repair cost or replacement cost; downtime which will incur loss in the form of loss of revenue when the building is closed and that is more of an issue in commercial and office buildings rather than in residential buildings; and injury which can have three vectors, minor injury, sorry, three components, minor injury, major injury and fatality or probability of being, people being dead. So obviously for residential buildings when a very big earthquake with very large return period which has very low probability of occurrence occurs then we can accept the building being, needing to be demolished, the building being severely damaged. We can accept the downtime being infinite because the building can, needs to be replaced, as long as the fatality probability is very small. Whereas when we go to our emergency facilities like hospitals, fire stations, police stations, nuclear power plant and the kind of facilities that need to be operational immediately after these significant events, in fact their importance gets even higher after those events, then for them money is a secondary issue. What we want to do is actually make sure that they are designed for a very low probability of damage, probability of downtime and very low probability of injury including death.

So the loss assessment process in general. There has been research ongoing and we know, or we have, we have known for the last decade or so, the (inaudible 10:34:44) have been ongoing and based on that theory what we can get is actually, we can estimate the likely loss from a given event which has got a given annual probability. So if the annual probability is small we are talking about a very, the probability is small we are talking about an earthquake which has a long return period, it means a very severe earthquake resulting in very intense ground motion. So of course when we apply these methodologies we will get a very high likely loss. Similarly, if we talk about a ground motion which has got higher annual probability or a smaller ground motion, then the loss will be less and we get several points like that. When we connect that point we will get a curve like this and the area underneath that curve which is called the “Expected annual loss,” takes into account the effect of all likelihood seismic scenarios ranging from very small annual probability or very large return period to very high annual probability or very long, very long return period. Okay, very short return period to very long return period and that expected annual loss, if we take into account only the direct repair cost, that in fact is direct indicate of the insurance premium that we pay. Of course it does not include the overhead cost and the profit of the insurance companies but that is an indication of the risk involved in repairing the damage, not including the downtime, and death. But the same framework can cover, can be used to cover all three DDDs. We can use the same format to cover for downtime and to cover for injury as well.

So if we look at that particular curve, annual probability and total risk ratio to cover the repair cost due to damage and if we divide that curve into different bands and please note that the horizontal axis in logarithmic scale. So if we take the first band or up to 0.01 annual probability or in rude sense 100 year return period and the second band, from 100 year return period to 1000 year return period. If you integrate that to get the area, of course, although it looks very small because that horizontal axis is in fact 10 times larger than the previous one because it is in logarithmic scale. So out of the calculated 0.25 percent of expected annual loss which is the total risk involved, half of that, .18 out of 0.35, comes from those very smaller earthquake, very small earthquakes which have annual probability of less than, sorry greater than 0.01, which means which are likely to occur once in every 10–100 years, smaller earthquakes and of course which cause smaller damage. But that covers for half of the total risk and smartly the insurance company they pass that onto the building owners as the insurance deductible, what we call excess. For each car insurance, for each house insurance the first few hundred dollars we pay for ourself and as that damage is likely to occur more frequently than the bigger damage the insurance companies are passing that risk which caters for a significant proportion of the total risk to the building owners.

So if we apply that process then what happens? If not only helps us to apply loss optimisation seismic design as a principle but it helps us to make informed decision in allocating resources. For example, if we apply that methodology with some assumptions of course to a 10-storey reinforced concrete office building with all the, you know, like, all the distribution of structural, non-structural components and the contents and put their costs and likely repair costs into the system and do the analysis in general ,then what we, what kind of information do we get, is an information like that. So for a given ground motion the total loss from the building that comprises of these contributions. Like we can see the acoustic ceiling contributing to 15, 14 percent of the loss, the partitions contributing to 20 percent of the loss, the slab frame connection contribution to 11 percent, beam columns contributing to 14 percent, computers and servers contributing to 19 percent. So these things which is, which probably gives a lot more information, like maybe we have some amount of money to be put into retrofitting or to put into improving the seismic performance in terms of reducing the loss then where we should be putting that money?

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In the beam column joints or in the ceilings or in the partition or in securing the computers and servers because that costs us 90 percent of the total loss. It gives us a lot more information. Not only that now. Let's look at the contribution of different storeys, different floors in the total loss in that 10 storey building. What it tells us is actually in the third storey and sixth storey they are contributing a lot more than other storeys in the total loss and why is that so.

Next slide – So when we look at that the third and sixth storey they are contributing more and I immediately know why because the servers were located only on the third and sixth storey. So that servers loss that is contributing to the third and sixth storey. So again if we have limited resources where should I be putting into is probably to do something on the third and sixth storey because the return of that investment will be more than if I put that same resources in other storeys. So that is informed decision making.

Now let's talk about efficient decision making. I have got several options left – two, three different options to retrofit an existing building.

One of the option of course is not to retrofit which doesn't cost me any initial investment. Initial cost is zero. If I retrofit a building using one of the technologies then my initial cost, let's say is $40,000, but if I do not retrofit the building the proponents will be deficient if I do the loss assessment analysis I'll get the expected annual loss will be large and if I take that expected annual loss over a long run then the line, accumulated loss over the lifetime should be a straight line if we do not take into account the rate of discount or rate like the present value of the money. The $10m a few years down the track will be less money right now because of the interest rate and those things. So if you take that into account you will get a curve like that. So if I retrofit the building using $40,000 scheme that will improve the performance. The expected annual loss will come down and the accumulated loss will go like that. So at that point in time we will break even the cost. If my building has an expected lifetime of more than that then it is certainly a wise investment, otherwise it is not. Then if it is not, what we do, we go to the second option which might cost more but we improve the proponents even more, and the curve might start from here but it will go flatter like that and the cut off time is earlier than what we have earlier so that will help us making efficient decisions using different options we have. The same thing can be talked about new building design. We have the existing building design technology. We have new building technology using brace system, using base isolation. Of course they might cost slightly more and that will reflect in the initial cost. The initial cost will be higher but definitely over the time that cost will be offsetted within a few years down the track so that's definitely a smart investment. Those sort of decision making will be helped by this seismic loss assessment method.

So if we look at the performance of our structural systems in general then what happens is the probable loss. If we plot that in terms of annual probability the damage starts kicking off a lot earlier. Even for very small ground motions we get some damage because our design philosophy is like that and downtime probably peaks a little bit later when the ground motion is of a reasonable intensity because only after the damage exceeds some level we require buildings to be closed for repair whereas the injury kicks off even later, right. But when we go to the last ground motion then the consequence of probably death and consequence of downtime is more than the consequence of damage so now if we divide that down into three basic categories of low probability, medium probability and high probability, what you get is when high probability even or smaller ground motions (inaudible 10:45:31) most of the damage or loss is contributed by damage which are of course minor damage but repeatedly and very small contribution from downtime whereas when we go to the medium probability which is the moderate level of ground shaking then the contribution of downtime peaks off quite severely. The damage will of course be high and the injury just starts kicking off whereas in low probability which is the high ground motion, very intensive ground motion would probably would require the buildings to be closed so the downtimes is very high, that damage is 100 percent and death also kicks off, injury kicks off, right.

Now let's look at it for the low probability event and high event. It is low risk but high consequence, very important for life safety criteria. It must be avoided at any cost. We do not want to compromise life safety, whereas when we go to that range these are low consequence to individual owners but it occurs to many buildings, hundreds and thousands of buildings and more often because of their probability, but in terms of the accumulated risk it contributes a lot. Big blow to the economy. Mainly contributed by minor to moderate damage and the resulting downtime, so this is what we must avoid in future. Not manipulating the life safety criteria but the damage in that minor to moderate ground motion. That is what we have to avoid and in order to reduce damage in minor to moderate shaking, what do we need to do?

How can we reduce damage then? We can either provide very high capacity. Of course you might have seen in several ancient cities you might have visited many courts, palaces, temples, churches which have seen through several significant earthquakes still standing there proudly merely because of their huge size, because of their elastic capacity, okay. We don't want to apply that principle in our regular building design because it would be too costly. It is not economically viable, so the other way is to apply low damage technologies or what we call new building technology and some of them I just briefly list here: base isolation, external braces, dampers, damage avoidance design which is named in different ways. So just to make my point clear. Despite knowing that there will be speakers speaking about this in a lot more detail I just would like to start basic of these technologies. How do they contribute to the purpose of loss optimising seismic design? Let's look at that.

Base isolation as we all know it is passive control technique to decouple a building from its foundation by using isolators. Base isolation does not make a building earthquake proof. Again that is the message to be given to public. It just enhances the earthquake resistance. It makes the likelihood of damage a lot less. Not suitable for all buildings. Most effective for short or medium rise buildings on hard soil. Consciously I'm going through this section a little bit quicker because I don't want to spend lot of time because there will be presentations specifically addressing these issues.

So what does an earthquake do to the buildings? Earthquake doesn't apply any force to the building. Earthquake just moves the base of building and in doing so it does it pretty fast. If the building base is moved slowly – no issue at all – the building can move together but as it moves the base very fast that will create what we call inertia force. That will force the building to bend like when we drive our car we suddenly accelerate, our lower part of the body which is connected to the seat which goes together with the car but our upper part of the body tries to swing so our body bends. If we had a technology which actually bends and doesn't damage in bending, the purpose is met regardless of what we call force, deformation or whatever it is but if we had a structure made of let's say for example a very deformable material, rubber, if it is viable then the purpose is done it will never be damaged, okay, but things don't work like that. Of course rubber is a good material that's why we do use that rubber in our base isolation technology for the isolators. What does it do?

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Is it now you know like separates that base movement the foundation movement of the ground and helps it not to be transferred fully to your structure by a very much deformable layer and that's what it is doing, okay? If we have our building on rollers the building doesn't deform but it vehemently moves related to the ground without any control which we don't want, okay? But that's why that deformation layer is put there and which helps the building to mitigate the demand from the earthquake. So of course that layer plays a very significant part in base isolation scheme and that has to deform at the same time. It also has to pass the vertical load coming from building to the ground through the foundation so it has to be very much soft in horizontal direction very stiff and strong in vertical direction. It's a complex requirement. That's why the design of base isolators needs very much technological or engineering thought. There's a lot of thought that needs to go behind designing those layers and there are some of them with New Zealand is one of the pioneer in designing base isolation technology. I wouldn't want to go through the details. There are several different types of base isolators and is it economically viable.

What are the cost implications? I just was searching the web and I found this one paper written last year from researchers in the United States University of California Berkley and that paragraph explains what they have found. *'Base isolation has a higher initial cost than conventional construction. The cost increase come from isolators, excavation, construction of an area, extra level that provides no additional useable or rentable space, stiffening of the superstructure, and a moat that surrounds the isolated layer. This leads to an additional cost of $50 per square foot'* – that comes from some research from them. *'Building owners and developers are reluctant to pay this additional cost. However, the benefits of isolation far exceed the initial cost of installing isolation especially in today's society'.*

I told you already the life time cost implications. If you look at that initial cost verses the return of that cost in long term then basically we will offset very quickly if we take into account the saving of the ongoing cost, annual costs.

The next, the second category I mentioned as damping devices. External dampers. What do they do? They absorb energy and aid damping to buildings in order to reduce seismic response. They specially, they are specially suitable for tall buildings which cannot be effectively be base isolated, okay? And the third point retrofitting existing buildings is often easier with dampers than with base isolators. So I am saying it often, I am not saying it is not possible with base isolation because my colleague, professional colleagues over here I know they have done it. Yes, so there are buildings which have been retrofitted by or improved by this isolation existing building so it is not impossible but in my opinion it is easier with the external dampers. Example, there are several types of dampers, viscous dampers, friction dampers, hysteretic dampers and these dampers are put into this structural system in such a way that when the building deforms, when the base is moved the building deforms and when the building deforms that exerts extra related movement in these dampers which inherits or which triggers the force velocity dependent forces, damping forces from this distance. In some cases by yielding dependent forces, in some cases friction dependent forces and that will add to the structural response and that can be arranged in building in different ways but the ultimate goal of all of these like the yielding dampers, the frictions dampers, viscous dampers, they work with different mechanisms but all of them are trying to improve this structural response of the building.

Applications in New Zealand for base isolation. There are several applications in the world, but I am just picking up a couple of cases of applications in New Zealand. A Union House building in Auckland that has got the bracing system outside pretty much visible and I have been told that on top of that it has got steel dissipaters, dissipaters to dissipate the energy at ground level between the column and the foundation. And Te Papa in Wellington, that has got base isolation system and late Bill Robinson, his company Robinson Seismic they have developed the lead rubber bearing which has been used in that Te Papa building in Wellington and that is one of the photo with Bill Robinson and his team during the installation of that base isolation. And Christchurch Women's Hospital it has got 14 lead rubber bearings, one of them pulls up a look at that this one here and as we all know the building performed very well in the, in the earthquake. Recent earthquakes.

Now the last one to achieve that reduced level of damage, damage avoidance design or we call it hybrid system, PRESSS system, rocking system, damage resistant design. The overall principle is the precast components are used which are assembled at site. They are used, tied using unbonded post-tensioning which provides the strength and we use external dampers for the energy dissipation and it is self-centring system. That is no residual displacement. What it means is for our modern design once the inelastic response happens, once the structure yields even after the earthquake stops, the building or the structures will be tilted. They will not return back to its fully straight state. Grand Chancellor, you can recall that building in a tilted position right, so that is residual displacement whereas in this technology in this new design philosophy the structures come fully back to normal. So I'm aware that tomorrow my colleagues are going to talk about the application of this, this design philosophy in reinforced concrete, steel and timber structures so I'm not going to deal with all the details so I'm just giving an overview and saying how it affects the loss optimisers in seismic design or how it contributes to.

So the structural damage in a nutshell it is associated with inelastic deformation. If the columns on here is expected to deform this much in an earthquake then currently that deformation will bank on the inelastic response of the column which of course doesn't collapse and doesn't kill people but it deforms elastically so the whole idea is if we avoid inelastic deformation of member we can avoid damage because that damage is affiliated to inelastic deformation. So for the same displacement demand we can have the column being deformed or being, responding in this fashion rather than that fashion and in this fashion of course there will be a huge stress concentration on that corner which might require the designers to concentrate on providing spatial detailing to that part through armoury and which has to be taken care of but that particular damage is like cracking, yielding, buckling, spalling, all those things, they are taken care of. That's the general philosophy in a nutshell.

We have applied that not only in theory also in practice. In the lab we have tested structural systems designs based on the modern design philosophy and the futuristic damage avoidance principle and we have compared the same structures which were designed for the same level of demand subjected to similar ground motions and at the end observing the damage in these systems and visibility. Of course both of these systems didn't collapse so the life-safety criteria was met but you look at the damage definitely it speaks a lot about the loss induced by our modern design versus what the new system can achieve and we have not only done that to a single pier but also done that to a three-D frame subassembly and again here we have achieved that rocking mechanism which dissipates or caters for the displacement demand and the external dissipaters which caters for the external energy dissipation. Different types of dissipaters have already been tested and my colleagues in the university have also tested this theory in walls and other systems, extensive testing have already undergone. So we have come to a conclusion after those tests that this works pretty well in reducing the structural damage. I am saying structural damage because still we need to look at what it means for the non-structural damage and for the content damage because we, I just thought before a few slides that actually in a building the contribution of non-structural and quantum damage outweighs the structural damage contribution in terms of financial loss.

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Of course structural damage is very crucial for life safety, for collapse prevention but the non-structural damage and content, they contribute heavily to the loss. So that part needs to be checked because we haven't put as much focus in checking the non-structural and content damage in damage avoidance systems as we have done for checking their structural stress points. Now even for this system the applications in New Zealand we have had a couple of applications. Victoria University in Wellington has the first PRESSS building in New Zealand and the second multi-storey PRESSS building in New Zealand which is the first in the South Island in Southern Cross Hospital in Christchurch and reportedly it performed very well, structurally performed very well in Christchurch earthquake. I don’t have any detailed information on how it performed in terms of content and non-structural system. Okay.

So having talked about all these things what we have come to is actually okay, a design scheme which we have been applying now that works very well in achieving life safety criteria but it doesn’t work well against minimising the loss and as we have seen through the backlash or through the reaction not only from the public and authorities from EQC but also from engineers that we are not particularly happy with the level of loss that we have incurred in these ground, in these earthquakes although in modern buildings the life safety criteria was achieved and on the one hand the engineers are saying if we talked to Rajesh the engineer he'd say that in general the performance of buildings in Canterbury earthquakes exceed the existed expectations. Why I say existed expectations is because many of these buildings which were designed earlier were designed for a lower level of demand. If you look at the earlier versions of our seismic design code we were designing for 8 percent of the width, the lateral force was equal to 8 percent of the width and now in some cases like for example if you are designing a building in, in Arthur's Pass which has a seismic design coefficient or zone factor of .6 and we are designing for a low ductile, ductility system then your demand, force demand is about, you know like almost 15 times of that 8 percent of the seismic width. So we have gone through that increase in seismic demand through various steps during our code update. The standards have been updated because we learned something, some lessons after each major earthquake and we keep on updating because the code which existed some time ago was as good as the state of our knowledge which existed then and of course after every earthquake we are becoming wiser and wiser and accordingly we are changing, we are updating, we are improving our design philosophy. That's why the expectation needs to be existing. We can exist, we can update our design code and for new buildings we can require a higher level of demand but we cannot automatically adjust the buildings which have already been built. That's why the need for retrofitting. Okay. In general the performance of building in Christchurch earthquakes exceeded the existed expectations.

That's what Rajesh the engineer would say and on the other hand Rajesh the citizen will say, hey look, really, the earthquakes have cost us $30 billion with amounts to be 15 percent of our gross domestic product. Did we expect to lose more than this? You're saying that we exceeded our expectation. So you mean we expect to lose more than this? So do we really need to go completely broke for you to say that we haven't done well. The citizen is confused. How have we done well? You know. But the engineer is right in his place and the citizen is right in his place too.

What my position, my, my position to the whole community including myself is, let's give up our resistannce to change attitude which I have observed to some extent in New Zealand since coming 10 years ago and work towards an approach that will require structures to meet the expectations of both Rajesh the engineer and Rajesh the citizen in future earthquakes. That's what I have to say and with that I thank all the audience for their patience. Thank you very much.

JUSTICE COOPER:
Dr Dhakel thank you very much for your presentation.

MR MILLS:
Thank you doctor and you'll be back for the panel later in the day of course won't you?
Now the next witness who's going to comment on that paper and make any points that he may wish to make is Dr Richard Sharpe who the Commission will be familiar with.

MR MILLS CALLS

RICHARD DEAN SHARPE (AFFIRMED)

Q. Now we've been through this before but let me again just for the record go through a few personal details. Your full name is Richard Dean Sharpe?
A. It is.
Q. And you're a resident of Wellington?
A. I am.
Q. You have a PhD in Civil Engineering from the University of Canterbury?
A. I do.
Q. You're a chartered professional engineer?
A. I am.
Q. You're currently a technical director in earthquake engineering in the Wellington office of BECA?
A. I am.
Q. You're a past president and fellow of the New Zealand Society for Earthquake Engineering?
A. Correct.
Q. In 2007 you were made a distinguished fellow of the Institution of Professional Engineers?
A. I was.
Q. And that was in recognition of your contribution to earthquake engineering.
A. I believe so.
Q. You were recently appointed the theme leader for building engineering resilience, the natural hazards research platform?
A. I have been.
Q. And with that I'm going to sit down and let you take the Commission through your PowerPoint. Thank you.

DR SHARPE:

A. Now Commissioners this is a rather daunting task I have to comment on, the Encyclopaedia Britannica of Earthquake Engineering but I come, I think you will find that I come to it from a slightly different perspective and listening to Dr Dhakel I, I made myself a few notes and I'll say what those are and then I have prepared something that might, counsel assisting spoke about this session being on the history of earthquake engineering. I'd like to dwell on some of it because I think there's some very good lessons for us in that and my comments on the last session in summary, I would say that earthquake engineering as the Commissioners will know is as much an art as it is a science.

I don't agree that we've got much wrong. There's always room for improvement but I think we've got a lot to be extremely proud of and I think that when I show a little bit of the history we'll see that. I'm interested in the statistics of payback on retrofits. I managed to escape from my secondary and tertiary education, most of the statistics courses and I regret some of that, but six or seven years ago we undertook a study in Turkey of 369 five-storey apartment buildings to look at the economy of, the economics of retrofitting them and of course one of the very strong things is how soon the lady upstairs who pulls the levers gives you the earthquake after the retrofit and if you get the retrofit very quickly after the, if you get the earthquake very quickly after the retrofit you get an extremely good return on your investment and, and I think that I will bring that out in some of what I say about, we don't know much about these earthquakes that we might get or when we might get them and I've made my notes and then Dr Dhakel also made the point later on that of course we can avoid damage by raising the loads and, as he said, well we can minimise damage by raising the loads but it has an economic consequence.

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So these are complementary views from a consulting engineer. I can go through some of the realities about design for earthquakes and you can see there some of the things that I will go through, how certain are our design loads, performance based engineering. If I understand it correctly it's not new and I'll talk about some of the innovation and design for earthquakes, give some of the examples that Dr Dhakal has talked about. I think one of the questions is, and I'll partly answer that, who is willing to pay for better resilience? Some realities and I've made some notes here that I'd like to talk to.

One of the main ones is that we forget often that moderate earthquakes occur more frequently on average than major design level ones and Dr Dhakal said we should be concentrating perhaps more on those moderate earthquakes than worrying too much about that very rare big one, and I believe that our design philosophies acknowledge that there's always a chance that a properly designed, properly constructed structure will still collapse in an earthquake and that's because our design philosophies are based on a probabilistic statistical basis. All our design, our material properties, our understanding of the earthquake, everything that goes into it has the probability of not being what you thought it was going to be and I think we'd had a huge experience of that where we got an earthquake of a magni..., we got an intensity and effects of an earthquake, say magnitude 6.3 or thereabouts, that we didn't really expect for one of that sort, but we got the power ball on top of the first prize in the lotto and we don't change the odds with the Lotteries Commission I don't think change the odds when somebody wins Lotto.

I think it's necessary for us to recall that the derivation of earthquake design loads is a huge amount of uncertainty. We have shown the Commissioners in the previous, in our investigations in the PGC building for instance we've shown some graphs which show the recordings of the earthquake in Christchurch compared with the design codes. If you look at those red lines and the green lines representing design levels and you see that the wiggly lines behind are the earthquakes you would be interested to see what goes into the process of getting those very smooth red lines in there. We're one of the few organisations that actually carry out this sort of cradle to grave analysis of how to derive earthquake loads and there are many times at half past seven at night we're just about throwing darts at the dart board or choosing a wider felt-tipped pen to be able to get these sorts of curves for design purposes. The seismicity model, even though I would defend this as being a world leading model, a huge amount of uncertainty in our seismicity model and when we go to come up with design, the hazard factor which sets the load for our design which tries to equate the risk across the country, look at the fineness of the curves and the little perturbations in them. When you see some of the processes that went into that, the choice of whether we take, or one of the ones that intrigues me is once you have a seismicity model and you have some idea of how often earthquakes will occur round the country, you then have to work out the effect on a statistical basis at any one point in the country and so you have a relationship between the size of the earthquake and the distance that you're away from it. That's called an attenuation relationship because the earthquake attenuates, it dies out with distance.

Well there's a huge amount of judgement goes into processing all the information from existing earthquakes to get that attenuation in relationship and it's got to cover different soils on the arrival place. It's got to cover the different natural periods of buildings. It's got to cover everything, and then right at the end of it you decide whether you're going to take the mean, the median or the mean plus one standard deviation and I can remember the bit in statistics which tells me the ratio between the mean and one standard deviation. It's huge, and so these numbers are interesting, these contours. We used to have something much more simple. New Zealand 40 years ago had a code with had three zones. We used to inwardly smile about the fact that just outside each of the higher zones there was a major city on the lower zone side and we thought that the person who was drawing the zone boundaries probably had an eye to the economics of what they were doing in drawing the zones but perhaps we should go back to one zone for the whole of the country.

JUSTICE COOPER:

Q. Well who did draw the lines defining the various zones?
A. Wise people who had some geological knowledge.
Q. Did you ever meet one of them?
A. Yes certainly. They were certainly well around in my earlier days.
Q. Where were they employed?
A. I think it was a combination of wise men, engineers of course, with the geoscience people.
Q. So it's the same today?
A. Yes, except ours is now based on much more sophisticated modelling and these theories that we've got. I don't want to take anything away from that. I do think it is the future but we need to have a grounding in reality as to where these numbers come from. When we put our graduate engineers, make them do some of these analyses, one good thing we cure them of is ever using more than two significant figures in any calculation to do with earthquake engineering because they realise that even one significant figure in a calculation can be commensurate with the accuracy of the loads but it meets regulatory requirements to show that .99 is smaller than one and therefore okay as opposed to 1.1 and over.
Now talking about building requirements. They are already implicit in the New Zealand Building Code. This is out of NZS, New Zealand Standard 1170.0 which is the sort of mother ship of the risk in the way -
Q. That's an Australian and New Zealand standard isn't it?

A. It was a combined committee that developed that standard and our Australian colleagues backed off having exactly the same section for earthquakes. I think they at the time, as I heard it from my colleagues, they thought it was a bit hard to convince their own colleagues to go exactly the same way and so within that family of 1170 the standard was actually there are actually two sections for earthquake – one for New Zealand and one for Australia, Australia going back to keeping with a slightly older way of looking at it, but you'll see from this table 3.1 that the consequences of failure of these performance requirements in an another way and put there and if you go to another part of the next page it sets probabilities for wind, snow and earthquake. You have to remember there are other natural hazards, including cordons now. We're getting a lot of enquiries about the natural hazard of a cordon around your site but you can see that both the serviceability that is something about damage or operability is built into the New Zealand codes in the way that we deal with them.

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COMMISSIONER CARTER:

Q. Was that paper, is that paper published in what?
A. That's in 1170.0 which is the –
Q. Thank you doctor.
A. – basis of, of setting of loads, design loads for New Zealand.

JUSTICE COOPER:

Q. Yes I thought 1170.0 was an Australian and New Zealand standard?
A. It is.
Q. And that 1170.5 was our special section on earthquakes. Is that, is that right?
A. We have to comply with all sections –
Q. Yes.

A. – obviously and that is correct, that 1170.5 specifically tells us how to take, go further but the actual probabilities are set in this, the mothership of that building series. I think it's useful to look at some of the history. This is Maui B. Some of our industrial clients in industries in New Zealand have been onto this a lot longer than commercial building owners. So what we can do is, you need to get the concepts right.

Performance based earthquake engineering's been practised for more than 40 years and if you take for instance, I was working in the nuclear power industry 30 something years ago and there the two levels of loading that they always looked at there, the lower one is the operating basis earthquake which is the one which the nuclear power station should shrug off, keep going, no stop, and then there's the much higher one which is the safe shutdown earthquake which will a much less probability of occurring. So these things are very, have been well established for many years and as my, I'm prompted by my photograph here, the Maui platforms have been designed with the same concepts. The, the owners of these have, you know these were designed in the 70s and the 80s, put in a huge amount of investigation into what earthquake loads they should put into these. Maui B, liquefaction studies done on the sea, marine sands and the typical return periods for, equivalent to those nuclear ones are I think something like the operability one here for Maui is something like I think 350 years and if I recall rightly that the, the safe shut down equivalent is something like 3,500 years I think. I, memory, I haven't been able to check those recently.

So we, we have been doing that with industry. Other people are doing it. Transpower. Transpower has a seismic policy which is very stringent and very comprehensive and only a few pages long but the, the survivability of the city depends on its infrastructure and we, we have survived this extremely big earthquake and a lot of it is due to people like Transpower who have, have the grasped the nettle on, on the standards and how they procure things and, and, and understanding performance requirements. So they in their policy actually have performance requirements which are translated back into our codes and into these return periods that we talk about.

Carter Holt Harvey with its Kinleith and Kawerau plants. These are huge structures they've got there and they have been over the years dealing, well before the Christchurch/Canterbury earthquakes, have been dealing with what to do about retrofitting structures built before earthquake codes were like they were now and even finding out that things weren't quite what they thought they were and this, this is the case here. The ringed one here is of a 1000 tonne steam boiler which they found was not up to standard and they went ahead with base isolating it. So over a two-week shut they, annual shut, after a hell of a lot of planning they lifted the thing up, took it off its existing bearings and put these base isolators underneath it.

Q. Was that at Kinleith?

A. That's, that one is at Kinleith, mmm, and – I don't think I've put it in, at Kawerau recently they've done a huge upgrade, seismic and a heart and lungs for the boiler on a, a number 2 recovery boiler which went through to the '87 Edgecumbe earthquake and that, that is a massive thing. That's a 1500 tonne big box suspended from, 1500 tonnes of steel, a huge structure and the, the boiler which comes from overseas suppliers who don't know too much about earthquakes is cocooned within, now within energy absorbers to keep the loads down inside it.

So what I, the point I'm wanting to make here is that many people are actually onto this performance based design. Interestingly in terms of all the, everybody now knows what new building, a percentage of new building standard means in the Christchurch context and I'm noticing that particularly the big industrial and utility clients understand what to do with the percentage new building standard because they can integrate it with a little bit of help into their health and safety policies.

They have developed the concepts of risk as the health and safety policies have ramped up and this is one more health and safety thing and so we are seeing some quite rational decisions on, on the way forward in dealing with the realisation that some of their office buildings are not quite what they might have hoped them to be and there's been some really good examples of that.

Q. Can I just ask you to clarify something which at least for me is not clear. These examples that you're giving of Transpower and Carter Holt. Are these examples of industries for particular reasons protecting their investment probably high up, are exceeding code requirements or complying with code requirements?

A. I think that's, Commissioner that's a very good point in that in many cases it would be uneconomic to meet 100 percent of new building standard on, on say the number 2 recovery boiler at Kawerau. In fact at a meeting on Friday this was discussed at length with an IPENZ working group on producing better standards for industrial structures, seismic standards. The, as many owners of commercial buildings are understanding there is a toss-up between where you want, the level of risk that you're prepared to take and we know that we, that we live with many risks that we decide are, drive a car with or without airbags or so on. They're not required by the Building Act to take this boiler up to more than one-third of New Building Standard. So what should they take it up to, given its design life and the cost and the implications on, on production and they, generically these industries are able to work through the problems I believe and that's what, that's the message I'd like to give to you that on the industrial side they are able to come to grips with it and I think, I'm saying that because I think many commercial building owners are trying to come to grips with the same thing and industrial people are ahead of them on that.

New technologies, new technologies have been around for a long time. Mr Holgate designed the Wellington Railway Station in the 30s and my understanding is that he had special bricks made which were notched to take reinforcing bars in them because this was after the Napier earthquake. So there's some innovation.

Q. He was the structural engineer was he?

A. He certainly was and I think in 1935 he got the equivalent of IPENZ gold award for earthquake engineering.

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Q. I think, most people think of that building as being designed by William Gummer. You've given us a different perspective on that.

A. Well nobody's - yes, yes. You could debate that one for a while. Where the real worth is, but another more interesting thing about this is that this building has been strengthened quite recently by a number of techniques including vertical rods down and stressing them up which I was rather amused to hear on the television last week that we were going to import this technology from, somebody was suggesting we should import expertise in this technology from overseas in terms of the Cathedral. Well it's been done here in New Zealand. It's not rocket science and it, but it requires some foresight and to get on and do it. It's been done. That's what I'd like to say here.

COMMISSION ADJOURNS: 11.31 AM

COMMISSION RESUMES: 11.47 AM

JUSTICE COOPER TO DR SHARPE:

Q. Yes well we were at the Wellington Railway Station.

A. Can I just, Commissioners, can I just go back a point before the Wellington Railway Station.

Q. Yes.

A. I had some notes that I overlooked here. I believe the commercial building sector is struggling to come to grips with the implications of the 2004 Building Act eight years later and prompted by the Canterbury earthquakes and I say that in contrast to what I've said about the, I think the industrial utility people are doing it better. There are more parties in the commercial building sector to be educated and any one party, the owner, developer, architect, the engineer, can stymie the implementation of a sensible approach to earthquake design and I think I just wanted to, as part of the history, to say that many of us recall the heady days of the construction boom before the 1987 Stock Exchange bust and that's, some aspects of that have continued in many ways until the 4th of September. Seismic codes were treated very much at that time as, as minimum standards. Selection of engineering consultants was on their reputation for meeting the leanest complying structures and I vividly remember the senior man in our office coming back from a meeting in the 80s there saying that a developer, a well-known developer had sat him down with our drawings, structural drawings and put his finger on various items and said, "Is that piece of reinforcing steel needed in accordance with the code?" And when he was told not he said, "Take it out," despite the protestations that we were ahead of the code and knew that that steel was required and the result of that was we never worked for that developer ever again which was the principled stand to take.

Q. Some buildings were designed on the basis that they would only have to last a comparatively short period because then it would become more economic to knock them down and build something else because there was the idea that property would go on increasing in value and price and the sensible thing to do was to plan, was to put up a building on the basis that it would be knocked down and something else would be erected in its place as I recall it from those decades.

A. You're probably referring to the loss of the Duke Hotel in Manners Street perhaps?

Q. I wasn't in particular. I was thinking of buildings in Auckland actually. But what, what I, could I just, if you've finished what you wanted to say on that point, can I come back to a question that I asked you earlier because I'm just wanting to understand whether you were saying that in cases that you referred to of public utilities and also examples in the pulp and paper industry that the people, that the owners of the structures were actually going out and exceeding code requirements. You gave me an example which was really about retrofitting but I'm talking, I suppose I'm interested in whether you were saying that in the case of new structures there has been for some time a posture by those involved in those other sectors to do what was necessary to preserve the utility of the items of plant that they were building as opposed to treating code requirements as a minimum, sorry, as a maximum I should be saying, as something which needs to be met but not exceeded.

A. Yes.

Q. Yes.

A. I think, I think one of the little points that I'd like to bring out Your Honour in, in just going through some more photos is that it's quite interesting to see who are the clients who are backing some of the innovation because they, I think you can pick a pattern and who are, who are going for innovation and that it's probably really the commercial sector as much as some of the institutions and some of the industrial sectors. So that might come out in –

Q. Right.

A. So I was just saying about innovation, these things have occurred long before the, some of the innovation that you will hear about, there we are - is the, getting away from buildings is the, the South Rangitikei Railway Bridge. Now this, this is something that has taken the world, this is well-known around the world because of its innovation. The piers, the twin piers down there for an earthquake are predominantly at right angles to the bridge. It will actually stamp its feet up and down as a way of relieving the forces in the bridge. So it steps. So those piers are stepping piers and I know that – although that's a pretty old example from the 70s that's showing that we were well on to doing some pretty innovative stuff in those days. Here's another one that you might have seen quite a few times without realising what it is. This is round the back of the number 1 hanger for Air New Zealand out at Christchurch Airport and this is actually supporting the flues from the boiler house and this object here can also rock around and when it rocks around anyone of those four protrudence, buttresses at the bottom and on two of them there are these steel plates in boxes underneath there which as, as it tries to uplift its foot it will yield that plate and produce a lot of damping. Now that's, that's the combined work of – well-known as one of the grandfathers of earthquake engineering in New Zealand, Dr Ivan Skinner, in collaboration with John Hollings of BECA. So there are other examples on our doorstep that people know nothing about, have probably forgotten. There's another very unusual one, this is the Hapuawhenua Viaduct just north of Ohakune and this is a particularly challenging one because as well of the earthquake you've got the braking forces of the train. It's on a slope. It's on a 420 metre radius. It's got everything and yet it's got some very interesting details at the abutments. That's where the, where the bridge starts off the land to accommodate the movement that's going on there for, for earthquakes.

So here's another one in Wellington. These are the, this is the Jerningham Apartments in behind Oriental Bay there. One of the earliest buildings in the world which was deliberately designed for high levels of ductility in the building, in the reinforced concrete frame and that was being designed in the late 60s so we were onto innovation even, even then.

Here's another one that's a little bit more related to domestic, this is the Wellington Emergency Management Office in Molesworth, in Mulgrave Street. One of the interesting things here if you go inside is that they've disconnected all the GIB board within it so that it sort of floats on the walls so that you don't get all the cracking around the GIB board because they didn't want to have that damaged in an emergency management office designed for earthquakes. So some innovation went into there to do that.

Q. So what supports it?

A. Well I, I don't know the exact details but the detail around the bottom, there's a sort of a gap and a floating architrave around there so that it's sort of like a fish scale that moves on the structure but doesn't cause cracks across, across the bottom. So people have been into this sort of thing. Another little interesting thing there is that picket fence is actually not quite what you think it is. In the corner, just behind the man there, those are steel posts to stop a runaway truck coming down the hill and going in there, so it's not what it looks like. But coming back to your point before, just across the road of the building that's an extension to the Royal Society building going up there and those panels on the left, I understand, have been designed to rock, so you will hear more about those rocking wall type technologies. So there you are, two buildings very close together but the technology being put in by an institution that might be quite interested in promoting that sort of thing. There are many other examples of buildings which have little bits of technology in them. This is on the Wellington waterfront and that's got bars down through the bricks again, to stress them, and it's got another structure inside with a little ductile pieces. You can't see any of that but it really does enhance the character of their….

Here's another interesting one and the point that I'd point out here is look at those chimneys. They're fibreglass, so you can get away with it in terms of seismic resistance, resilience, innovation by doing something like that. I couldn't help touch on base isolation because we're actually working on the first lead rubber base isolated building in the world right at the moment to give it a longer life and this was originally the Ministry of Works district office for Wellington and, you know, it's still showing up really well and it'll even be better when we've finished making the gaps a bit bigger and so on.

Wellington Hospital, of course, and the hospital's, I was trying to find the reference to it but there is a directive, I believe, from the Ministry of Health about the performance requirements of hospitals, major trauma hospitals, to be able to operate after an earthquake which really forces you, because of the contents, to base isolate these and so you're getting a rash of these, but some hospitals, I know, have gone against doing it and one example, a colleague said, for a cost of two million dollars that was what was estimated would be the extra, they didn't go ahead with it because it was probably competing with an MRI scan machine or the salaries of the doctors or something. So, you get what you pay for, and there's the example there again.

Value of the contents. Well here's base isolated building in Wellington you may be aware of and, of course, there are many other base isolated buildings but we haven't –

JUSTICE COOPER:

Just for the record it's the old Supreme Court Building.

DR SHARPE CONTINUES:

Yes, to the old part is base isolated.

JUSTICE COOPER:

Q. And is the new part not?

A. It's not no. I think I'm right. Yes, I believe I'm right in saying that. And some people have now perhaps forgotten that this whole block of what was the old BNZ in the Featherston Street, Lambton Quay, Willis Street corner all the buildings were tied together and the whole group have been base isolated.

Q. Were they and all those buildings in different ownerships or was there more than owner of that block, do you recall?

A. I don't know. I think it was done, I suspect it was one owner, if I remember, yes I think the history of it was, yes.

Q. Because often in these cases of these commercial buildings the sensible retrofits are dependent on the goodwill of a number of owners potentially, aren't they, and if they're practical, difficulty.

A. Yes I think that must be true yes and Dr Dhakal mentioned the Union House, which I had on my list, on the Auckland Waterfront which has floppy piles in it so the piles are within a tube so the building, and not on rubber base isolation, but the actual flexibility of the piles is allowed to occur with isolation with energy absorption at ground level and this is the Wellington police station, of course, which has the same thing although there are many examples around but the rest of the world's overtaken us. We're not doing it nearly as much as the people that we've been teaching overseas and we're now, because in our company we do bearing design for one of the international suppliers of bearings, we've done trial bearing designs for hundreds of buildings overseas in every country you'd like to think of. We've one being built in Tehran at the moment, Turkey, Greece, India, you name it, Taiwan and China, thousands of buildings being base isolated and, as I understand it, in many countries, Japan, you get a premium for your property because, or your apartments if you're letting it out, because you've got superior technology, so it's been driven in some cases by the market but it's hard work to get people to pick it up in New Zealand. So, having led the world we're now supporting and watching. If you go to the capital of Armenia and the international airport the bearings were designed in New Zealand and supplied from a New Zealand company.

Q. What's the reason for this?

A. It's because, I think, very simply we've been a little bit complacent. People are not in the frame of mind of understanding the benefits in terms of the long term benefits, the self-insurance nature of base isolation and I think we might see some of that come out of the insurance industry where effectively putting a little bit in to self-insure by having something like base isolation or any of the other techniques will reap its rewards in terms of premiums reductions, we would hope, and one of my colleagues was telling me on Friday that he reckons that the information that we get for bearing designs is actually the Turks have come onto it better than us in terms of their understanding. They're really starting to work at it, particular in hospitals because, as you know, Turkey has a huge, is very vulnerable to earthquakes, is going to get more big earthquakes and is doing something in a big scheme that is spending billions of dollars on upgrading public utilities, schools and so on.

It was great to see in our newspaper in Wellington on Saturday that the proposed replacement for the BNZ building, Cathedral Square, may have dampers incorporated and visible behind the glass façade, so making a bit of a feature of it. In Auckland at the university they're talking about ripping the tops off some of the buildings and putting even more stories on the top and to make that all work they're talking about using buckling restrain braces which are these criss-crosses on the side of a building but they are a little bit of innovation in those so that when you compress them they don't buckle, that they actually maintain their straightness and take the loads. There are examples in New Zealand of that being used already. There are hundreds of examples of it being used in North America, retrofitting as well, and these things can be bought off the shelf. There are at least three companies that make these where you can go and order them from a catalogue and that's an interesting thing where some of these devices are captured within the commercial situation and, you know, that's what it might look like inside which some people don't like the look of those things and we always get told about it but Associate Professor Andrew Charleson at the School of Architecture, an embedded engineer in Victoria University in Wellington, he is well known round the world for his trying to bring along the architects to embrace some of the aspects of the bracing in the seismic design in the architecture so I draw that back to the BNZ example. So, in some ways, buildings you get what you pay for and how much you're prepared to pay to get something better than the minimum. Here's one that I have something to do with in the 1970s and it's been recently rated up about 100 percent of the current code standard.

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Q. Just tell us what that is?
A. That's the ANZ building on Lambton Quay, Featherston Street and so 1970s, late '70s and that's good to go now and then it's been publicised recently, a much more recent building has been found not to be close to the mark.
Q. Which one's that?
A. And that's the Majestic Centre.
Q. In Wellington?
A. In Wellington. So just to sort of conclude on the aspects of you get what you pay for and you can choose to have higher standards. I think we've forgotten some other lessons in all that and one of them is the fix, fasten and forget as I once heard a Christchurch columnist saying we've got one part of the three right – forget. And there's no doubt that for a very small amount of money we can do far better. I know that many buildings I went into here after the first earthquake and saw a lot of destruction, went back and there was the same stuff lying on the floor in the second one, all for the sake of a $2 little strap on it to hold it and I think we have to pay more attention to building services even though the

codes are nominally dealing with it, this shouldn't happen to a building that's been occupied for only five weeks.

Q. Where was this?

A. It was in Christchurch. And here's another example. It was lucky. This was the 4th of September. You wouldn't want that ugly bit of steel to come down on your head there if you had been sitting in the chair at 4.30 in the morning.

So in summary I think getting the concepts right is far more important than getting the load level right, and as Dr Dhakal has said we don't design earthquake proof buildings. Even the best design building, best constructed may, there's always the probability that that building might collapse because we get a earthquake with a very low probability unfortunate characteristic in it. Damage reduction we have embraced aspects of performance based design for decades so in concept it's not new but we are starting to understand more about that and the need to take that into account in the particular structure getting away from the minimum standards where the owner wants something better. Damage reduction technologies are developing rapidly but if you are worried about the contents and you want to keep going immediately after this earthquake which we don't know much about – its size, its direction, I reckon in my opinion base isolation is the best one for reduction of damaged contents and institutional and industrial clients I believe are ahead of understanding risk in the commercial building ones.

WITNESS EXCUSED

MR MILLS CALLS

MICHAEL JOHN NIGEL PRIESTLEY (AFFIRMED)

Q. Again, just for the wider audience for these hearings Professor Priestley I'll just run quickly through some key points about your background. First, your full name is Michael John Nigel Priestley?
A. It is.
Q. You're a resident of Christchurch?
A. Yes.
Q. You have a PhD from the University of Canterbury?
A. Yes.
Q. You're currently an Emeritus Professor of Structural Engineering at the University of California, San Diego?
A. Yes.
Q. You are the Co-Director of the European School for Advanced Studies in Reduction of Seismic Design at Pavia in Italy?
A. Actually Emeritus Co-Director.
Q. But you were previously the Co-Director?
A. Yes.
Q. You're an Honorary Fellow of the Royal Society of New Zealand?
A. Yes.
Q. A Fellow of the American Concrete Institute?
A. Yes.
Q. A Fellow and past President of the New Zealand Society for Earthquake Engineering?
A. Yes.
Q. And your full curriculum vitae is on our website and of course it's enormously impressive and the topic that you're now going to speak on is Displacement Based Design.

WITNESS PRESENTS POWER POINT PRESENTATION

A. What I want to talk about is a bit of a discussion about the way in which we do our structural design of buildings which includes structures in general and to raise the possibility that we could be doing it better and thus within a performance based environment asking a number of questions – What should be the structural strength – what we call the base shear force – to satisfy certain performance criteria? Then how should this strength be distributed through the structure and is the way that we're doing this at the moment correct, and is current design philosophy adequate particularly with reference to new technology systems and that includes seismic isolation and some of the types of technology that you'll hear more about tomorrow in particular.

My contention is that the way in which we do our seismic design at the moment is too coarse and results in very non-uniform potential for damage in earthquakes and I think that, to some extent, the evidence is before us in Christchurch that some buildings seemed to perform very well and some did not perform very well though they were designed to code and there are a number of reasons for this and I'm not suggesting that these are the only reasons, but problems with the way in which we do design at the moment which is a force base environment that we do this within. We think about forces rather than terms of displacement and it's important to perhaps notice that damage is related to strain or drift, not to strength and there has been already in the two previous speakers have said, "Well yeah of course you can increase the strength and make the building safer for damage" and that's not necessarily the case and I'll talk about that a little bit later.

So some of the areas of problems that I see is with current design the estimation of elastic stiffness which is a fundamental aspect that we need for determining the period of the structure and as a consequence of its period its base shear strength. The distribution of required strength which is also based on elastic stiffness. Displacement equivalent rules and what this means is that we use elastic design or analysis to predict what the displacement in the inelastic system which is responding with ductility. The specified ductility or force reduction factors that are used in the code. In other words, we have specified levels of ductility which are permitted for different performance limit states such as the anomaly elastic and so forth on the assumption that increased strength reduces potential for damage.

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A. Just to provide some illustration of what the problems might be I've chosen a very simple example here, because it is so simple, but the principles apply also to other structures such as building structures and what we've got here is a concrete bridge under longitudinal seismic excitation with three piers of different heights and, essentially, this is a single degree of freedom system, making certain assumptions which are not worthwhile going into here, but, essentially, what we would do is, if we were doing the analysis of this under longitudinal seismic excitation, we would determine the stiffness, the elastic stiffness, of each of these three columns and sum them together to get the total longitudinal stiffness. We would use that then to calculate the building period and then, from that period, we could calculate both the elastic response or the ductile response and determine then what the design force would be that we would have to design the structure for and note that in this there is a force reduction factor which depends on the ductility, the anticipated ductility. We would then distribute this force between these three piers, these resisting elements, in proportion to their stiffness, their elastic stiffness, and to determine their design displacement, if we were looking at that at the very end of the design, we would use the elastic spectrum and determine the equivalent period based on some, sorry the equivalent displacement based on some assumptions about the relationship between elastic and inelastic response. Normally we would say that the displacement of the inelastic response would be the same as the elastic response.

Looking at that we can pose a couple of questions. First of all at the start what we determined was the sum of the total stiffness based on, essentially, what's called the cracked section moment of inertia of the piers itself and what's the value for this. Second question – what force reduction should be used in the previous equation, which depends on the ductility, and is the strength distribution in accordance with this relationship here logical, and the answer is no in any of these.

If we just take a typical column, a circular column, and look at what happens when we change the strength by either increasing the reinforcement ratio or the axial load ratio, what we find is that the stiffness changes and these are curves of force versus deformation for different levels of axial load on the column and the difference between these two is that the flexural reinforcement in this case is one percent and in this case is three percent. But if we do the normal linearization of this response by a bi-linear curve, which is having an initial elastic portion and a subsequent plastic or, at least, ductile portion, then we see that these dotted or dashed lines have different slope and the stiffness of these elements is, essentially, the slope of these lines.

So current design would say that the stiffness of all of these are the same and you can see that if we compare this stiffness here with this stiffness here it's very different and the difference might be as much as a factor of four or five or even more and, yet, in our current design we assume that the influence of axial load or reinforcement ratio is zero and we would use one value – typically .5 times the gross section stiffness – to characterise it.

If we actually calculate what the stiffness ratio is for these different possibilities for design as the function of the axial load ratio and the reinforcement ratio and express those as fractions of the uncracked section stiffness we can get a value that's between 12% and 90% and yet our design assumes that they would be all the same, that this is not a significant variable. The same thing applies not just to circular columns but to rectangular columns, to beams, to walls. We make these very coarse assumptions about the stiffness. So what we do in our design is, this is just again an expression of the stiffness and the relationship between strength and deformation, what we assume is that regardless of the strength this is determined by how much reinforcement perhaps or what the axial load is, we'd say that the stiffness is constant and, as a consequence, when we get to the inelastic stage of response the deformation increases as the strength increases and this just isn't the case. If we do analyses of this sort of thing what we find is that the point at which we get into the inelastic portion, what we call the yield curvature, is independent of the strength but that the stiffness, which is the slope of these lines, which we use to characterise the period of the structure and also to distribute strength through the structure, this stiffness is almost exactly proportional to strength and we don't take that into account in our design. So, looking at this structure here, we've already said there's some problem associated with what we would use in determining the stiffness to calculate our period and, therefore, to calculate the design force but we also have some problems about the ductility. What value of ductility should be used to determine the force reduction associated with ductility and if we look at those three columns again – this is the short column here – and what we find is that the yield curvature for each of these columns is going to be essentially equal if they have the same dimensions, regardless of what strength we put into them, but the yield displacement is proportional to the inverse, or is proportional to the square of the height. So the stiff short column goes into its inelastic range very early on. The very long column probably never goes inelastic and maybe the intermediate column becomes inelastic at this stage. Now this implies that column C has very large ductility demand, column B has no ductility demand, and column A might just have a ductility demand of unity. So very different values of ductility. How do we determine in our design which says for a particular level of design or damage we use a particular ductility factor. How do we do that? We can't do it in a logical sort of fashion.

If we think about this also a little bit further in the way in which the force is distributed, having calculated this erroneous base shear, how is this distributed between the piers itself? Well the shortest piers are the stiffest and the shear is essentially in proportion to the inverse of the height Q. The numbers don't really matter too much but what it does tell us is that the short pier should have a lot more reinforcement in it but, if we do that, I've shown you some results that would indicate that by doing so, by putting more reinforcement in it we will increase its stiffness still further. Well if we do a re-analysis based on these revised stiffness it will tell us again that we should distribute more force to that short pier, which means we need to put more reinforcement into it. The whole process is illogical and ends up with us attracting very high seismic shear to the most stiff element itself. We note that allocating high shear to the short column increases its susceptibility to shear failure and the displacement capacity of the short pier is actually decreased making the structure less safe if we increase its reinforcement ratio.

If when asked to look at a component of a building column, this is just a portion of a column from mid-storey height to mid-storey height and a portion of beam from mid-bay length to mid-beam length, often this is called a beam column self-assemblage, of which there have been thousands tested by structural engineers, and if we look at the deformations of this they're composed of a number of elements. The deformation of the columns and the beams and of the joint region – I won't bother to go into in any great detail – but one can come up with detailed analyses which indicate that the significant aspects of this are that the yield drift of beam column self-assemblage is related to the yield strain of the beam reinforcement and the aspect ratio of the beam, that's its length divided by its depth. A very simple expression which does not take into account all sorts of things such as the aspect ratio of the unit itself, the concrete compression strength, the beam reinforcement ratio, the column axial load ratio, these come out as being rather insignificant and if we compare that with test data we find that is the predictions, which are the theoretical drift ratio, match the experimental drift ratio rather well but this would imply again that there is no influence of the strength here that this yield drift is independent of the strength and just dependent on the geometry.

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Again our design at the moment would not say that. It would say that the yield drift depends on the strength that we build into the elements not just the geometry and we're wrong in doing that. Then by looking at these sorts of aspects we can come up with details about the yield curvatures or drifts for different elements of a structure including concrete frames and steel frames, walls and columns and beams.

One of the aspects that's perhaps of relevance to this is that when we look at damage avoidance in the codes that one of the aspects that is in there is to limit the drift to something like 2% or 2.5% typically and if we look at the simple expression we've got here mentioned in a previous slide and take a typical example with beam length or distance between columns of six metres a beam depth of .6 metres and using 500 megapascal reinforcement this would show that the yield drift would be 1.25% and so if we limit the total drift to this level here would imply that the ductility level for this example, not for all examples, would be about 1.6 to 2% and yet we design for very much higher levels of force reduction factor.

Looking at this force reduction factor we find that the different codes in different parts of the world have different values, typically depending on the limits state we might say for a concrete frame building the ductility factor and the force reduction factor might be six for concrete walls, it might be four, concrete bridges might be three. Well how consistent are we in different parts of the world in choosing these numbers or are they merely arm waving values and here's an example, steel frames, US west coast a value of eight, Japan between two and four, New Zealand the value if we take into account that, this thing which we call the SP factor essentially is nine and in Europe about 6.3 so very considerable variation between different parts of the world and this indicates there is significant uncertainty as to the relationship between strength and damage as we would perhaps expect.

And now if we look at perhaps how we do our analysis at the moment what we do is modal analysis where we model the structure, in this case a building frame, as well as we can and look at the response of this building under different modes combine these, that's the first mode, second, third and fourth and combine these in some fashion, and it's a rather sophisticated way of approach based on the elastic properties but one of the things that it doesn't take into account is that if we look at the first mode in which all the forces on the building, the inertia forces act in the same direction then this will create additional compression in column C and additional – and provide some tension in column A itself and that changes the stiffness of these so we can something or other where this column is very much stiffer than this column and we can't model that in a modal analysis because though we might say under the first mode that will be the case, in the second mode this column might be in compression and this one in tension and we get very coarse estimate of the, even the elastic distribution of forces in the structure from the modal analysis. When we take into account that in the inelastic response the columns remain elastic if our design philosophy is adhered to, whereas the beams become, will go into the inelastic range. The results of the modal analysis become increasingly coarse and give us a very poor idea as to how the structure is actually going to perform. We also, I won't dwell on this, but the way in which the equivalence between elastic response and inelastic response has been developed is by computer analyses using an inelastic time history analysis with I believe an incorrect value for the elastic damping which is rather critical and as a consequence the equivalence between elastic displacement and inelastic displacement values are incorrect.

Finally in these preliminary areas the question of, "Is high strength necessary or even desirable?" and both previous speakers today have said that by increasing the strength we increase the safety of the structure and it would be worthwhile noting that the Structural Engineers Society of New Zealand has recommended high strength to reduce damage of frame buildings by limiting the response to a ductility factor of 1.25 corresponding to limited ductility. Well high strength certainly does reduce damage in small to moderate earthquakes but it doesn't necessarily reduce damage in the design in extreme earthquakes. It does not necessarily reduce the displacement demand. It generally reduces the displacement, the capacity. It subjects the contents to higher acceleration and increases foundation forces and the cost so there is a very severe cost associated with this not necessarily for any good end result and I just illustrate this again by that simple circular column that I was talking about before where I varied the strength and on the left-hand side we have the current design philosophy which says that the stiffness which is the essentially strength divided by displacement or the slope of this line, the strength is, sorry the stiffness is independent of the strength and also a second aspect is to say that the displacement demand is independent of the strength itself, that's the so-called equal displacement approximation. So if your strength is this, then here's your yield displacement and here's the design level displacement. If I double the strength then this is the increased strength, we see that the displacement is the same but the yield displacement has doubled and therefore the ductility demand is halved. That's the philosophy. Well we'll take our little column and start off from a standard reinforcement ratio of one and a half percent and then we'll change the reinforcement ratio over the range that would be applicable from half a percent to 4% which is basically the full range that you could do with standard designs. As we increase the reinforcement ratio as expected the strength increases. As we reduce the reinforcement ratio the strength reduces and these are completely expected results. However, we see that the stiffness almost follows exactly the same trends. If we do some detailed assessment of the displacement capacity based on an ultimate concrete strain, what we find is that as we increase the reinforcement ratio we reduce the displacement capacity, sorry, the displacement capacity is this line down here. So if this was correct in saying that the displacement demand would remain constant then by increasing the strength we would have reduced the displacement capacity so we would have completely the opposite effect of what the intent was because we've reduced the, the displacement capacity, we've made it less safe. However, if we again take a more realistic look at it and say that as we change the strength we change the stiffness and then look at the relationship between the displacement demand and the displacement capacity then what we find is that as we do increase the strength there is a benefit but it is extremely small. Now here's the critical value her. This line here which shows the ratio of displacement demand to displacement capacity taking into account all aspects of the design. That is the change in period and so forth associated with the increased stiffness and you find that as we reduce the reinforcement from one and a half percent to half a percent there is no change of any significant nature to the displacement demand capacity ratio. As we go out to this value at reinforcement ratio of four, and note that there is a fourfold change in strength, we get a 7% improvement in the safety. So we have to use a factor of four times in the strength in this example to get a 7% improvement in safety which is hardly worthwhile doing. So the point here is saying is it not necessarily a good idea, particularly when you take into account the additional cost and perhaps additional risk associated with high foundation forces associated with strength in just treating strength as something or other to reduce damage.

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The second point is to essentially put out a different concept about the way in which design is done and this is something or other where we look directly to the limit state effects. If we say that damage is related to strain or structural elements or for drift for non-structural elements, should we not be designing to achieve those particular limits in the design rather than using a strength based aspect which is using a value of stiffness that we don't know and an irrational distribution of strength, and I'm going to skip through a lot of the material here but just to say that the approach is well developed. It is used and codified in some countries. There's a book here which has 700 pages of how to implement it in a design environment. Several dozens of PhD students have been involved in this and it is getting to be accepted in other countries, even Australia, but not yet in New Zealand.

JUSTICE COOPER:

Q. What's the book called?
A. Displacement Based Seismic Design of Structures, published by the IUSS Press in Italy.
Q. I'm told we've got it. I haven't read it as yet, you'll understand.
A. It works well as something to induce sleep or to hold doors open with – it's pretty heavy.

WITNESS CONTINUES WITH PRESENTATION

A. Anyway just to point out the difference, a fundamental difference between the current force based design and displacement based design is this represents perhaps an idealisation of the response backwards and forwards of a concrete structure in an earthquake itself and the current design uses an estimate of the elastic stiffness which can't be made at the start of a design process because it's dependent on the strength which we don't know and an estimate of the damping which is taken to be five percent. It uses those values to try and predict this point here which is the maximum response. Displacement based design uses the limit state displacements and a Secant stiffness which is this line here to try and represent the response. So a different stiffness and also uses a damping which is dependent on the shape of this hysteresis loop itself. So it appears that, well it is more logical to use this type of approach rather than this elastic one to try and predict this point here when there is no connection between these. So displacement based design is based on the observation that damage is directly related to strain (the structural effects) or drift (non-structural effects) and they both can be integrated to obtain displacements. Hence the damage and displacement can be directly related, and the design approach achieves a specified damage limit state. It's not possible to formulate an equivalent relationship between strength (or force) and damage and this is one of the major deficiencies in current force-based seismic design. The level of damage when they are designed properly to our current codes is uncertain and variable. The risk may differ by a factor of at least 10 between different buildings.

It's a very simple design approach based on the fundamental inelastic mode of response.Hysteretic response is represented by equivalent viscous damping, and higher mode effects are considered by new capacity design provisions. I would emphasise that New Zealand has been the world leader in developing capacity design approaches but there are some deficiencies and recent work which has been done as part of this displacement based design approach has indicated that the response can be improved and I'll talk about that a little bit later.

There are four basic components to displacement based design. The first is to represent the multi degree of freedom on a multi-level building if we're talking about a building by an equivalent single degree of freedom system where you have a characteristic force displacement response, including the inelastic response. We have relationships between displacement ductilities and equivalent viscous damping and I would mention, without going through the reasons, we know at the start of the design what the displacement ductility demand will be for a particular damage limit state. So we know that at the start of the design without knowing its strength so we can calculate what this equivalent viscous damping would be and then instead of using an acceleration response spectrum we use a displacement response spectrum but with different values for different levels of displacement.

Again mentioning that an initial stiffness approach would use this value here, the initial stiffness with five percent damping. A Secant stiffness approach that is used and it's an approach that was developed in the States by a metasosan and others in the 1970s and then largely forgotten about. We use this approximation with effective damping to characterise the response. Now since we don't know the strength at the beginning of the design we don't know what the stiffness is initially but we do know what the design displacement is and if we could find some way of estimating what the stiffness is, that would tell us what the strength would be and essentially the approach is to say with this displacement ductility known at the start of the element and the displacement also known then we know the equivalent viscous damping we can enter with the design displacement to this displacement spectra set to determine what the equivalent period would be for the appropriate level of damping and from this level of period we can calculate what the effective stiffness would be. We don't need to worry about the equations but we can calculate that at the start of the design process with confidence and that enables us to determine the correct value for the strength of the structure that we need to design without making the approximations that are inherent in force based design. Now of course it's a multi degree of freedom system and there are various aspects that have to be determined but these are pretty straightforward.

It's the displaced shape and I think the characteristic displacement, effective mass, dampings, stiffness and so forth but I don't plan to go into these at all. The start though is to look at the design displacement for a single degree of freedom systems, depends on the design limit state. We can have a limit state which says we don't want the strains to get beyond the stage which would be appropriate for the onset of repairable damage and that would correspond to a limit state, a serviceability limit state without actually putting a unique value on the ductility factor as is currently done because this is highly dependent on the geometry of the structure.

Structural displacement limit is strain related, non-structural displacement limit is drift related. You'd calculate each of these for a building structure anyway and take the most critical as the limits state to use. I don't propose to go through the way in which this is done but the calculations are very simple and straightforward.

With building structures it's normally the drift limit state that governs and that makes it even simpler. So I'll just skip through those. The critical displacements for frames normally governed by structural or non-structural drift in the beams of the lower storey, cantilever wall buildings normally governed by plastic rotations at the wall base or drift at the top storey and bridges that's normally governed by plastic rotation or drift limit of the shortest columns and these are the sort of magnitude and I'm not going to explain the equations but these are the sort of equations that we would use. They are very simple and the design process is in fact extremely simple.

We need a value for the equivalent viscous damping which is a composite of both the elastic damping and the hysteretic damping associated with damage in the structure itself, or that hysteretic damping might come from a seismic isolation unit if you're doing an isolated value and then these are typical values for the equivalent viscous damping as a function of the ductility demand for different types of structural systems. One of the advantages is that that problem that I talked about on the very first slide associated with the longitudinal response of a bridge with piers of different lengths and I pointed out that the ductility demand of those piers was different, that's easy to cope with in displacement based design as is the case here of a wall building with walls of different lengths, also different lengths will have different yield displacements and therefore different ductility demands and a rational means for determining what the equivalent viscous damping is quite straightforward with displacement based design.

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If we're talking about a bridge like the one we were talking about before with different height piers then we can determine a value of damping which takes into account the different ductility of the piers, the elastic damping associated with a superstructure and the damping associated with foundation or abutment movements as well and we can incorporate as well the foundation damping forces that might be significant.

So these can all be incorporated in a logical fashion in displacement-based design but they cannot in force-based design, the way in which we do it at the moment.

The seismic input. I don't want to spend any significant time apart from to say that we can use existing accelerations at response spectra to generate equivalent displacement response spectra. These two are exactly equivalent here and also to mention that we can generate approximate ideas as to what the spectral shape will be for earthquakes of different magnitude. This is what we call the corner period which is when we get to the maximum displacement of the response spectra itself and we also have, this is related to the magnitude of the earthquake and the maximum displacement at that point is also related to the magnitude of the earthquake and the distance to the fault plane and these show typical sorts of, ones at a distance of 10 kilometres, for earthquakes of different magnitude and you see that there are a couple of aspects here – the, for small magnitude earthquakes the, the plateau displacement occurs at a quite low period whereas for large ones it occurs at a much larger magnitude and something or other which is also immediately apparent here but which is not apparent from the acceleration response spectra is that the plateau displacement for a magnitude 6 earthquake is comparatively small whereas for a magnitude 7.5 can be extremely large if we're very close to the earthquake epicentre itself and this has some significance in terms – these are very approximate but they raise some significance when we're talking about different levels of limit because, of, of performance limit because typically if we're talking about a serviceability level of earthquake we're talking about a comparatively minor earthquake, perhaps magnitude 5.5 or 6 which occurs several times in the life of the structure and as a consequence it has a comparatively low corner period. As we get to the life safety issue we have something or other which is much more severe and using this type of approach enables us to have spectral shapes which depend on the limit state as well. Currently in our design what we do is we have one spectral shape which we factor by different amounts depending on whether we're talking about the serviceability damage control or limits state. In other words they would all have the same corner period which is incorrect and if we convert these back to acceleration spectra you can see the difference in terms of the shapes.

I think we can skip this. It's, within the displacement-based environment it's rather easy to incorporate effects of what's called velocity pulse or forward directivity effects which is not easy to do except in a very empirical fashion in current design. Another aspect which comes out of the displacement-based design is the way in which we do our structural analysis of the frames under the lateral forces and I mentioned before that one of the problems with force-based design as we currently do it is during the structural analysis when the, when we don't know what the period of the building is and we don't know what the relative ductility demands of the beams on the columns would be, the same thing occurs to an extent in displacement based design once we've determined what the base shear should be and distributed this in the form of forces to the building itself.

What, how do we do the analysis, the structural analysis? The structural analysis normally requires us to estimate the effective stiffnesses of both of the beams on the columns and then do an analysis of this. We don't know for either force-based or displacement-based design what those relative stiffnesses are at this stage. But we don't need them. In fact we can do a equilibrium-based design which is much more logical and which distributes the forces through the structure dependent on the, the characteristics that we want to have in the structure. For example, we note that the tension force under these forces down here has to be balanced by the seismic beam shears in these beams up here. So that tells us something or other about what the seismic beam shear should be and there are logical ways for distributing those, that tension force there, between these four locations. I won't go on any further. But you don't need to have any idea of what the structural stiffness is to come up with a logical distribution of strength through this element.

It also provides us with a much more logical way to combine the seismic and the gravity loads in structural design and again I won't mention that.

Finally, for these aspects torsional response, and you've heard a lot about the torsional response of different buildings and how significant this was in the response of buildings. In the elastic response of buildings the, the torsional response in current design is based on the centre of rigidity which is to do with the elastic stiffnesses of the various elements resisting the structure. Now that is, it is significant but it's not the only significant thing. In inelastic response the centre of strength and the eccentricity of the centre of strength from the centre of mass is rather more important than the eccentricity between the centre of rigidity and the centre of mass and this is not something or other which can be incorporated easily in force-based design but again is in displacement-based design.

Now I mentioned at the start that there, the capacity design in New Zealand is extremely good and has formed the basis for capacity design in many other codes overseas but there are some aspects which need to be updated if you like and in terms of direct displacement-based design there are two effects. One is drift amplification because we're designing essentially for a first mode response and we need to take into account the significance of higher modes on drift and essentially what this has done is particularly the design drift for frames higher than about 10 storeys is reduced based on time history analyses, and then moment and shear amplification for capacity protected actions and members is the second aspect of significance and these have been extensively reworked by a number of hardworking graduate students in the past to determine what the amplification due to higher-mode effect should be based on inelastic time history analysis and the results are found to depend on effective structural displacement ductility demand and one of the significance of this is that if we are considering the response of a building which is designed to, let's, as is called in New Zealand the ultimate limit state but then want to make sure that it survives in a maximum credible event we know that the ductility demand will be higher under those circumstances and the capacity design forces will also increase and I'll show an example of that very shortly. But on the basis of analyses of this sort then we've come up with different levels of amplification. For example, for column moments and frames which are rather different from what we've used in the past and depend on the ductility, the design ductility for the structure. If we look at cantilver walls and see what the significance of increasing the intensity or the ductility demand is then what we see is that at the moment the design would be using a straight line distribution of strength from the base to the top of the wall and this is satisfactory provided the ductility demand is low but if we increase to the design level of ductility, generally we don't have protection against plastic hinges forming up the wall and if we get into the maximum credible event here at twice the design level, you can see that we have a demand which is very much higher than in current codes. The same thing, this is for an eight storey and this is for a 12-storey building. More disturbingly when we look at the shear strength and the shear demand we find that the shear demand predicted by the current approaches, in particular by multi-mode analysis, are extremely non-conservative.

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A. Here is the value that would be – this dash/dot line – is the value that would be predicted by multi-modal analysis. Here is what the demand is for the design level of response. Here is what the demand will be for two times the design level of response and you can see that there is a very big disconnect between demand and capacity. Here's the, and at least what the design is, and this, again, is shown for two levels, and one wonders whether this is part of the reason why we had such bad performance in the plastic hinge regions of a number of wall buildings in the Christchurch earthquake.

Modified methods have been developed to determine what the value for the strength should be in terms of capacity design effects and compared with the values that we would get from time history analyses and the agreement is rather good. This is shown for 12, 16, and 20 storey buildings, the design, 1.5 times the design level of intensity and two times and this is the simplified method of analysis and the solid line is the results from time history analysis. When you look at shears the agreement is even better. You can see here that the agreement between the time history analysis and the shear distribution is, on the simplified method, is extremely close.

And there are values that have been developed for cases where we do not wish to go to the complexity of the approach, either inelastic time history analyses or what we call the modified modal approach, showing what the distribution of strength at the height of a cantilever wall should be and the distribution of shear should be. Essentially the difference between this line here, which is the value for the design level of response and this is the amplified value using capacity design to approach. You can see there's a significant difference in these.

Well does displacement-based design make a difference? And in determining the moment demand by elastic modal analysis is inappropriate, it's just wrong. It results in poor distribution of lateral strength based on incorrect periods. Force-based design uses the elastic stiffness which is not known at the start of the design. Direct displacement-based design uses yield displacement or drift which is known at the start of the design. Displacement-based design achieves a specific limit state at the design intensity which implies uniform structural risk from building to building. Force-based design, at best, is bounded by the limit state and vulnerability to damage is variable. The design effort with displacement-based design is less than with force-based design. It's a very simple approach and, in my mind, the simple design approach is what one should be doing, particularly for simple structures and even for complex structures you can use this as a means for determining the strength that you should have in the building and how it is distributed. If it is a complex building then the correct way of checking it is design verification using inelastic time history analysis which does capture all of the response. So in extremely important structures or complex ones I think that is the way that we should be moving and away from modal analysis.

Just something or other which, again, I won't develop in any detail but I would point out that if we take two buildings of identical geometry and with force-based design place them in, say, Auckland and in Wellington, say with a design intensity difference of two, then what we would determine with our current design is to say that the base shear in Auckland would be half of the base shear in Wellington because the intensity of the earthquake is considered to be twice as much. However, if you look at it from a damage-limit state, that is from the point of view of saying we would want the two buildings to suffer the same level of damage under the same level of earthquakes, you find that the ratio of required strength is not two to one it's four to one. So it is a complete disconnect between these two again. What we're saying is that if we know what the strength would be, the design strength for that building would be in Wellington, then to get the same level of damage under an earthquake at the same return period which has an intensity of 50% of the Wellington one, the required strength is one-quarter and what it also implies is that if you are in a very high seismic area you have to get the strength up in at a much larger rate to keep the damage down to an acceptable level.

The mathematics behind that is extremely simple to explain. Another thing that is a matter of some interest, perhaps, is that if you take a building of three storeys and then keep all of the proportions and the mass per storey the same but increase the height to six storeys the base shear demand is the same for the two.

And then, finally, in terms of bridge columns of unequal height that first element that we looked at at the start. If we're doing this from the force-based design we start off in this area here, we start, we estimate the stiffness, we know we're going to get it wrong because we don't know what the reinforcement ratio is but we would say it's proportional to the inverse of the height Q. The shear force would be in the same, the design shear force between the columns would be in the same ratio.

The moments would be in proportional to the inverse of the square of the height and the reinforcement ratios approximately in the same proportion and this is done on the assumption that doing this design will make the ductility demand of these three piers equal, which is wrong, it cannot be done. Displacement-based design recognises that the ductility demand is going to be different for each of these piers because the yield displacement is proportional to the square of the height and so it says that the ductility demand is proportional to the inverse of the height squared. We would design with equal reinforcement and have the same moment capacity at the base of each column and that would tell us that the shear force between these columns would be proportional. The difference between them would be proportional to the inverse of the height, not the inverse of the height Q and the same with the stiffness. So there's some very significant differences when you're looking at it from a philosophical point of view.

Just a couple of points in conclusion. New Zealand needs a measured and rational response to the Christchurch earthquake experience and I think a knee-jerk band aid repair to the existing force-based design approach is, in my view, inadequate. Displacement-based design codes have been developed, and Australia is in the process of implementing one for bridges. Certainly there are model codes in Europe which are being used for design and they're also being designed, displacement-based design methods are being used for design of buildings and bridges in a number of countries, including the United States, Canada, Israel, Europe, as well as New Zealand where it has been used and it can be used in New Zealand directly, provided that you use an inelastic time history analysis to verify that the drift limits are within the limits required by the New Zealand code. So you can do it but we would like to see it as a codified method as well.

A couple of things that I think also which are not on this slide which are of some importance, is that for a system such as will be described tomorrow and this afternoon – seismic isolation – and the new technologies associated with concrete or timber presss systems, essentially you have to use displacement-based design because the structural response does not fit within those which were envisaged when the force-based codes were developed. So you have to use displacement-based codes, particularly for seismic isolation systems as well. I emphasise again the non-uniform nature of damage potential for structures designed to the existing code, and there was one other point I wanted to make and I've forgotten it so I'll stop at that point. Thank you.

A.

JUSTICE COOPER:

Q. Well if you think of that you can tell us later in the day. Are you staying?

A. No, that's, I'm staying for the rest of the day, yep.

COMMISSION ADJOURNS: 1.06 PM

COMMISSION RESUMES: 2.04 PM

JUSTICE COOPER ADDRESSES MR MILLS

MR MILLS CALLS

JARG DIDIER PETTINGA (AFFIRMED)

Q. Right. Well just before you begin with your presentation just the usual steps of getting some information about you into the record. You have a Bachelor of Engineering Degree Civil from the University of Canterbury?

A. That's correct.

Q. And from January 2003 to December 2006 you studied at the European School for Advanced Studies in Earthquake Engineering in Pavia?

A. That's correct.

Q. And in that capacity you were supervised by Professor Nigel Priestley and Associate Professor Stefano Pampanin?
A. That’s correct.
Q. And there you did both a Masters Degree in Earthquake Engineering and a Doctorate in Earthquake Engineering?
A. Yes.
Q. You’ve spent the last four years or so working in British Columbia?
A. That’s correct.
Q. And in that context you’ve been practising as an earthquake engineer?
A. Yes.
Q. And during that time you’ve been specialising in performance-based seismic design methods for high-rise buildings?
A. Mainly, yes.
Q. And you're now a project engineer with Holmes Consulting Group here in Christchurch?
A. Correct.
Q. Thank you. Well I'll then leave you to just take us through your presentation.
A. Thank you.

MR PETTINGA:

A. I’d like to provide comment on the displacement-based design presentation from the perspective of a practising engineer. In particular looking at how practising engineers are going to identify with performance-based design as they know it at the moment and how they identify with the concepts behind displacement-based design. In particular I want to look at the efficiency of application of these methods. I'm going to try and draw from some personal experience in applying displacement-based design, the reasons for turning to it, the opportunities to apply it and the difficulties that we find in trying to do so, and from that try and draw out what have been effective and efficient means to applying these, these ideas in practice and from that hopefully generate some ideas towards the current direction that we're seeing things move.

So with force-based design there are some definite practical advantages. It is apparently a simple method to understand and that's largely driven by the familiarity of the method and therefore a comfort level that practising engineers feel when using it. As a method it's quickly and efficiently adopted into computer models so therefore when we have the complexities of real buildings in practise they can be easily or explicitly incorporated into the modelling process and that is a definite positive but it's also quite a negative. I know that Professor Priestley has mentioned that and indicated that we put too much reliance in the use of computer models and the numbers that come out of them to drive the designs of our buildings in current practice and I think that that's justifiably a worrying direction that things have taken. The familiarity of the method across the profession and that's not just within New Zealand but internationally as well makes it easy to communicate and this is particularly important when we're dealing with major projects that are under peer review. The ability to communicate at a, at a known level is obviously a fundamental to getting ideas across. However, there are definite disadvantages to force-based design and these have been well outlined by Professor Priestley this morning but principally the big issue with force-based design is that we don't really know what the damage of the building is going to be given a design-level earthquake. Further to this when we do go to the extent of using non-linear time history analysis then even if we use it as a verification method we may be getting results from these analyses that we can't relate back to our design assumptions. In particular the assumptions that the code drive us towards and for a design engineer that could mean that there are a number of surprises, a number of setbacks and ultimately it becomes a very time consuming process to use that type of verification.

So when we then look at this with respect to displacement-based design there are some definite practical advantages, and as outlined by Professor Priestley, the design engineer has an immediate understanding of building performance because it's implicit and, and well defined within the design method and so it forces the engineer to target levels of performance and therefore as the design is carried out there is a known damage potential given a design-level earthquake which is something that force-based design definitely doesn't have. Within the New Zealand context I believe that the engineering community here has already had some decent exposure to displacement-based design. A number of the advances in the last decade have been presented within the New Zealand publications. There have been presentations, seminars and within the code context here there is already a certain amount of terminology that is in displacement-based design that we deal with. The target parameters are a known if not used and they are accepted by virtue of the, the use of our current design codes.

The disadvantages, however, when trying to apply displacement-based design are, they largely come down to this idea of familiarity. The full methodology, while terminology is understood, the full methodology in practise is not well understood and I would say by comparison with force-based design it's an unknown, and that means that for practising engineers they find the interpretation of key assumptions quite difficult to, to kind of wrap their heads around, and therefore when dealing with new material there is a significant hurdle which engineers in practice are going to find more difficult to, to get to grips with than trying to iron out the major bumps and inconsistencies that are in force-based design especially once we recognise what they are. Therefore adapting simple and published examples for displacement-based design into complex structures that we deal with in practise that maybe have less predictable behaviour is quite a time-consuming approach and there is a tenet that we tend to work to within engineering that to get an answer one must already know the answer. And displacement-based design is a good example of that because if we don't already have a feel for what the building's going to do then it's very difficult to advance the method. And that is borne out by comparing some of the key requirements to go through a displacement-based design.

Firstly, the estimation of yield, yield behaviour of the structure or yield displacement is a parameter. It's a calculation and it's a calculation that can be done up front and there's, there's really nothing stopping an engineer doing this. That's, that's quite an easy step. The estimation, however, of the maximum displacement shape of the building, that's not so easy. Again, it's a calculation and therefore those two parameters combine for an allowance for the energy absorption and how the building is going to behave in elastic state. Now without those key pieces displacement-based design can't progress because if you can't get past a certain point then there's no going further without some major assumptions. By comparison force-based design, everything is given to you by the design codes. So there are assumptions on crack stiffness which may or may not be correct but they're a given. The assumption on energy absorption is a given by the code and therefore a design engineer can progress with force-based design quite quickly and efficiently and there's no real sort of hold-ups in terms of getting to a point of designing the building.

Now to sort of exemplify a little bit more with the potential sticking points in displacement-based design. This is an adaptation of the figure from Professor Priestley where we have some sort of structure which might be simple, as in this case, with an idealisation for the sake of the design. The key parameter in looking at the response of the building to start off with is the yield displacement. So this circle value here. And as I mentioned that's quite a, quite a definite step that we can always take. However, to get out to this ultimate displacement or peak displacement the number itself might be defined but what the building looks like at the point that it reaches this displacement is a bit of an unknown. Now for standard buildings with simple forms such as this example in the extreme it's quite an easy process and there is the formulation provided in the published material from Professor Priestley to determine that.

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A. But in the case where we have a building that doesn't look like something from the examples and something in the published material then this can be the hurdle. If we can't get a displacement profile then we can't calculate the ductility demand in the building and we can't really move beyond this sort of middle point in the design and that can be, obviously, a hurdle then that, you know, would stick up, stick the design.

So to exemplify when we might get these sorts of scenarios the code environment that we work in here in New Zealand is one that still permits a certain and significant amount of freedom in architectural and structural form, and that means we get buildings that almost always have what we call irregularities to them and that means that they're not consistent in elevation or plan. So it might be the diagram here on the left is the strength and stiffness change with the long shear wall in the bottom of the building that sets back to some slender wall higher up. This case in the middle is a very difficult one to deal with and one that we see more and more in practice where we might have a mixed use building, say with residential units in the upper floors that have a column layout that fits the unit shape, and then in the lower floors maybe an office which requires an open plan and a completely different column layout and the interface between those will be what we call a transfer beam or a transfer slab and the strength of those has nothing to do with the seismic demands. It is solely driven by gravity loads from the columns. However, they can end up being very stiff and very strong and will interact with the seismic system in a way that's not necessarily anticipated and is very difficult to grasp without doing some sort of computer analysis, and that computer analysis was normally where we'd get an answer using force-based design. Using displacement-based design I'm not sure that we necessarily have the means at the moment to deal with something like that. In plan here, this right-hand diagram, is a very simple form of a building that you might see in the CBD where you have some bounding property lines that have to have a firewall and that will be one single solid concrete wall and, therefore, is a very stiff and strong element that changes the way the building wants to move, particularly when we compare that we might have a tower extending above the lower floors, that's a smaller shape and a more simple structure, and how the bottom half of the building, which we term 'the podium', and how the tower interact can be a very difficult thing to anticipate.

To take the middle example there of this transfer scenario and demonstrate where in displacement-based design it becomes a difficulty is that without doing some sort of analysis that we would usually use in force-based design it is difficult to know whether the wall is going to bend in the top half only and be restrained at the bottom half or whether maybe the whole wall bends from the ground up to the roof and, therefore, we don't really know what the displacement profile's going to be for that so we can't calculate any of the further steps to the displacement-based design process. So this is really an identification of where the sticking points are in displacement-based design for practitioners.

So, in those sorts of circumstances, a practising engineer will naturally try to move back towards something that they are familiar with and that they know is going to be efficient to getting the job done and that is going to be force-based design.

Now, that all said, there are, in my experience, buildings that very much lend themselves to applying displacement-based design and these tend to be simple and close to regular buildings. These two examples are low-rise structures which I was involved with in Vancouver. Both of them post-disaster facilities. Both of them key to identifying the performance of the building during a major earthquake. One, this top right-hand one, was a shear wall structure. The bottom left-hand here was a steel braced frame system. Both of those were easier to get to a final answer for design than if we'd used force-based design so it was a, displacement-based was a much better option for those types of buildings.

Further to this, and this was mentioned this morning by Professor Priestley, that when we're using advanced seismic systems such as viscous stampers, base isolation, self-centering post-tension systems, the use of displacement-based design is a definite step in the right direction and makes the design much easier to develop. The displacement-based approach effectively removes some of the unknowns from the design itself for these types of high performance systems and because we're working to set performance limits in these cases it makes a much quicker design process. So this is definitely an indication of where displacement-based design becomes a benefit. My experience from working on the west coast of North America, from projects in San Diego ranging right through to Vancouver and British Colombia, was often under the pretext of performance-based design which is a design philosophy for seismic design that's been generated and certainly advanced in the last five years in California and it's a means of dealing with the code in a better way and, in fact, actually raising the level of the code and what we've found is that in working under these types of design philosophies there are key components of displacement-based design that can be adopted as additional design tools to the code's force-based design approaches and what we've found is by enhancing the code they actually quicken the design process and quicken the review process which is fundamental for the time and efficiency for an engineer.

So, just to give some background as to why performance-based design has moved through in the west coast of North America. It's largely to do with the American codes being quite prescriptive, in particular for tall buildings. There are, therefore, recommendations that have been provided through performance-based design that have been published in order to allow engineers to circumnavigate the code restrictions under a controlled environment, I guess you could say, and the aim, therefore, is to produce a more efficient and, arguably, safer building. So the process itself is intensively peer reviewed because it's no longer a code compliant design and the review team consists of a consulting firm, an academic who is an expert in the field and the city Chief Building Inspector will review it. So when faced with a design review team like that it's understandable that the immediate aim of a design engineer is going to be to try and make the peer review process as efficient and painless as possible, and the direction that one takes in that circumstance is to set up a force-based design per the code but apply these enhancements based on displacement-based design in order to provide the best option for the overall solution.

So what we end up with there is that the actual design is best driven by the code analysis but with some key check points in changes that incorporate the displacement-based fundamentals. So the basic building strength is set by a code level analysis. However, we use displacement-based concepts to estimate yield curvatures displacements and use those as a means to sizing the building and hopefully to match up the building response to the assumptions from the code. Now we also use this to update the crack stiffness values. This is a point that is very important and, as was discussed by Professor Priestley, that a lot of our code driven design at the moment is driven off incorrect assumptions for the actual modelling of the stiffness of our buildings and, therefore, we are able to use some of these pieces of displacement-based design to enhance or bring the code up to speed. In comparing the yield displacements and the maximum code allowed F displacement limits then we can get an estimation of the ductility demand on the building and that then can be reflected back to our assumed values from the code and this starts to give us a better idea of where the damage is going to be in the building, what sort of damage that's going to be.

So in application an approach that's based around this type of philosophy is quite applicable to really any type of building and what we've found is that even though we've had to go to non-linear time history analyses for verification we've not really had any surprises come from those because ahead of doing that high end analysis work we've already been doing these checks to confirm that the building response will match the output from such analysis. I do think, though, it's worth pointing out that even when using non-linear time history as a verification method the peer reviewers still lean very heavily on the code as a reference for what's expected and what's acceptable. So I really do think that if this idea of using displacement-based design plus time history verification, it may not be the way forward because peer reviewers are still going to look for something that they're familiar with. In this case it is going to be a code-based force-based design and something that they are comfortable in accepting. So I'm not sure that displacement-based design plus time history analysis is necessarily the way forward for this, particularly because there aren't many firms in New Zealand that are capable of doing time history analysis. So we start to make it very difficult for a number of companies to participate in this.

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JUSTICE COOPER:

Q. Is that, if you don't mind me interrupting you, that's a comment though presumably based on the current state of affairs, or am I missing something?

A. Yes, it's based on the current state of affairs but I guess with keeping in mind that you're looking at a generation change of engineers to be able to get up to speed to be able to do that sort of high end analysis or to be familiar enough and comfortable enough with a new design method, so it's really that we're somewhere in between.

Q. And also it's a reflection of what the current code says, but there's no reason, if it were thought appropriate, the code couldn't be developed so as to provide more explicitly for displacement based design?

A. That's correct.

WITNESS CONTINUES WITH PRESENTATION

A. So while I've shown a couple of very simple buildings as examples of using displacement based design to get to an initial design step, this adapted code base design with displacement based checks is really applicable to a range of buildings and these are three examples from California that are obviously very large buildings and also very complex buildings and were completed with I think a greater level of understanding and a greater level of protection towards building performance.

So it would appear then that we're at a point where displacement based design can be used to enhance the force-based code approaches and in the cases where this has been applied it has helped the peer review process both in early and late stages of the design, and this is largely because it provides an up-front identification of potential problems but within a code context and it allows us to deal with the very very important and quite problematic inconsistencies of the code based design. What is also meant is that when it comes to the end of the analysis phase and we've got these verification results we can actually confirm whether they make sense, the analysis itself, so it's sort of a loop cycle on that.

So I do believe that displacement based design can be adopted into our design codes and I think it is an appropriate alternative method but on the application to a restricted range of buildings and in doing so what I mean by that is that it would need to satisfy a rigorous set of regularity checks such that it is applied initially to simple structural forms and therefore we could reliably apply the published method without needing to do time history analysis for verification because if we have to turn to time history analysis then we start putting that method out of application for the vast majority of practising engineers that we have in the country at the moment. So buildings that cannot meet these types of rigorous requirements then I think we're left at the moment with force-base design as our current best option but if we're going to apply that then we do need to start developing specific displacement-based enhancements to the code approach that ensure the designer identifies likely performance of a structure, and can actually better say what the damage potential for the structure is going to be.

And so in summary from all of that displacement based design has reached a level of maturity that means it is applicable in practice. The complexities, however, of modern architecture and particular multi-use buildings can make the adaptation of the published methods for displacement based design quite difficult and quite time consuming and at the moment they would be quite a deterrent to applying them in practice. For practising engineers ultimately the major issue largely comes down to time and getting the job done efficiently and in this case force-base design and the ease of applying it in the computer analysis makes force-base design the most appealing option for most structures.

What we have found, therefore, is that experience is starting to show displacement base design is a better option for getting towards design solutions in simple structures in particular those that use supplemental damping, base isolation and self-centering building design. All of those will be developed further in the following sessions I believe and displacement based design, therefore, can be considered an acceptable alternative for this restricted range of buildings. For more complex structures force-base design I think is going to need to remain our accepted method but with this addition of enhancements that are displacement driven and force designers to start identifying and confirming their performance targets. Thank you.

COMMISSIONER FENWICK:

Q. The major problem is what stiffness do you use. Currently, of course, with force-base design if you are following the concrete code the deformation you can sustain is controlled by material strain limits, which really does bring in a sort of displacement control but I say the major problem is really what's the period of the structure, what is the stiffness of the structure that you're looking at. Now, as I see it, there is still a problem with displacement based design on the knowledge of the stiffness. At the moment we can quite easily work out the stiffness of a section before it yields but when one then has to allow for tension stiffening and one looks at the performance of a number of the buildings in Christchurch where only one crack is formed and all the bars are snapped at that level there's clearly been, whether it's force based or (inaudible 14:30:07) there's clearly been an underestimate of the effective stiffness of that building as a whole. I'd just like you to comment on that if you could please. As I see it, it's a major problem we have.

A. And that's true. I think that where we stand with our assumption that the crack stiffness, and this tends to happen in practice, the crack stiffness applies to say the full height of the building, so we have a wall and it may be 10 storeys high and we calculate a value for the reduced stiffness and we put it to the whole height of the building but I think what you're sort of getting at here is that within the plastic hinge region of the building – or what we assume is the plastic hinge region – it's very different and in fact the reductions, whether they are in the upper half of the building or plastic hinge level do not apply to the full height and I think that also starts to come down to maybe some of the mechanics behind what we have assumed with regards to steel compatibility and concrete and some of the strain behaviour. As for where we go from what's in the code at the moment with regards to strain limits which would be in some ways the ultimate displacement based design or deformation based design to getting back to what the crack stiffness limits are. I'm not actually sure. I'm not sure we're at a point yet of putting a number to that but it would suggest from what we've seen here with behaviour that the deformation zone for that reduced stiffness is a lot smaller than we would have thought.

Q. There's one other issue which you might like to comment on and I'm sorry this refers back really to Professor Priestley's presentation but he showed that if you've got a wall you get higher moments up the wall and that's certainly something I've observed in the analyses, but the second thing I've observed in the analyses is the amount of curvature you need higher up in that wall is very small and so there's sort of a problem. What should one take as the envelope because presumably one can accept what is a little bit of yielding. It's not going to significantly damage the structure and you've got the same problem with the shear. You get very high shear deformation when you start doing time history analyses and so on but again the amount of movement the shear deformation is associated with something which has sometimes got very very high affective modes and the actual displacement you get from that you could probably easily accommodate it by cracking at the concrete, diagonal cracking of the concrete or a fracture or yielding of the steel, none of which would necessarily seriously damage the structure. I just wonder if you've come across any of those problems and you could comment on them?

A. Yes I think in California in particular there has been some very sort of directional work to indicating that, well for tall buildings this was originally driven by, but I think the effects occur in shorter buildings than we might appreciate. There is good evidence to suggest that we could start to allow a minor amount of yield to occur half way up the building or two-thirds of the way up the building to get rid of those types of peaks that Professor Priestley had quite clearly demonstrated did occur and if that's the case then for the cracking that we might see from that I don't think that we need a particularly large amount of inelastic behaviour so cracks would be expected to close and I don't think there would be any residual damage from that sort of behaviour and it would certainly help out lower levels and that's what both experiment and numerical analysis has demonstrated is that just a little bit of allowance goes a long way to making the rest of the building lower down work very efficiently so I think there is a lot of merit to maybe looking at that further here as a means.

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COMMISSIONER CARTER:

Q. Just taking the case in which a designer that's proficient in both techniques and the design of the building didn't have those complexities that make displacement-based design more problematical perhaps, is there any difference in the resultant design cost from using either technique?

A. It depends on the size of the building and the form of the building. Certainly with I think concrete frames which we've used a lot here, steel frames as well. We might well see a minor increase in cost of the structure.

Q. For what?

A. For using displacement-based design and then I think you start to see not much difference as you move up into tall buildings. It's something, I haven't actually run the numbers through intentionally. But I guess where it comes back to is, what is the overall cost of the building, the structure, relative to the cost of the project and you start to see it's not a very big change and I think where we're at now in reflection of what we've seen with the performance of buildings here and the idea of life-cycle cost that increase in cost is, is relatively minimal and would certainly provide you with a building that you have a better estimation of the life-cycle cost and that might well end up being lower because you can design for levels of damage much more efficiently than we can do with the code design at the moment.

JUSTICE COOPER:
Mr Mills I thought we'd just ask Professor Priestley if he's prepared to come back and comment on those two issues that were raised. I'll just get you to acknowledge the affirmation you made earlier so we can dispense with that. Is that all right?

PROFESSOR PRIESTLEY:
(inaudible 14:36:13). I didn't hear that.

JUSTICE COOPER:
I said I'd just get you to acknowledge the affirmation that you made earlier at the outset – thank you.

PROFESSOR PRIESTLEY:
(inaudible 14:36:27) I would like to comment on all those things from Dr Fenwick.

JUSTICE COOPER:

Yes.

PROFESSOR PRIESTLEY:

The first related to analysis in general and some of the difficulties that were being suggested by Dr Pettinga. One, we certainly always suggest that above 20 storeys that a time history analysis be done but that's not an issue from Richard.

The, the first one was to do with the stiffness particularly of walls and tension stiffening and the effects on this, particularly where one gets a single crack at the base of the wall. My answer to this is that it's a problem for force-based design or displacement-based design and I think that the answer is in the fact that there should be a minimum level of reinforcement that, of flexural reinforcement for walls that's substantially larger than we've got at the moment to ensure that there is a spread of plasticity in the plastic hinge region.

The second point associated with that is the distribution of elastic cracking if you like up the wall and the, the recognition that you might well have a system where the top half of the wall for example is uncracked. If you do analyses on the significance of that and taking into account general tension shift, the potential for cracking and things you find that this is really not very significant to the estimate of the yield displacement of the wall. There is an example with some calculations in the book which, which goes through a typical example of this and you find that the, the difference in terms of, assuming that you have the same yield curvature up the height of the wall and assuming that you have it up the bottom half of the wall and uncracked above there is about 7%. If you assume the, the shape of the moment diagram is the code one but if you assume something or other which is more linear in terms of the moment with height it comes down to be less than 1 or 2% so it's really I don't think a particularly significant one unless you get situations such as did occur in Christchurch where you have very low levels of reinforcement in the walls and I'm more concerned there that you get fracture of the reinforcement than that you might underestimate the displacement, underestimate the stiffness of the wall itself.

The second was related to the peaks of moment at mid-height layer of the wall from time history analysis and the increased shear that is predicted. The first of those, I'm a little uncertain. I agree that you don't need much curvature ductility in the upper regions but there is a difference between the possibility of a plastic hinge forming at height in the wall and at the base of the wall. If it's occurring at the base of the wall the most severely, well the section with the, the greatest strain in the wall is confined by the foundation itself. It's a massive system there. If the, if a plastic hinge is forming further up the wall then you have the potential that the maximum moment region is either confined by the floor slab which is not going to be very effective or it may actually be occurring midway between floors and in such circumstances there it's not as stable. If you get a plastic hinge forming in a wall particularly if it's a rectangular wall midway between two floors then it's more likely to be unstable in terms of the potential for buckling. Having said that I certainly don't believe that curvature ductility factors in the vicinity of two or something of that sort of nature are a particular issue but generally for a very small amount of reinforcing steel you can get additional comfort out of the performance of the building for flexure.

In terms of the shear, there are two issues I think there that are of significance and the first is if we're talking about maybe a 20-storey wall building or even a bit more than that the fundamental mode is likely to be in the vicinity of two and a half maybe three seconds and the second mode which contributes about 80% of the additional shear from the higher mode effect is going to be in the vicinity of .5 to .7 seconds and I don't think that that's an instantaneous sort of level or an instantaneously fast duration where we're going to have maximum shear. If we do believe that then we should relax the shear requirements for maybe five storey and lower buildings on the same basis that the shear is only going to be there for a fraction of a second. If that fraction of a second is enough to actually release energy by cracking we may get into some second order effects which are a bit of a problem and again I feel comfortable with the, the fact that a very little amount of reinforcement provides you with additional safety.

And then the second point, the point that was really being made in that very hurried presentation I made for the higher mode effects was, at the moment we believe that capacity design gives us the necessary protection for higher mode effects. There are two reasons why this is wrong. One is that the modal analysis that we're doing, as we do it at the moment, incorrectly reduces the effects of the higher modes by the ductility factor that we use for design and quite a large number of analyses now have shown that the reduction factor should only be applied to the fundamental mode and that it is a much better approximation to only do that, and the second point that I wanted to make on this was that we understand that there is the possibility that the earthquake may be of larger intensity than the designed level of earthquake, and the point being shown there was that if that's the case then the moment at the mid-height of the wall and the shears at the base of the wall and at the top of the wall increase and there is therefore an increased probability under those circumstances that you will get unacceptable shear performance. And again, if you don't think that's the case because of the fact you have protection because they're in a higher mode. That higher mode occurs at the design level as well and so presumably you don't feel that you need to have the level of protection that we existing provide, existingly provide for shear at that sort of level which I don't think is the case. I think also there is the, if you get into the higher intensity levels the fundamental period of response increases as does the higher mode effects as well, the higher mode period will also increase. So as you get into that extreme behaviour of the wall of getting into a maximum considered earthquake rather than in the design level the significance that the higher modes may have in terms of shears would seem to increase. Again we've always taken the attitude in New Zealand that it's worthwhile being a bit conservative with our capacity design effects because for a very small amount of additional cost we can improve the safety and the comfort level of the design by a significant amount. That's my short answer. Anyway good points Richard and I'm glad you gave me the opportunity to at least put my view on those.

JUSTICE COOPER:
Thank you.

MR MILLS:
Well we're now going to turn to the issue of base isolation in rather more detail than we've heard already on this and for this Trevor Kelly is the principal witness.

MR MILLS CALLS

TREVOR EDWARD KELLY (SWORN)

Q. Again just before we get underway with your presentation I'll just get a few details for the record. Your full name is Trevor Edward Kelly?
A. That's correct.
Q. You have a Bachelor of Engineering with Honours from the University of Canterbury.
A. Yes.
Q. And also a Masters in Engineering (Structural) from the University of Canterbury.
A. Yes.
Q. You have, I see, professional registrations in California as well as New Zealand.
A. That's correct.
Q. And you were, for a period, the Vice President of Engineering for Dynamic Isolation Systems in Berkley, California.
A. That's correct.
Q. You are currently a technical director of Holmes Consulting Group.
A. Correct.
Q. And, in that role, you are involved particularly with advanced analysis and computer programming support for the company's five offices nationally.
A. That's correct.
Q. And your responsibilities include the design analysis and testing of base isolators for all types of structures.
A. Yes.
Q. On that basis then I'll leave you to run through what you want to say to us. We're running precisely to time, despite having fitted in that previous extra piece, so we're running through till 3.30. If that's not enough time we can accommodate you but if you can keep to that we'd be very please.

A. Okay, thank you. Commissioners, Ladies and Gentlemen. The topic of my talk is, as you know, base isolation. It's been mentioned several times already today by each of the speakers and so I apologise if some of what I'll be talking about will duplicate what you've already seen.

Base isolation is one of a set of strategies to reduce demand. With conventional structural design we increase the building's strength if we want to increase seismic loads. As you heard this morning there's some dispute over whether increasing a building's strength is effective at that but it's really the main tool the structural engineer has available. An alternative strategy is to reduce the seismic demand so that the forces we need to resist are lower. Instead of making a building stronger we make the forces which come into the building lesser.

The strategies we could use, there's three main categories – base isolation, in-structure damping and active control. I'm only going to be addressing the first of these, base isolation. In-structure damping has been mentioned today. There are quite a few applications worldwide of that. Active control is still more in the academic field. I'm not aware of any practical applications of that.

The concept of base isolation, as you've heard earlier, is to separate the building from the ground so the violent earthquake motions will not be transmitted into the structure. It's considered analogous to adding suspension system to the building. You're adding springs plus shock absorbers. You consider similar to an automobile you don't have any suspension system you'll have a lot of high frequency very abrupt vibrations. Once you put a suspension system in you reduce that to a more gentle rolling motion. It's also been compared to a boxer rolling with the punch so, instead of standing firm and taking the full force, you roll back and reduce the effect of that force.

If we wanted perfect base isolation we would totally separate the building from the structure so the structure would stay still and the ground would move underneath it. We'd have to achieve that total separation by, our concepts would include air gaps, sky hook, magnetic levitation or a very well oiled sliding surface. The practical problem is in that these devices don't exist in the first place, but even if they did we wouldn't be able to use perfect isolation because the building would tend to move under all sorts of external vibrations like just a bit of a wind load or traffic vibrations would set it moving. The other reason we couldn't have perfect base isolation is the building would end up in a different place from where it started after the earthquake. The ground would move away, it could be several metres different.

So in practice we have to look at practical base isolation which separates the building from the ground but instead of total separation we add devices between the structure and the foundation and these devices allow movement of the earthquakes but not wind, so they have a trigger point, they don't move under small loads but once larger loads occur then they start allowing displacements. And these devices also control the magnitude of the displacement so that we don't get movements of several metres in a very large earthquake.

When it comes to the history of base isolation the first patent in this field is over 100 years old now. In 1909 there was an English physician proposed a patent using a layer of talc, sand or mica to allow a building to slide. It's a sound concept, it does achieve base isolation. I'm not sure what the motivation of an English physician was to propose it but it was a patent of 1909. Nothing much happened for the next 50 years although there are some anecdotal applications in the early 20th century. For example, the Imperial Hotel in Tokyo survived the 1923 Great Kanto earthquake; one of the few buildings which did. It was a Frank Lloyd Wright design and he said it was designed to float on the alluvial mud as a battleship floats on water, which is a pretty good description of base isolation. Though there are some people who aren't sure whether he realised that ahead of time or it was really with hindsight once it survived the earthquake and they figured out why. But nothing much happened until about 1960 the elastomeric bearings were designed for bridges and those are rubber bearings which have layers of steel allowing them to support very heavy loads. And once those bearings were developed researchers realised that they could be used to add flexibility underneath a building or a bridge and New Zealand was a pioneer in that field and the researchers at the DSIR laboratories in Lower Hutt in the early 1970s. There are key inventions made by the late Dr Bill Robinson, in particular the lead rubber bearing. Those applications, the devices were developed and because of the system with the Ministry of Works working closely with DSIR they were applied in the field almost immediately, within a year or two. We were finding there were applications in the bridges in the early ‘70s and then by the late ‘70s the first building was isolated which Dr Sharpe showed you earlier, the William Clayton building in Wellington, which his company is currently evaluating.

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In terms of the current status, since the early 70s in that 40 year interval growth has been quite slow in New Zealand and in the US. It has been very rapid in Japan. I’ll describe that in a little more detail later. The applications, there's been applications in Europe, Asia and the Middle East, they’re in between those two markets. They’re growing faster than the New Zealand and US, again as Dr Sharpe said there's quite a few applications in Turkey and other parts of the Middle East although they’re not growing there as rapidly as in Japan. In New Zealand we’ve got about 50 isolated bridges to date and I think about a dozen isolated buildings. There may be a few more but not many. We tend to have a higher uptake for bridges because we put bearings under bridges anyway and it’s not a big change to use isolation bearings instead of conventional bearings. Buildings it’s quite a bit change for a structural engineer to actually separate his building from the ground.

JUSTICE COOPER:

Q. Why do bridges require bearings?

A. Because they move under thermal, thermal movements that the bridges contract and expand so they have bridges to take that – ah, bearings to take the movement.

MR KELLY:

A. So we've only got about an average of one building per two years and that just shows how low the market penetration is if you consider the number of buildings that have been constructed over those 40 years and only about a dozen isolated. So even though we're talking here and the supposed category of new technologies, base isolation is really classified as a mature technology. Academics are more focused on active control and other technologies and damage avoidance systems for example, so most, most advances in base isolation now are dependent on the manufacturers, suppliers and engineers to make those advances.

Because base isolation has been around for 40 years we would expect to have recorded instances of the performance in isolated structures in real earthquakes. There's no doubt that a real earthquake shows you things about a structural system which the laboratory can't show you. In Christchurch here there's only one isolated building. You saw a photo of it earlier, the Christchurch Women's Hospital, and that performed as designed and it remained operational. So in that point it can be interpreted as a demonstration of the effectiveness of seismic isolation, but in itself it can't be shown to be superior to other systems because there are a lot of 1950s and 1960s era hospital buildings which also remained operational in Christchurch as I'm sure you know. So the system works but it's not, Christchurch does not necessarily prove that it's better than other systems. If we look overseas we do find some more convincing evidence though. For example the 1994 Northridge earthquake. The USC Hospital had been base isolated on lead rubber bearings and that suffered no damage at all, whereas other buildings in the same region had irreparable damage and had to be replaced. There are other reports of successful performance from Californian earthquakes both Loma Prieta and Northridge and also from Japan but there are some negative aspects from Japan as well which I'll discuss later. Japan is a market in itself in a way because it has so many applications of isolation.

The performance of the isolated structures what we've observed, they have identified the need to ensure the movement joints are maintained as, as you see earlier we have to put a moat right around the building so that it can move several hundred millimetres at least in an earthquake. As the years go by some owners may not be aware of the purpose of that moat and they may start using it for other purposes. There is a case in California, a 911 centre, emergency control centre, was isolated quite early on and three times during small earthquakes the isolators had moved and fractured the tiles leading into the building, so the third time the tile contractor came down he thought he would fix it once and for all and he put reinforcing across the moat and reinforced the concrete heavily so when the big earthquake in 1994 occurred the building could not move in that direction and it didn't function. Again because a system had probably been in place 15 or 20 years by the time that the occupants weren't aware of the importance of that gap around the building.

The other aspect which has come out from the Californian earthquakes is that isolation doesn't work for smaller earthquakes. I mention we don't want the building to move around in wind and traffic like so you have to have a trigger point and that trigger point also helps, as you'll see later, define how much damping is in the system. The way isolation design works the bigger the earthquake you're designing for the higher the trigger point has to be, so if you're for example in near a fault in

Wellington designing for very large earthquake you would have a very high trigger point and that means in just smaller earthquakes the building would act like a non-isolated building and some occupants again aren't aware of this feature of isolation. They expect the building to protect them from every main shock and aftershocks and that won't necessarily happen.

Looking at Japan as a special case because Japan has extremely regulated seismic resistance rules for structural engineers and because of that I've extracted this quote from Japan Property Central a website which was published after the great earthquake in 2011. This discusses the three levels of seismic design in Japan of which the highest level is *Menshin* or base isolation. That's the most extensive level of seismic design. It's always used on all buildings over 20 storeys high and all other important buildings. I think there's a translation problem. Although they call it base isolation I suspect it also incorporated energy dissipation and in-structure damping because there are not many buildings over 20 storeys high which are suitable for isolation, so I think it incorporates some form of vibration control and then because it's been in place for over 30 years these rules on earthquake design there are over 2600 *Menshin* buildings now in Japan so these are buildings which the highest level of earthquake design.

And after the great 2011 Tohoku earthquake they surveyed 327 of these buildings which are in the area affected by the earthquake and the majority reported no damage but surprisingly there were 90 properties which is 28% of the total did report some kind of damage. It quotes, quoting the website, said, *"Resulting from the dampers or moving parts not functioning properly"*. The majority of the buildings with damage were in greater Tokyo and Miyagi prefecture so they certainly had to withstand quite large ground motions but nevertheless the failure of the base isolation structure in some buildings is of concern and it would be of concern to us too, so it's something that people in this field are going to have to keep a watching brief on to get details from Japan just why these systems such a high, relatively high percent did have some form of damage. Again, as I say, you can't really duplicate the effect of a real earthquake in the laboratory even if we do shake table tests as we've done you can't really duplicate a full sized earthquake on a full sized building and so we count on experiences such as these in Tokyo to refine our procedures.

Moving now to the actual technical aspects of base isolation and why it works. The most fundamental aspect is what we call the period shift. We increase the period of response of the structure, and a typical example we'd go from .5 seconds to 2.5 seconds and if you look at the top graph you'll see once we reduce, we increase that period we reduce the acceleration and the site in Christchurch from .9 g down to .3 g, so we're reducing the acceleration by a factor of three. If you look at the bottom graph that reduction in acceleration is accompanied by an increase in displacement and in this case the displacement increases from 40 millimetres up to 400 millimetres. The difference is that the 40 millimetres would be deformation in the building itself but the 400 millimetres now occurs in the base isolation devices in the base of the structure so they're not in the building itself. You can look at the period shift as what you obtained in an automobile once you put springs in. It makes the short period rapid vibration that translates it into a more gentle rolling motion which is characterised by a longer period.

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The second major factor of a base isolation system is damping. It's not essential to a system but almost all practical systems do incorporate damping. We add that to the devices to control the displacements. If we look at the top graph again, the acceleration graph, we increase the damping from five percent in an unisolated structure to something typical of isolated structure, 25%. This has two effects. It reduces the acceleration further, in this case from .3 to .2, so now we're getting a reduction from .9 to .2. But it also reduces displacements rather than increases them so now the 400's got down to 250mm which is a bit more manageable in terms of separation around the building. So the damping aspect you could look on as equivalent to the shock absorbers on your automobile. If you didn't have shock absorbers it would keep bouncing at large amplitude for a longer time. The shock absorbers reduce just how high it bounces and also bring it back to rest much quicker.

You've hear earlier some speakers have mentioned that base isolation is not suitable on all soil types. The reason for that is soft soils transmit more earthquake energy in the long period range and so isolation is less effective. If you look at a two and a half second isolated period I quoted earlier for the isolated system. If you've got a very stiff soil, like Class C, the acceleration would be .16 and displacement 250mm. If you have that same isolated period on the softer soil type in our code, which is Class E, you get both the acceleration and the displacements will more than double. So you've gone up from .16 to .4G and you've gone from 250mm to 620mm. So, in terms of a comparison with a non-isolated structure, we're getting much less benefit in reduction of acceleration but we're still happy to accept much larger displacements. So as you get into softer soil types you'll find base isolation will become less and less effective and so less and less economical.

Another effect which can determine whether isolation is suitable for a site is the 'near fault effect', which is the increase in acceleration displacements you get when you're located within about 20km of the epicentre of an earthquake. A near fault effect, or forward directivity it is sometimes called, increases the displacements and accelerations for long period structures. Our code applies it for any period greater than one and a half seconds. So almost by definition all isolated structures will be affected by near fault effects because they have a period greater than one and a half seconds and, again it has a similar effect to a soft soil. If you're near a fault you'll find both your accelerations and displacements will increase and so your benefits of isolation will be reduced.

Those considerations as to building a period shift, damping, soil type near fault can lead us to parameters which determine whether a project is suitable for base isolation or not. The first aspect is the building itself. Because we're getting benefits from shifting a period the building should have, non-isolated building should have a relatively short period, usually less than about one second. This is because the benefits of period shift reduce when your non-isolated period's long and also resonance can occur. If the isolated and non-isolated period are similar you can actually get an amplification rather than a reduction.

Another feature which relates to the building type is that isolation devices don't function well under tension loads. Most practical devices are designed to take very high compressions but only quite low tensions. So if you've got overturning moments causing tension under the columns or walls then they're not good candidates and this tends to rule out tall slender buildings. These buildings are probably already ruled out because of period anyway.

When it comes to the site it's always preferable to have firm soil conditions. The softer the soil type the less effective the isolation, larger the displacements. You have some subsoil conditions have a site period in the isolated period range. An example's Mexico City. It sits in a large alluvial basin. The seismic waves bounce across that basin with a travel time of about two and a half seconds and so they set up resonance on any building which has a period of about that range. If you put a building on isolators in Mexico City you'd likely increase the response a lot over the non-isolated one. As we'll see later there are some local aspects in Christchurch which make me suspect we could have similar situations in some places here and it's something a designer has to be aware of and take expert advice on. Another aspect which determines suitability for isolation is that you've got enough space. The compromise of isolation you get reduced forces but at the penalty of increased displacement so you have to have clearance right around the building. A typical clearance of about 250mm in a lower seismic zone, they could be a metre or more in a high seismic zone. As you can imagine a lot of central business districts which are quite congested that may rule out base isolation where your buildings are all quite close together or if you do isolate the owner may lose a significant amount of floor area by providing that clearance.

In terms of the benefits of base isolation, the key effect is it reduces the amplitude of motions transmitted from the ground into the building. This reduces the inertia loads which results in reduced forces in the structural elements and those reduced accelerations within the building reduce the contents damage and they also lower forces of non-structural components. So we reduce loads on the structural system and we reduce loading on non-structural elements of the building.

If you look at an example of what happens to the floor accelerations this is a plot from an isolated project in Wellington, the main parliament building. It was an ideal candidate for isolation because it's a very stiff building on quite a stiff site. The design basis ground motion there was .35G. Without isolation that would amplify up the height of the building to a bit over 1G. With the isolation system in place we actually get a de-amplification of those ground accelerations from .35 to .25 and then it stays quite constant up the height of the building because the building is just moving as a solid block on top of the bearings. So, in this case, we're reducing floor accelerations by a factor of over four. As I said that's an ideal candidate. That's about as good as you'll get in terms of a very stiff building on quite a stiff site.

When it comes to disadvantages of the isolation, they all really relate to the movement you have to allow between the ground and the building. You need to have clearance on all sides so the building doesn't impact on anything and that space has to be kept clear for the life of the building. Anything connecting between the ground and the building has to be able to move by that, whatever the design amount is for the isolators, up to a metre, so that includes sewer pipes, water supply, electricity, all elevators, stairs, entrances to the building all have to be able to accommodate that much movement. So that complicates design aspects for all the people dealing with those issues.

The other disadvantage for existing buildings is installation. You have to cut the building off and hold it up while you put the bearings in. If you look at a building like parliament which had really thick foundation walls, it was a couple of metres thick in places, they had to use diamond studded wire ropes sawing for days on end to cut through those walls to make spaces for the isolators so it's a huge undertaking and, as you'll see, only generally very important historic buildings warrant that level of effort.

When it comes to new building applications the first question is almost always how much will it save. Most private sector owners are focused on first cost of their building. At the stage we become involved they've heard of isolation, it reduces forces by a factor of three or four, they think that must be a cheaper building. Unfortunately when you answer how much will it save we almost always have to say nothing. The reason there's not cost savings really come down to one word you'll be familiar with this morning – 'ductility'.

The reality of seismic design is that structural engineers use ductility to reduce forces by factors of up to six. That's typically higher than the factor we're reducing them by for isolation, which is up to about four.

1514

Now if we could use ductility of six times the factor of four then obviously we'd get to a huge reduction in design forces. Unfortunately they're not multiplied together. Isolated structures are not permitted to use the same ductility as non-isolated structures. The main reason for that is because if you allow ductility the building period will get longer you may get resonance and amplification so there are severe restrictions on the amount of ductility you can use in a non-isolated building. The end result of that is that a new ductile building will have similar design forces whether it's isolated or not isolated. There will be some savings because you won't have to have such stringent ductile detail in the amount of secondary reinforcement, confinement, etcetera, but those savings generally are enough to counter the cost of installing the isolation system.

Which leads to the next question: If you're not saving anything why use it? The reason now is important to people here – increased safety. We're reducing the accelerations and forces and so we're greatly reducing the life safety hazard but the prime motivation for base isolation is to increase safety by reducing the accelerations getting into the building. We also get an increased performance. The fact is ductility is another word for damage. A structural component which has gone through a ductility has lost some of its capacity so that's the definition of damage. That damage is permanent and, as you've heard here, the damage may not be repairable. The building may not be repairable.

There are a lot of existing buildings which don't have any available ductility. For example, historic buildings. Again as we've seen in Christchurch once the loads exceed their elastic limit the buildings are susceptible to collapse. Another reason to use isolation, apart from the structural aspect, is that we can protect the contents. We're reducing the floor accelerations. We're reducing the inter-storey drifts and so we reduce the damage to building parts and contents and going along with that is continuous functionality. The absence of structural damage and non-structural damage permits the building to function during and immediately after the earthquake event.

And so those reasons lead to a best candidate set of types of buildings. You'll find not all buildings are suitable for isolation and we tend to focus the solution on buildings where continued function is important, ductility is not available or buildings with very important contents and in applications world-wide they tend to fall into one of five main categories – the central facilities which are Wellington Police Station, 911 centres, emergency operations, civil defence type centres where you want continued functionality and the code prescribes this with a high return factor. Other facilities are health care facilities – hospitals and medical centres which we require to be accessible straight after the earthquake. Again we want continuing functionality. The category of historic older buildings, the motivation there is preservation. They have low ductility and so the reduction in forces from isolation translates directly into a lower design force. We have museums – buildings which had very valuable contents and in that case motivation might be more to reduce the floor acceleration than to prevent damage to the structural system. In some types of manufacturing plants, less common in New Zealand, but in California some semi-conductor chip manufacturing plants have contents which are worth many times the value of the building and they will spend money isolating the building just to protect those contents.

Running through a few examples in New Zealand and overseas you'll see most of them come into one of these categories – the Museum of New Zealand, Te Papa Tongarewa, is constructed on a raft which sits on lead rubber bearings and Teflon sliding bearings. There's a visitor gallery there you may have been down which allows access to view one of the isolators and a side effect of putting it on lead rubber bearings was that the raft was built as one big concrete slab with no shrinkage joints and so the base isolators have all moved in towards the centre of the building by about 25mm over the 20 odd years since they've been installed so they are taking up shrinkage movement as well as the earthquake movement.

As a side point that architect of Te Papa was very aware of seismic issues and the whole building layout there is determined to mimic a tectonic plate layout. You can see the big inclined wall which is intended to represent the Wellington fault and the offsets on either side of the west wing in the main building.

The New Zealand Parliament buildings. They were base isolated in the early 1990s. The Parliament, the main building itself, plus the two general assembly library buildings have been isolated. The whole building was cut off, supported while lead rubber bearings and high damping rubber bearings were installed. I think about 300 bearings in the main building. It was a huge project because even with base isolation the main building was still too fragile to resist earthquake loads and so concrete walls were added as well. You've seen earlier some other historic buildings in Wellington which have been isolated. They include the old BNZ Arcade, the Maritime Museum, and the Supreme Court building.

Health care facilities, as Dr Sharpe discussed earlier. There's several health care facilities in Wellington which are base isolated. Another example in California is St John's Hospital in Santa Monica, isolated on lead rubber bearings. This is a replacement facility for a hospital there which was irreparably damaged in the 1994 Northridge earthquake. Since that time most health care facilities in California are isolated.

Apart from vulnerable buildings there are also vulnerable bridges which are being retrofitted with base isolation. This example is the Benicia-Martinez Bridge which is one of the San Francisco Bay crossings and they are base isolating the superstructure to reduce earthquake vulnerability and ensure functionality.

Perhaps an unusual case of a valuable contents is the Missouri Botanical Garden. This is a herbarium which contains a collection of some of the world's rarest plants and books about plants and the owner was very concerned about protecting the contents so even though

Missouri has very infrequent earthquakes, historically it has had some very large earthquakes on the New Madrid fault system and so the owner spent the extra money to have the building isolated. It's on high damping rubber bearings.

When it comes to historic buildings this one shown here is Berkeley Civic Centre. There's a whole series of municipal buildings in California which are historic but very fragile. Typically unreinforced construction, have a steel frame built in the early 1900s, so base isolation has been implemented on the Berkeley Civic Centre, the Oakland City Hall, San Francisco City Hall, LA and Haywood City Hall, so there's a lot of major projects there retrofitting them to existing buildings. They've almost always needed some structural strengthening in addition to isolation so they're expensive projects.

1524

Historical note, the Oakland City Hall at 24 storeys was the tallest city hall in California when it was constructed. Then it was trumped by the Los Angeles City Hall which is 34 storeys high. In the early 1990s Oakland base isolated their city hall making it the tallest base isolated building in California. Then two years later Los Angeles base isolated their city hall so they now hold the record of the tallest base isolated building in California. The LA City Hall is very unusual in that it's over 30 storeys tall which generally would rule it out because of its long period but it's a huge building. It extends over a full city block with a very large podium, very thick unreinforced masonry walls. So it's very stiff for its height and the base isolators it's on are very large and take a very large displacement. So it's isolated out to about three and a half seconds and can move a metre or more in a major event.

You've seen this example in Auckland, Union House so I won't go through that again. It's used by an earlier speaker today. It's isolated on sleeve piles, and another example, there's been a lot of interest in base isolation which again Richard Sharpe mentioned in Turkey because of the major earthquake events they've had over there. This is an existing viaduct being retrofitted using lead rubber bearings. These bearings are supplied by Robinson Seismic in Lower Hutt.

It comes down to the requirements of a practical system. Any base isolation system has to provide flexibility. That's the basis of the concept. It needs damping which is not essential but is preferred to reduce displacements and also sensitivity to different types of earthquake. And it also required that the system be rigid under service loads so the occupants aren't alarmed by frequent movements and wind storms and under traffic movements.

Inventors around the world have come up with probably seven dozen types of devices which have been promoted for isolation but not many of them have so far made it into practical implementation. There's a lot of legal issues, code compliance issues which make it quite expensive to bring a new device through into use. The most common type in New Zealand is the lead rubber bearing which was invented by the late Dr Bill Robinson in the early 1970s at DSIR. That's layers of rubber and steel with a lead core forced into a hole down the centre. Examples of that at Te Papa, Parliament Buildings, the Maritime Museum, Petone Press Hall There's quite a lot of applications in New Zealand.

High damping rubber bearings. They look similar to a lead rubber bearing externally but they don't have a lead core. They have a special rubber compound to provide the damping. Parliament has a mix of high damping rubber and lead rubber.

Another type quite commonly used are flat sliding bearings, usually Teflon sliding on stainless steel. Because they're flat they tend to just move in one direction and if they don't have anything to return into initial position they'll tend to offset the building after an earthquake and tend to keep moving in aftershocks. So you have to have some other device in parallel with them. Like Te Papa for example has lead rubber bearings plus sliders so the lead rubber bearings bring it back to its starting position.

An improvement on flat sliding bearings are what are called inverted pendulums. The Teflon sliding bearing instead of being on a flat plate it sits inside a spherical dish. So as it moves the whole structure lifts up and gravity tends to bring it back to its original starting position after the earthquake. So it's a sliding bearing which doesn't need other devices in parallel. It's used quite extensively in the US but not so much in New Zealand. I think mainly because of cost and patent issues.

Other types such as sleeve piles, lead extruders, yielded steel cantilevers are supplementary devices. They're not used as often as the lead rubber and sliding type bearings.

This diagram doesn't seem to have come out quite right. Lead rubber bearing. I think you saw one earlier today anyway – a sketch. They may be square of circular. A typical dimension's about between 350 millimetre in plan size up to about 1200 millimetres. They can be 200 to about 800 millimetres high. So they can be very substantial items. The bigger ones would weigh a couple of tonne and the lead core can be anything from about 75 mm up to 300 mm diameter. So a very large diameter core. The lead yields plastically and absorbs energy under earthquake. It converts the energy to heat. It heats perhaps one degree per cycle. So intense cycles at an earthquake, it doesn't get anywhere near the melting point of lead which a lot of people ask. The lead core is press fitted after, after you've moulded the bearing. We make the building, the bearing in a mould. You apply heat and pressure for sometimes a day or more for a big bearing and once it's cooled down you squeeze the lead core into the middle so it's jammed tight. The high damping rubber bearing looks similar except no core.

JUSTICE COOPER:

Q. This drawing. Are you able to get us what you really meant, give us there?
A. No it doesn't seem to have reproduced here. It certainly did on my computer at home.
Q. Yes.
A. I'm not sure what it looks like on the printed copy.
Q. Well we've got something that looks like you're really preserving the intellectual property of the contents.
A. That was not the intent. I can perhaps provide it in another format later for archival reasons.

COMMISSION ADJOURNS: 3.31 PM

COMMISSION RESUMES: 3.47 PM

MR KELLY:

Your Honour, Commissioners, I've obviously mis-timed the length of time it would take for my presentation.

JUSTICE COOPER ADDRESSES MESSRS KELLY AND MILLS – DISCUSSION REGARDING TIME AVAILABLE TO MR KELLY

MR KELLY:

Okay. Well as I said I'll move quickly through the device types, the flat sliding bearing, the friction pendulum bearing as I said is a slider inside a dish. When it comes to installation typically we have to cut through in the basement level. We have the bearing there. We have to often build a lot of new structure above and below the bearing to take all the forces arising from the isolators. As with all new technologies there are a lot of impediments which have stopped widespread adoption. Base isolation, there's a list of things. Probably the most important are aspects of codes. Engineers feel most comfortable when they're designing according to codes. In particular design ground motion, types of analysis and building ductility. Other implementation issues as listed there. It's surprising with codes given that these devices were invented in New Zealand. We don't have an isolation code. Structural codes don't prohibit isolation but they don't encourage it either. They don't give any rules for use. So we've commonly been adopting versions of American and European codes which is not really satisfactory because there are some fundamental differences in practice between New Zealand and those markets. Probably the most important aspect where we'd look for guidance from our codes is seismicity. With base isolation we rely on an decrease in acceleration as we increase the period. As I mentioned earlier some soil types and near fault events don't exhibit that characteristic. Unfortunately some of the stations from the recent Christchurch events show a lot of energy concentrated at longer periods. Ironically these records are close to the only isolated building in Christchurch, the hospital. And if you see on the top they show a moderately strong acceleration peak at about two and a half seconds but because that's at quite a large period it translates into a very high displacement peak as you see on the lower two graphs. So in this case we've got about a thousand millimetres movement under these recent events. In fact if, if you isolate when you have that type of spectra you might be moving into harm's way instead of out of harm's way. You're moving into a higher energy regime rather than vice versa. I'm not sure whether that's a function of the site itself or of the particular event and we will need some seismological input before we take isolation much further in Christchurch I imagine.

I'll skip through the details of foundations but the fact is that isolators provide point lines and there are quite a lot of structural engineering aspects to foundation design which don't exist in a non-isolated structure.

Installation. I mentioned earlier on Parliament buildings just how complex that can be. In a way you have to consider that, as Frank Lloyd Wright said, that the building is now like a ship at sea. So all the entrances must be like gangways. You have to be able to cross the moat to get into the building.

And bridge engineers are accustomed to using bearings but building structural engineers specialising in buildings typically aren't. They're not conversant with how to select the device and how to design it. Because of that they tend to have to rely on suppliers to give technical advice and assistance. Naturally each supplier will promote the virtues of their own devices and so often there'll be conflicting advice and it may be difficult to resolve those conflicts.

Isolator testing adds quite an expense because we have to test full size isolators for every project. Because of the test sequence they go through they can't be used in the structure so they're an added cost and there's more continuing testing cost because most production bearings are also tested at lower levels of load.

And then of course as in all projects there are cost factors. They're going to be discussed in more detail later but the big isolators for a large project typically cost about as much as a small car. About $10,000 to $20,000 each. So a building like Christchurch Women's, I think the hardware costs were a little under a million dollars for a building that size. Often the hardware costs are not the highest cost increase. There'll be costs associated with architectural changes, utility costs, there's a lot more engineering involved and often you need a suspended floor above the isolators which may not be required otherwise.

In my final slide, based on all those considerations my impression of the future of isolation. I think it will be continued to be used for specific types of project. They'll be the same ones up-to-date which are usually publicly funded buildings of high importance or a fragile historic nature. A feature of isolation is that we tend to trade off an increased first cost for a decreased lifetime cost. As you've seen some of the slides earlier today, if you invest a certain amount of retrofit you get a payback over 20 to 50 years. But of course for a lot of private sector building developers they are not really interested in a 20 to 40 year cost because they don't intend to be an owner over that period of time. So really the clients who are most likely to use isolators will be people who tend to be long-term owners of the building and that's why it tends to be health care facilities, motorway bridges, museums and historic public buildings. It's possible the use might extend to other types of building as impediments are removed but I don't see much evidence of that. We've had a fairly steady uptake over the last 20 years in New Zealand and there's no indication that will change although of course it could happen as a result of the events in Christchurch. Implementation needs a high degree of co-ordination between all parties from the owner to all the professionals involved and the fact is any one of those can derail it. If they're not enthusiastic then it's quite easy to come up with reasons not to go ahead with isolation and that has occurred on quite a number of projects.

Thank you. I'm sorry for going over time.

JUSTICE COOPER:

No not at all, thank you Mr Kelly. It's been most interesting. Thank you very much. We'll see you later.

WITNESS EXCUSED

MR MILLS:

Right we've now got a final session for the day before we turn to the panel which is going to look specifically at the question of costs and we've got two speakers on this. Initially we've got Megan Devine from Robinson Seismic and she'll be followed by Grant Wilkinson from Ruamoko Solutions. So I'll call now Megan Devine.

1557

MR MILLS CALLS

MEGAN LEIGH DEVINE (SWORN)

Q. Well as with other witnesses just a few details before you run us through your presentation. Your full name is Megan Leigh Devine?

A. Yes, correct.

Q. You're the General Manager of Robinson Seismic Limited.

A. Yes.

Q. And you've worked for that company ever since it was first incorporated by Dr Robinson.

A. Yes I have.

Q. You have a degree in business management.

A. Mhm.

Q. You're a past Associate of the Bankers Institute of New Zealand.

A. Correct.

Q. Your background prior to your current role was in international finance and management.

A. That's right.

Q. Thank you, I'll leave you then to just go through your presentation.

A. Thank you very much. Commissioners, Ladies and Gentlemen. I thank you for the opportunity to address the Commission today and the wider public on the subject of seismic or base isolation. As you've heard this is a technology that was developed by Dr Bill Robinson here in New Zealand in the late 1970s with his invention of the lead rubber bearing.

So it's not exactly a new technology but it remains by far the most widely used of earthquake protection devices worldwide. Unfortunately, also as you've heard so far, it's uptake in New Zealand has been quite slow, especially outside of Wellington but overseas we're seeing its use increasing at an exponential rate. As a matter of interest Dr Robinson retired to Christchurch two years ago and so, of course, he experienced the earthquake events of September 2010 and February 2011. He passed away here in Christchurch last August but right up to his death he remained adamant that base isolation must be widely used in the rebuild of Christchurch. In fact he said probably about two weeks before his death, he said it is crucial that devices such as lead rubber bearings are used in the reconstruction of the Christchurch CBD so that Cantabrians can go to their places of work and study with the confidence that they are as safe as possible. Base isolation is the technology that can help provide that greater level of safety and piece of mind.

So by way of introduction Robinson Seismic Limited is based in Wellington, out in Lower Hutt and we're the only New Zealand developer and supplier of base isolation devices. We manufacture in Malaysia and we're active in Asia, India, Turkey and the Middle East. In many of these countries base isolation is now considered mandatory for earthquake protection. So we've been directly involved in new projects in New Zealand such as the Wellington Hospital, the North Shore Hospital, the Whanganui Hospital, and in retrofit projects such as the Supreme Court in Wellington, Victoria University Library and we were indirectly involved in Te Papa and Parliament Buildings.

Mr Kelly has covered the technical aspects of base isolation which is just as well because I'm not an engineer. I operate the business side of our company so I am going to address you today on the business case for seismic isolation and specifically the all important question of cost. But I am interpreting this as a cost benefit because I don't believe that you can look at the costs without also considering the benefits of seismic isolation.

So we have limited the discussion to new structures. Base isolation can, of course, be retrofitted to suitable existing structures, as you've heard, but the variables of existing structures make it really difficult to give you a meaningful indication of cost, and whenever I refer to buildings or structures base isolation can, of course, include bridges, storage containers, tanks, power facilities and most other structures requiring protection from the damaging forces of earthquakes and of vibration in general.

I guess it does concern me that there is a widely held misconception that seismic isolation is very expensive. Of course there is a cost but viewed against the savings it can, in some cases, result in a slightly lower construction cost overall. The Union House, for example, which I think has been mentioned today, was built in 1983 with base isolation. It produced an estimated 7% cost saving in the total construction cost of 6.6 million dollars which included a construction time saving of three months due to the structural form requiring less seismic force ductility demands and structural deformations. However, a general rule is that the inclusion of all aspects of seismic isolation in a new structure will add no more than 3% to the total construction cost and considerably less when assessed against the benefits of such a system. Here I do want to acknowledge the work of Associate Professor Andrew Charleson who, I think, is appearing this week and Nabil Alif, apologies if I have mispronounced his name, they are from the Faculty of Architecture and Design at Victoria University and they've prepared a paper on this exact subject which will be presented at the New Zealand Society for Earthquake Engineering Conference here in Christchurch next month. They've looked specifically at four recent new buildings in New Zealand built with seismic isolation. I can talk to you in depth on two of the projects because we were directly involved but Andrew has consented to me using his broad data.

So this is the Wellington Regional Hospital. It was completed in December 2008 and it was fitted with 135 lead rubber bearings and 132 slide bearings. It's a seven storey building, a total floor area of 44,700 square metres. It is designed to withstand a magnitude 7.8 earthquake on the Wellington-Hutt fault and a magnitude 8.3 on the Wairarapa fault. The total construction cost was $165m. The cost of just the seismic isolation bearings was 1%, just under 1% of total construction cost. The cost of all the components of the base isolation system, which includes the installation, the seismic gap or moat, as you've heard, and the basement was around 3% of the total construction cost which translated to approximately $110 per square metre.

You'll understand that by its very definition base isolation is most commonly underneath the building so, therefore, a crawl space is necessary. The architects of the hospital have used the base isolation installation to their advantage by actually turning the crawl space into a basement carpark. The lead rubber bearings have been fitted to the top of the pillars to maximise floor area for parking. So that was an architect's thing to do. You can see the base isolators indicated on the picture. While the basement added slightly to the initial construction cost the DHB now has a large carparking area through which they can now recoup some costs in carparking charges. I rather suspect that they have more than recouped those costs at the level of their charges.

This is the Whanganui Hospital Peri-Operative Block which was completed in 2008. This was two single storey buildings which were the first in the world to be base isolated with a newly patented device called the RoGlider. The RoGlider was developed with funding assistance from what is now called the New Zealand Ministry of Science and Innovation. It is designed specifically for smaller lighter buildings so where the load is between 5–80 or 100 tonnes per column. They can still accommodate large displacements, so they are as effective in earthquakes but they now give us the ability to seismically isolate lighter smaller buildings. Due to the sloping ground on this particular site the RoGliders were installed either on typical columns, which you can see on the right, or directly on the ground, the one indicated to the left. So part of the created basement houses the maintenance services of the building and part is unused crawl space over the ground.

1607

As the total construction cost for this was $18m and the cost of the devices on their own was two percent of that. The total cost of the base isolation system incorporating all factors was three percent of total construction cost or approximately $140 per square metre. These devices were literally hot off the R and D table when they were installed so manufacturing costs have been refined significantly, so the cost would now be proportionally less.

Of the two other buildings assessed by Andrew Charleson, the total cost of the base isolation systems, incorporating all factors – so installation, seismic gap, etcetera – were all three percent or less of the total construction cost although I do offer the qualification that many documents on the construction of Christchurch Women's Hospital have been lost so we can only be certain that the cost of just the lead rubber bearings themselves was 1.3 percent of the total construction cost.

So far I've mentioned the direct costs but it's important in any analysis to also note the savings of seismically isolating structures. For example base isolation allows for a reduction in structural elements of the building with less ductile detailing needed and is demonstrated in Wellington Hospital crawl spaces or basements can have multiple benefits. For example in the siting of services the additional income from a carpark and also flexibility for future development. One of the main points that I want to emphasise here and was mentioned by Mr Kelly is the advantages in the protection of the contents of the building.

With the controlled movement of the building caused by seismic isolators, contents are not subject to violent and sudden shakes so it significantly reduces the impact on the contents. Aligned with this is the protection of the integrity of the internal structures of the building so internal stairs, internal walls, partitions, etcetera. So the building is safer for occupants. The contents are protected and, as a result, continuity of operations is much more likely. Often the value of contents represents a significant percentage of the total value of a building and the protection of contents can impact hugely on business continuity. So where we consider particularly things like computer main frames, hospital diagnostic equipment, etcetera. We had one nurse on duty at Christchurch Women's Hospital during the 4th of September 2010 event and she told us that they barely lost bandages off the shelves although any beds where the wheels weren't locked glided gently across the floor.

Could I have the video please. So this is a simulation of how base isolation works. So you can see in that video simulation that the building is subjected to much less destructive force so it's easy to imagine that what's inside – the contents, the internal stairs and partitions etcetera – are not going to be thrown about or damaged or disengaged from the external structure.

I want to just touch briefly on the topical issue of insurance and I would really like to be able to tell you that there are significant premium reductions offered to seismically isolated structures in New Zealand but we do need to commence discussion and education of the insurance industry before we can claim, for example, the 30 percent discount in insurance premiums that apartment owners in Japan of base isolated apartments can claim but we will be entering those discussions very very soon. Initial approaches to insurers in New Zealand reveal only an increased interest in insuring buildings which are seismically isolated. It is likely that in the future seismic isolation of a building will be an increasingly important factor in the acceptance of building insurance, contents insurance and business interruption insurance. Incidentally this issue is also currently being addressed with insurers in the USA based on the fact that insurers routinely offer premium discounts for protection measures such as fire resistant construction and fire alarms, theft alarms so it's generally been accept there in the United States that a seismically isolated building will remain fully functional so that eliminates losses caused by down-time, loss production, lost data and lost building contents.

I am also aware of at least one municipal authority in New Zealand considering self-insurance of its municipal buildings through the retrofit of a seismic isolation system. For new buildings, however, of course, this has implications on the finance industry through mortgage requirements etcetera, so this is another industry sector which we will be initiating discussions.

Next, considering the issue of maintenance. Contrary to what some believe, seismic isolation devices require no maintenance during the life of the building. Following any significant earthquake event they should be inspected to ensure bolts and load plates are still in place but they devices themselves don't need replacing after an earthquake unless the event was significantly in excess of their design specification in which case even then we wouldn't take the isolators out. We would recommend removal of selected isolators for testing. So maintenance costs don't need to be considered at all and it's worth pointing out that because the building is protected from major damage, repair costs following an earthquake will be lower to non-existent.

So of course we acknowledge that there are other methods of protecting the integrity of external structures from the damaging forces of earthquakes so we can ensure safe egress for building occupiers according to code but we remain convinced of the effectiveness only of seismic isolation and the protection not only of the external structure but also the contents and also in maximising the potential for immediate business continuation. Traditional earthquake strengthening methods can also detract from the aesthetics of a building so this of course becomes an issue for heritage and historic buildings and many heritage buildings are appropriate for base isolation as you have heard today and they could be retrofitted to achieve earthquake protection without compromising the aesthetic integrity of the building.

This is the Supreme Court building in Wellington which was originally built in 1881. It was restored for its new purpose in 2007, including earthquake protection through the retrofit of lead rubber bearings. This building was literally cut off its foundations at ground level and the floor removed and excavated down so the bearings could be installed so since the main strengthening work was done at its foundations there was no need for intrusive strengthening methods to take away from the beauty of the exterior or interior walls and ceilings.

JUSTICE COOPER

Q. What about the new building?

A. That’s on the right of the building.

Q. On the Quay?

A. That's right, and that was not seismically isolated. The old building was particularly suitable for seismic isolation and the two buildings are joined together with a walkway which is designed to break away in the event of an earthquake so that the buildings will move independently.

1617

Q. Why wasn't the new building built in that way do you know?

A. I, I don’t know why.

Finally I think it’s worth mentioning the economic and the social benefits. If we can ensure uninterrupted functionality of buildings and places of work. For many people their continued employment is secure and they will be able to stay and assist in the rebuild and recovery of their community.

So in summary therefore analysis of recent new buildings in New Zealand fitted with seismic isolation during construction indicates a total gross cost of that seismic isolation being around 3% or less of total construction cost, but the cost should also be considered against cost savings. We've got a reduction in structural elements savings and contents replacement, reduced to non-existent repair costs, the benefits of maximising the potential for immediate continuation of business, although of course this can be dependent on external factors such as services infrastructure. Additional, the likelihood of insurance and deductibles benefits can't be discounted and intangible considerations such as economic and social benefits.

So just making brief reference to the paper 'Design of Conventional Structural Systems following the Canterbury earthquakes' prepared by the Structural Engineering Society of New Zealand for submission to this Commission in December 2011 it notes that *"although most buildings in Christchurch have achieved the primary objective of saving lives levels of damage have been high."* Their recommendation, *"Unless a building contains high value and/or sensitive equipment designers are advised to use stiff lateral load resisting systems such as walls or braced frames but if a building contains high value or critical contents consideration could be given to using other methods of protection such as base isolation".*

I would submit to the Commission that given the cost benefit analysis of base isolation this is a method which should be considered for all appropriate structures in the rebuild of Christchurch. We talk about building to current code meaning a life-safe building but not a damage-free building. Seismic isolation can increase the performance expectations of structures to be both life-safe and to minimise damage with the advantage of significantly reduced seeing the time required for a city to recover and resume normality. So while, of course, it's vital that the importance level IL4 buildings are seismically isolated so that they remain fully functional both during and immediately following earthquakes, the benefits of seismic isolation for other buildings can't be underestimated either and neither should the value of peace of mind be. I believe Canterbury is in a unique position. It's got to rebuild a city and it should aim to rebuild that to the highest levels of safety possible. Thank you.

JUSTICE COOPER:
Thank you.

QUESTIONS FROM COMMISSIONERS CARTER AND FENWICK - NIL

WITNESS EXCUSED

MR MILLS CALLS:

GRANT WILKINSON (AFFIRMED)

Q. Well again just some preliminaries, your full name is Grant Wilkinson?
A. Correct.
Q. And you're the managing director of Ruamoko Solutions Limited?
A. Correct.
Q. You hold a Bachelor of Engineering degree with honours, Civil?
A. Yes.
Q. And you have been involved in the structural design for the base isolated Christchurch Women's Hospital?
A. Yes.
Q. And also similar work for the proposed rebuild of St Elmo's Court office building in Christchurch?
A. Correct.
Q. And you were the structural director of the detailed specifications stage for the work that was done on Parliament Buildings?
A. Correct.
Q. With that background I'll leave you to take us through your presentation, thank you.

WITNESS REFERS TO POWER POINT PRESENTATION

A. Well good afternoon Commissioners, ladies and gentlemen, and thank you for that introduction and the opportunity to address the Royal Commission on base isolation of building structures.
Trevor Kelly has presented in detail on the technical and analytical aspects of base isolation and Megan Devine has presented on issues of cost and cost effectiveness of base isolation for buildings. My presentation briefly examines two recent examples of base isolations of buildings in Christchurch. I was and I am the project design director of both of these buildings.
While I'll cover aspects relating to the cost of base isolation I'd also like to share with you some points relating to the decisions to base isolate

those buildings. The first building is Christchurch Women's Hospital. Built in 2003 and 2004, and the second building is the rebuild of the St Elmo's office building in central Christchurch with construction due to start in May this year.

JUSTICE COOPER:

Q. What street's that in again sorry? What street's that St Elmo's building in?

A. It's on the corner of Montreal Street and Hereford Street. It was one of the first, one of the first major buildings to be demolished in the city.

WITNESS CONTINUES POWER POINT PRESENTATION

A. So there we have a picture of Christchurch Women's Hospital. It is quite a large building and a total of nine storeys high. A lot of people will be familiar with that driving past it in Hagley Park. Right, the Christchurch Women's Hospital is the first and currently the only base isolated building in the South Island. It has a total floor area of approximately 20,000 square metres and it's spread over nine levels. That floor area is similar to the Clarendon Towers building and similar to the Price Waterhouse buildings in Christchurch. It has a floor plate of approximately 76 metres by 32 which is pretty large and a height of 33 metres. The construction cost was approximately $60 million and it was opened in March 2005. The CDHB, the owner, considered international best practice before selecting base isolation for this building. It was part of our brief. We researched international best practice for hospital buildings in particular by exploring practice in California where they have an active programme for upgrading their hospital building stock and in particular by building new critical and emergency hospital buildings with base isolation. In selecting base isolation for Christchurch Women's Hospital our client CDHB were mindful of the resilience or perhaps rather the lack of resilience of some of their buildings housing theatres and critical care wards and the like. While Christchurch Women's wasn't intended to function, to normally function as an emergency or a critical care facility, they opted for the added seismic security of base isolation as they would be able to rely on the theatres in the critical facilities recovery and critical care wards in the new base isolated building being available during or after a major earthquake.

The superstructure of the Christchurch Women's Hospital has a relatively conventional reinforced concrete perimeter frame, plus it has two low height eccentrically braced steel frames to provide added stiffness and strength.

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It has 41 lead rubber bearings and the bearing size is approximately 830 millimetres by about 275 millimetres high. The bearings are square. It also has four pot bearings, those pot bearings being under the steel, under the eccentrically braced steel frames and several Teflon sliding bearings which are underneath the main entrance where the, where the building is much lower height and the loads are much lower. The approximate cost for the bearing supply was $800,000 in March 2003 making them approximately $18,000 each. That is the supply only of the hardware. And just confirming Megan's figure of 1.3%, yes, the supply of the base isolators was about 1.3% of the total construction cost. The isolators for Christchurch Women's were designed for plus or minus 420 millimetres of movement in a maximum credible earthquake with a expected return period of 2500 years. That return period was calculated before the seismologists have recalculated return periods for earthquakes. I personally observed approximately plus or minus 40 millimetres of movement across the isolation plane based on the scratch marks on the, on the retaining plates, on the gravel spaced plates in the 4th of September 2010 event and the isolators after that event had about a 25 millimetre lean in one direction and after the February 22nd event I observed approximately plus or minus 120 millimetres movement across the isolation plane and the isolators had corrected and were pretty much straight up and down. While I haven't inspected the building in detail after the earthquakes by all other accounts I've heard it had performed really well and as intended.

Q. Those measurements are, you're observing a kind of, just a signature left on the plates, it's not –

A. On the plates.

Q. It's not a sort of inbuilt measurement system. It's –

A. Correct. It had a series of, of plates that flip up in the, in the earthquake. They're quite heavy.

Q. Yes.

A. Because the ambulances drive over them.

Q. Yes.

A. And you could see, if you look carefully you could see the scratch marks where the plates had moved backwards and forwards. The 100, plus or minus 120 millimetres was only looking at it in terms of movement in one direction only.

Q. Yes.

A. It's not possible to actually look at it in other directions because it was of a different design of the, of the cover plates, the isolation plates.

Q. So you, they would mask the movement or what are you saying?

A. The other ones were more, the plates that I observed them on were these ones which were designed to accommodate vehicle movement across them. They're heavy plates and elsewhere around the hospital had light-weight steel plates that just moved out of position or at worst would buckle and fail. They didn't really, they wouldn't really scratch the surface that they're sliding over.

Q. And why do you think there was this correction after the February earthquake?

A. Why do I sorry?

Q. Why was there a correction after the 22nd of February earthquake?

A. I don't know. It's probably, if you, if you were to look at the total signature of that earthquake it just, it just brought it back to more or less a neutral position at the end of that event. So I haven't really considered that in detail.

Moving onto the St Elmo's Courts office building rebuild. So there we have a, have an artist's view of the building, about six storeys high. A handsome building and it sits right next to, just on the right-hand side is effectively just the outline or the top line of the Christchurch City Council civic office building. So it's right next door to the civic offices in Hereford Street. So currently this project is at the detailed design stage with the base isolator supply contract out to tender. So we'll, we should know what the base isolator's going to cost by the end of this week. The completion of the building is scheduled for June 2013. It's six storeys high plus it has a full depth carpark basement and the building having a carparking basement is really, really great from the point of view of making it a candidate for base isolation. So we didn't have to create a basement or a floor space. The building requires carparking in the basement. The floor plates are approximately 21 metres by 40 metres. It has a total gross floor area of approximately 4800 square metres including a large penthouse terrace and in addition it has about a 1000 square metre basement. So it's only about a quarter of the total size of the Christchurch Women's Hospital building. The owner is offering tenants a premium level of seismic safety and anecdotally there currently appears to be quite a strong tenant demand for superior seismic protection in buildings – I think for fairly obvious reasons. Initially the owner had a preference to build using wood construction, utilising emerging technologies for engineered LVL wood. The owner at one time had Professor Andy Buchanan as a tenant in his building and has continued a relationship with Mr Buchanan and I think Mr Buchanan might have prompted the idea of a wood building. We seriously considered the option for an all timber-frame construction of beams and columns and post tensioning the beams but we struggled with the requirement to achieve inelastic energy absorption at the beam column junctions that didn't overstress the columns, the timber columns and cause the floors to crack in large earthquakes. We also expected difficulties in accommodating the two-way post tension anchors in the timber beam column joints. For that all wood type construction we considered in-structure damping devices at beam column junctions but quickly ran into difficulties with the very large numbers of dampers and their fixings that would be required. For example, an internal column could require up to 80 dampers in this particular building and 160 fixings to columns and to beams. While that type of structure might be self righting after an earthquake it would undergo large and potentially damaging inter-storey movements that could severely damage floors, linings, building fixtures and contents. On balance we decided that it would be much better to replace up to 240 elements, that's the dampers and the fixings with one base isolator per column and get the added protection for the building's superstructure, linings and fixtures from the damaging effects of large earthquakes. We were designing for close to nil structural damage in a 500 year return period earthquake and for very low non-structural damage in that event.

We've taken advice from Dr Brendon Bradley at the University of Canterbury on the selection of the seven earthquake records and scaling factors that are required to be used for the design of the isolation system. The building super structure layout together with the basement carpark configuration suited a two-way structural framed building above the isolation plane where we have columns on a grid layout of approximately nine metres by nine metres. Our design will be checked and supported by a detailed structural peer review by Mr Alistair Cattanach, a consulting engineer at Dunning Thornton in Wellington and the designer of the Wellington Hospital complex. St Elmo's will be supported on 16 lead rubber bearings and we expect that the supply of the lead rubber bearings will be less than $200,000 in total making it about $12,500 each. Remembering this building is quite a bit smaller and much lighter weight than the Christchurch Women's Hospital building which is why we're expecting a much smaller bearing size and hopefully cost for this particular building and once we've allowed for the cost of the heavier construction required for the ground floor level above the isolation plane in the special seismic gap and level space cover plates that will surround the building we expect that the total cost of base isolating the St Elmo's building will be less than 5% of the total construction cost of that base building. The supply for the base isolators is currently out for international tender and the first stage of the building consent will be lodged at the end of this month with construction starting on site in May of this year. The building is expected to be completed on or before June 2013.

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Now for the St Elmo's building the lead rubber bearings we expect will move by plus or minus 220mm in any direction in a 500 year return period earthquake and will move plus or minus 400mm at a maximum credible earthquake it would have a return period of about 2500 years. The lead rubber bearings in the isolation system will only transmit about 0.24g accelerations across the isolation plane in the DBE or the design basis earthquake event which is expected to occur once every 500 years and the period of vibration of the building will be about 2.2 seconds which is well down on the period of vibration of the undamped building which is about .7 of a second. The isolated damping is provided is approximately 30 percent and the effective system damping is 20 percent and that compares to a figure of around about five percent damping that we'd expect from a normal conventional building are up at 30 percent and effective system damping at 20 percent.

We are expecting this building will be the first base isolated office building in the South Island and we hope that others will follow our clients' lead and we'll see more base isolated buildings constructed over the next few years. That concludes this presentation and I thank you for your attention and I will be happy to answer any questions.

JUSTICE COOPER

Q. What sort of site investigations did you carry out in each case before concluding that base isolation was an appropriate seismic resisting system?

A. Sorry, could you repeat the first part of that question?

Q. What sort of site investigations did you carry out because we've heard evidence and I think Mr Kelly has given evidence that base isolation is problematic where there are soft soils?

A. Well a site investigation was carried out and on this particular site, as is pretty typical of quite a bit of central Christchurch, we have very strong gravels at about three metres depth and our basement floor will sit down into those gravels so they will be intimate with the gravels, and in the context of considering base isolation and the effect of the particular seismic signature of the February earthquake that affected Christchurch which had quite a strong response in the longer period range, there we've taken advice from Dr Brendon Bradley. He has actually used, of the seven earthquake records that we are using for our non-linear study of the isolation plane, isolation system, four of those records are for earthquakes – two from the Darfield earthquake and two from the February Lyttelton earthquake with appropriate scaling factors and being very mindful of the period of vibration of the superstructure and the period of vibration of the isolation system and he has come up with scaling factors that we are very happy with. We did a preliminary design on that building using Professor Nigel Priestley's displacement-based building design textbook and when we did the final design using the specific spectra from these earthquakes – four of them from local ones – the two sites we used were the Botanic Garden site and the Christchurch Hospital site where between ourselves and Dr Brendon Bradley, we established that the soil conditions were a pretty good correlation with what we had on our particular site and the St Elmo site and in comparison with, once we used those particular records and did the non-linear study we in fact found that we came within approximately five percent of the earlier preliminary and concept design using the displacement-based designed theories in Dr Priestley's book. So we felt confident, we feel confident with the result and I'm sure that confidence will be shared by our peer reviewer in support of a building consent application. With St Elmo's I think it's also, we are not designing an IL 3 building or an IL 4 building. We're designing it using the spectra for an IL2, a normal building, but we're delivering up a standard of protection which far exceeds a contemporary building design for the new IL2 (inaudible 16:42:25).

COURT ADJOURNS: 4.42 PM

COMMISSION RESUMES: 4.48 PM

PANEL DISCUSSION WITH DR RAJESH DHAKAL, DR RICHARD SHARPE, PROFESSOR NIGEL PRIESTLEY, DIDIER PETTINGA, TREVOR KELLY, MEGAN DEVINE AND GRANT WILKINSON

JUSTICE COOPER:

The idea is that the panel discussion should deal with issues that we've had in the course of the day and I wonder if perhaps the best way of starting would be to ask you one by one whether there are any further comments that you'd like to make in relation to your particular presentations now that we've heard from everybody. So I'll do that and I think I'll just go through in the order that we've had and ask Dr Dhakal to speak first. Are there any issues that strike you as matters, concerning the issues that you raised with us, merit further attention having regard to what others have said today?

DR DHAKAL:

Thank you very much, Your Honour. Yeah I would like to add a few things on top of what I presented in the morning, especially given the level of details now we have got into some alternate design methods like displacement-based design and the technologies surrounding base isolation and its effectiveness that have been discussed so far. So, finally, what comes down when it is the time for public and the technical people to judge the performance of different systems, be it the physical system or physical technologies or the designer proofs that we use. Finally comes down to whether that helps us in achieving the two objectives. The first is life safety. The second is cost minimisation. So unless we do that the cost to minimise something we directly put that as the target in our design approach because indirectly trying to come up with a solution and saying that, hey this has worked well, but when it comes down to the cost factor and if we cannot prove or give evidence that any system that we have come up with has not reduced the economic implications to the community and the society that probably will not be accepted wholeheartedly by the community and the public. So definitely it is my personal impression is that in order to achieve that loss minimisation which comes from the three Ds – the damage, which includes the structural damage, non-structural component damage, and the damage to the contents – that those three factors are going to contribute to the damage; and the downtime which is the business interruption which is a direct by-product of, in fact, damage, because if we avoid damage then if we avoid damage to the structural and non-structural system and content we can avoid the downtime; and the injury or fatality which is also the by-product of damage. So ultimately it comes down to avoiding or restricting damage and for which, based on what we have seen so far, base isolation is definitely a better approach to go because it not only reduces the demands on the structural component of the building but it also reduces the floor acceleration which is a key component when we look at the damage to the contents, because the contents, for contents the demand is the floor acceleration. So if we can reduce that on top of the demand on our structural system that is definitely going to be an extra advantage because we know from quantity surveys that we have done for different structures that the non-structural and content value of that in several buildings, in most buildings I would say, apart from residential buildings, easily out-weigh the value of the cost of the structural component. So preventing damage to the structural component is key for life safety but for cost minimisation we cannot just keep on the laurels of different design philosophy or different technologies that addresses the minimisation of structural damage only. We have to pay huge importance or we have to give a very much intense care to the non-structural components and content which has given in the recent Canterbury earthquakes has contributed immensely to the total loss from the series of earthquakes we have seen. So, definitely, it comes down to controlling the non-structural component damage and the contents damage once the life safety target is achieved, which is directly dependent on structural damage. Thank you.

JUSTICE COOPER:
Thank you. I was going to go to Dr Sharpe next.

DR SHARPE:
Your Honour I think you've heard quite a consistent message from all of us today and that is that we have the technologies, we have the innovation, we have lots of things up our sleeves that we can do for buildings. We can deliver more than we are often asked to do but there needs to be a desire to do that. We are hearing the pros and cons of various systems. There will always be pros and cons and my colleagues have set that out in some detail and I think I, at the end of the day, are very heartened to hear the storey of St

Elmo Courts because I think that is, that's showing the right decision's been taken into account by the developer. He's got one eye perhaps on the demand that might be out there and I think we've had a wake-up call that it's in the consumer's hand as to what level of protection that you can have in a building. If you're prepared to pay for it, it suits your purposes, mmm.

PROFESSOR PRIESTLEY:

A few points largely in relation to Dr Pettinga's presentation which I'd just like to mention. One of these is to do with the irregular structures in particular. You're saying that there are problems in using the technique for irregular structures at the moment. I think that there are some advantages actually with irregular structures in that displacement-based design principles enable you to minimise the effects of irregularity and that you can, for example, if torsion is shown to be a problem with the layout of the structure itself you can minimise the effect of that with displacement-based concepts by shifting the strength around in the building which is something you can't really do conceptually in force-based design. An additional problem, perhaps, there is associated with the displacement profile where he mentioned that if you've got an irregular structure the profile may not correspond to the profiles that are suggested for routine design but it is a comparatively simple matter to iterate your design, in other words start off with an approximately displacement profile, go through this with the conceptual design without going into the details of it, see what the actual displacement profile looks and then use that for the final design.

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As far as familiarity and review of familiarity or designer familiarity and review of familiarity I don't think, these are certainly a problem at the moment but the way in which the displacement based design is likely to be implemented and the way in which it has been so far is that it provides an alternative method of design which is not compulsory in the initial stages but is given as an option that the designer can use and there is a lot of historical precedence for precisely this approach. In the 1960s the structural design of buildings in New Zealand changed as it did in most parts of the world from elastic design to ultimate strength design. Not just for earthquakes but for all aspects and this was something that was very unfamiliar to the older people in the design profession but easily picked up by the younger designers. But the two methods were kept as acceptable alternates for a considerable number of years, so that it’s not a matter of suddenly changing everything but actually having something or other where the options are there. And if the options are there in the code it also makes it rather more simple for what Dr Pettinga was suggesting and this is for cherry picking bits out of the displacement based design and using those to advantage in the force-based design which I think is one of the, one of the significant advantages of having it in a codified form as well. Essentially that's the only points that I had associated with my own presentation.

Am I able to ask a question that perhaps could be clarified by some of the other people?

JUSTICE COOPER:

Certainly.

PROFESSOR PRIESTLEY:

In relation to seismic isolation. First just one point. Megan Devine mentioned that there was no, there was no maintenance required of seismic isolation units and I think that this is in relation to New Zealand and in relation to buildings and in relation to lead rubber bearings primarily. In the United States for example, with other systems that are being used as well as lead rubber bearings, there is very detailed and code specified approach required for routine removal and testing of bearings particularly for bridges to check these out and not, you know, some devices I think particularly friction sliding or pendulum, friction pendulum devices are possibly more susceptible to changes in material properties and stiffness over the years than perhaps are lead rubbers, I'm no expert in that. But the other thing that I wanted to briefly mention is related to the Christchurch seismicity and we've heard from Mr Wilkinson that in the St Elmo's Court there's good local soil conditions underneath there in terms of deep gravels which can be expected to provide a good base for seismic isolation, but I'm still a little concerned about the apparent bowl effect of Christchurch's deep sediments in general and the fact that or the appearance that there seems to have been particularly in the Darfield earthquake that the energy of the waves went through the city and bounced off the, off the hanging wall of the port hills and came back to provide this natural period with a big spike at around about the 2.5 to 3 seconds. And I wonder if that would also be transmitted through the gravels as well as through the softer sediments. I don't really know enough to, to argue about that. But I'm, I presume that because of the fact that he is using actual records from the Botanic Gardens and the Christchurch Women's that this is automatically considered in the, in the analyses but I would be interested to know what sort of scaling factors were used to the records in his design?

JUSTICE COOPER:

Mr Wilkinson can you help us with that as we speak?

MR WILKINSON:

I can if you just bear with me a minute. I've got the references and scaling factors. If I can get that?

JUSTICE COOPER:

Yes that's fine. Well perhaps we'll come back to that and ask Dr Pettinga for his comments.

DR PETTINGA:

I guess really it's just a couple of points that I see as being a general agreeance from the presentations we've seen today. The first one I think is that we're all starting to focus in that the current design philosophy as laid out in our codes isn't really providing engineers with enough focus towards identifying damage potential for buildings, and that's design in general and really then that draws from this idea that Professor Priestley had just reiterated that there is a need to start moving our code towards some more explicit detailing and cherry picking from the better options, parts of displacement based design, and I think that if we do have displacement based design as an alternative listed out in the codes then it does start to bring us closer to moving into something like that. The other point towards that, and it really seems like given the heightened awareance [*sic*] sorry yeah, awareness of the developers in Christchurch with regards to building performance that we will start seeing more advanced systems such as supplemental damping, base isolation and so on being implemented in which case the use of displacement based design again we do seem to have an agreeing idea that this is something that we can apply quite efficiently to these types of systems. So it does seem that we're all starting to come down today with our presentations to a common line and it is just a matter of determining what that line has got to be.

JUSTICE COOPER:

Just on that. What is the current status of attempts if there are any to give code expression to displacement based design?

DR PETTINGA:

In New Zealand or colloquially?

JUSTICE COOPER:

Yes in New Zealand?

DR PETTINGA:

In New Zealand, ah, probably your colleague is more attuned to that. Yeah, it's, we already have some inherent pieces in our, in our design code such as material strain limits. I have to admit that I'm getting familiar with these again having been out of the country for some amount of time but I have noticed that some of those particulars are quite explicit and then there are some gaps between the loadings and the design codes that we use for materials that leave an engineer sort of floating a bit as to what they're actually shooting for.

PROFESSOR PRIESTLEY:
Could I make some additional comment to that.

JUSTICE COOPER:
Yes, yes.

PROFESSOR PRIESTLEY:
Firstly New Zealand Transit Authority has a research or a consulting contract out at the moment where the displacement based design code is being considered for bridges in New Zealand.

JUSTICE COOPER:
It's the Transport Agency?

PROFESSION PRIESTLEY:
So that is some or other which is underway though it's got rather put on the back burner because of other work associated with the earthquakes. They're a very useful document which is, perhaps can be considered as a starting point in New Zealand is a draft model code which has been developed in Italy based on input from quite a large number of people. It's in the second literation of this and it just printed now and it's based on the principles of displacement based design for a wide range of structures and is a good starting point anyway I think.

JUSTICE COOPER:
So that's an Italian document?

PROFESSOR PRIESTLEY:
It's Italian in English.

JUSTICE COOPER:
It's in English. All right.

PROFESSOR PRIESTLEY:
But it was just, it was generated largely in Italy is what I'm saying there at the Rose School.

JUSTICE COOPER:
I was going to ask you if that was coincidence?

PROFESSOR PRIESTLEY:
It's based on the chapter in my book which puts the thing but that's the extent of it.

JUSTICE COOPER:
But there's no, there's no formal step underway through the standards.

PROFESSOR PRIESTLEY:
In New Zealand no. No. Apart from in the transit.

COMMISSIONER FENWICK:
Can you let us have that reference please.

PROFESSOR PRIESTLEY:
Yes I'll give you, I don't have it with me now but I'll –

COMMISSIONER FENWICK:

No, no if you could just email it to us that will be fine thanks.

PROFESSOR PRIESTLEY:

Sure.

JUSTICE COOPER:

Mr Wilkinson did you manage to find what you were looking for?

MR WILKINSON:

Yes thank you. And the scaling factors that we've used for the Darfield earthquake for the 500 year event we're looking at scaling factors between 1.9 and 1.95 for the 500 year event and between 3.4 and 3.5 for the 2500 year event and when we consider that the 22nd of February event for the, the scaling factors as used for the two sites vary from 0.85 to 0.88 for the 500 year event and for the 2500 event they vary from 1.54 to 1.59. We've used three other records as well as being most appropriate to the sudden acceleration. Those seem really quite conservative.

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JUSTICE COOPER:

Do you want to comment on that Dr Sharpe.

DR SHARPE:

You probably saw me raise my eyebrows. I'd like to comment on a slightly different thing that I'm just hearing here. There's a lot of concern in the market place that the Royal Commission is going to lift the requirements of the code and our commercial clients are asking us for advice on what the Royal Commission is going to come out with.

JUSTICE COOPER:

Well I'm not sure where this is heading but we ask the questions here.

DR SHARPE:

We are saying that in our opinion we don't think there will be great cost implications of the changes to the code. There might well be changes in certain aspects but, and so I'm very concerned that some of the things that we're saying today could be interpreted that oh the code's going to increase mightily in a whole lot of areas and I don't think we've actually been saying that today.

JUSTICE COOPER:

Well I must say one hears rumours about what one is thinking and it's just in the nature of the process but we've been going about the task for many months now. For some of that period we have been in hearings. People should not confuse the process of hearing with the outcome. A hearing is a process in which you listen to people's views and I don't think there have been many occasions where we have expressed views but when we have done so we've done so clearly I think and in all other cases the kinds of things you're talking about are merely speculation I'm afraid, and they will be until we make a final report. Don't know if that helps but….

DR DHAKAL:

Can I ask a question to Mr Wilkinson please?

JUSTICE COOPER:

Yes.

DR DHAKAL:

Grant, about that scaling factor. We all have the impression that looking at the response spectra of the ground motions that the February earthquake, the ground motions recorded in the February earthquakes were thereabout or slightly higher than 2500 year event and given that the natural period you're talking about for St Elmo's Court is 2.2 second, that's what you said right, and still using a scaling factor of 1.54 to 1.59.

MR WILKINSON:
Correct.

DR DHAKAL:
Is that a conscious decision to over-estimate the hazard or is it actually that's what it came out.

MR WILKINSON:
Effectively it meant, we've taken the advice of Dr Brendon Bradley as being an expert in this area and when we use those scaling factors we find we get a very very, a very good match, within 5%, of the early work we did using the displacement-based design approach from Dr Priestley's book. So we're feeling confident about that and, as well as that, unfortunately, our peer reviewer is overseas until the 15th of this month so we haven't had feedback from our peer reviewer on those scaling factors but I think we'll find that we'll get good support for them.

DR DHAKAL:
And I hope you're aware that the New Zealand Code, the 1170.5 currently doesn't allow scaling factors outside the region .3 to 3 so that is not allowed and for some of the earthquake records you have used scaling factor beyond three.

MR WILKINSON:
Yep we've looked at that and taken that into detailed consideration because we're using scaling factors that are as high as five for some earthquakes. For instance in the Turkey earthquake, was it Koseli –

DR DHAKAL:

Yeah Koseli.

MR WILKINSON

– and effectively we've specifically queried our advisor on this and he's put forward a very good case for using, for going outside the code limits in terms of scaling factors for this particular site.

DR DHAKAL:

Thank you.

JUSTICE COOPER:

All right now well I'd like to ask Mr Kelly if he could reflect on anything that he would like to at this point.

MR KELLY:

My main interest is in non-linear time history analysis and most of the speakers have mentioned that being used for isolation system evaluation or verified displacement-based design, but as Dr Pettinga has pointed out very few structural engineering firms are competent in doing this type of analysis. We certainly have the computer horsepower nowadays. My own desktop computer is not particularly high spec but it can run eight time histories simultaneously. When I graduated from school the big drawback was computer hardware. We didn't have the hardware to run it. Nowadays we have the hardware but we don't have the training of our engineers and we don't have the software to run it. I would like to see some initiatives to remove those impediments to a more general use of time history analysis. I don't know where they'll come from. Certainly the training is going to have to come from institutions but I don't think engineers we're getting in our company now have any more training in analysis than they had 20 or 30 years ago, even though the computer applications have gone up by about four orders of magnitude in that period.

JUSTICE COOPER:

So is this something that's been neglected at the universities? Is that what you're saying?

MR KELLY:

In a way, yeah. The era of the amateur programmer is gone with Microsoft Windows taking over. Richard Sharpe, for example, wrote among the first programmes able to do time history analysis on a main frame that had less power than his phone's got now really I think and since then it's got very difficult to do anything other than run PowerPoint, Excel and Word on our computers so there's a lot of impediments. Academics could explain it better than I could.

DR DHAKAL:

I do not know how it was probably a couple of decades ago but looking at the last almost 10 years is what I have been seeing is actually we give basic training to our graduates, specifically those who take the final year structural engineering papers in the under-graduate degree so they are required to use quite a limited analysis programme and until last year it used to be Raumoko. From this year we have changed that to Open (inaudible 17.15.04) because it has got open source so we are conscious that that is a very very important skill in structural design and analysis and from an academic point of view we have some limitation because we cannot spend a lot of time in one particular module and we expect that basic training that we give and a couple of examples that our students run for their assignments we expect the recruiters, the companies to build up on that and give a basic training on top of that. Definitely there is a basic understanding there when students come out of, ideally those students who have taken the final year under-graduate papers but they will need further training on top of that.

JUSTICE COOPER:
We're going to have a session later in our proceedings which focuses on the training of engineers so that's perhaps an issue that can be brought up in that setting. Sir Ron would you like to ask some questions?

COMMISSIONER CARTER:
Well I'd like to hear if the panel's got any views on a couple of matters. They're quite different from each other. But one is that we've heard other countries are moving more rapidly in picking up these new technologies or base isolation, perhaps, to be more specific, than the USA and New Zealand. Both of those countries have a fairly free market wide penetration of the design practices across many many firms. I'm just wondering if you can give us any advice as to why some of these countries might be putting much more into this type of device. Turkey was mentioned. Is the uptake in Turkey because of some institutional government directed advice to a whole range of buildings that the government can control and say well all hospitals are going to have these devices. I'm not sure from my brief visits to Turkey whether the general building standards would direct the broader population to start thinking in these, in this way. Perhaps there's something that could be useful to us in directing our advice back to, back to the, those that will receive our reports of what might be done to accelerate the acceptance in New Zealand of, of these new devices. So that's one question and I'll put the other one later.
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JUSTICE COOPER:
Does anybody have any comments they wish to make on that?

MR WILKINSON:

The focus this afternoon has been to talk about the cost of base isolation and there's been very little said when you talk about the benefits of base isolation but really at the end of the day it's all about the value of base isolation and if we make the comparison to the, to the car, nobody really questions the cost associated with the air bags of a modern car and everybody's prepared to pay a little more for that added level of security I think. So I mean I think it's a process of actually sort of moving beyond the cost and actually start thinking about the value. And just to expand on that point a little bit. An observation I've made is that, and I'm sure you've all made it, that everybody that's been through the, the major earthquakes in Canterbury has a different reaction to the earthquakes. They, some are terribly frightened, terribly, terribly frightened about aftershocks and really, really worried that we might get another big earthquake. Others are less so. Others are much more philosophical about the whole process. But in terms of moving forward when you're looking at decisions about whether buildings might be base isolated, what I'm detecting is, is that those that are making the decisions which might be a tenant committing to a long-term lease on a space are listening to their key employees that may be more frightened of earthquakes and effectively what I sense is that the decisions are being made on those that are most frightened and it's really difficult to tell a person that is really frightened about the earthquakes to not be frightened. I'm saying it's really easy to default to the more conservative view in terms of making decisions with regard to accommodation or other matters. I mean I've found that it's impossible to talk to people and try and rationalise their perhaps extreme response to earthquakes along the lines of, well, gee whizz, it's two orders of magnitude greater risk of driving across town or swimming at the beach. So there's a whole lot of sort of approaches to these things that are not directly rational and it may well be that there's just a short time window when this might apply and these effects might, might wear off. The other thing just in terms of value rather than cost is, is trying to sort of establish a long-term value for these buildings that might be base isolated or have other special protection in them and who can really tell as to how the marketplace might value those buildings. It's not about, so much about cost. If they have a greater resale value, if they can command a greater rental rate then cost just doesn't really come into it. It's about value. Thank you.

MS DEVINE:

I think perhaps with the experience that we've had in actually working in, in each of these countries I would suggest that maybe your question can be answered with one word and that's earthquakes, because we've found that the occurrence of a large earthquake actually makes people wake up to the need to not only be reactive after an earthquake but pro-active prior to. So we had the Kobe earthquake in 1995 in Japan. There was a base isolated building in Kobe. It performed extremely well. Use of base isolation in Japan went up exponentially. The graph just soared. We had the Gujarat earthquake in India around 2001. That created an interest in seismic isolation in India. We had the Marmara quake in the Middle East. We had the Kocaeli quake. We had the Bam earthquake in Iran. Each of those events has actually created an awareness of the need to be prepared for those earthquakes, particularly for very important buildings. So, you know, I suggest –

JUSTICE COOPER TO MS DEVINE:

Q. It's not just the occurrence of the earthquake. It's the occurrence of the earthquake in or near a major source of population, a major centre of population – important for attitude isn't it?

A. Yeah, but there's actually quite a long time delay, you know, between going to these places and saying you need to be base isolating so that you don't kill 30,000 of your people in an earthquake. There's actually quite a long lead-in time, education time for them. We've actually, you know, in New Zealand we are particularly fortunate in having experienced structural engineers. We're three-quarters of the way there really. You know these other countries are having to build up that expertise but I really think that possibly that's why we're seeing such a fast uptake in these other countries as compared to New Zealand which up until 2010 really hadn't had any major events in modern building history and so it was a lot more difficult to actually address the what ifs because the what ifs were, you know, in the never never. They would happen somewhere else and to somebody else and they haven't, they've happened here so.

COMMISSIONER CARTER:
So now is our moment.

DR SHARPE:
I would add to that Your Honour that we've also played a part in some of those countries taking out the technology. In the case of India which perhaps has five, five base isolated hospitals. After the Gujarat earthquake the New Zealand Government gave I think it was about $150,000 of design advice to them to replace the hospital that had, that had collapsed killing all 200 patients and that set it alive because they needed the confidence of other people. Turkey is the same thing. We've been promoting very heavily in there with some of the work that groups of us have been doing over there. Mr Wilkinson and I and some of our colleagues worked both there and in Romania where we actually wrote part of the code that would include base isolation and they're currently retrofitting a town hall in (inaudible 17:24:18) in the North of Romania. So it's a bit ironical that some of this comes from, from us and other people promoting heavily after earthquakes in these countries.

MR WILKINSON:
Well I can add a little bit to that in terms of that comparison about why the other countries might be having much more significant uptake with base

isolation. One of the big differences that we have between New Zealand and the countries that Dr Sharpe was talking about, Romania and Turkey for example, is that the general standard of new construction, new design and new construction I think in New Zealand is a great deal higher than it is in say Turkey or Romania or some of those other countries. So when they have their big earthquakes you are talking about very wholesale destruction of buildings including major loss of life. So, so maybe that is the trigger for them to move a quantum step forward into base isolation whereas we've had a large earthquake here and by and large most of our modern structures actually performed and remained stable enough such that people could get out of them whereas that is not always the case in some of the other countries that we're comparing here.

MR KELLY:

Yes and I think that's shown by the fact that after California earthquakes there has not been an uptake in base isolation the same. Both Northridge and Loma Prieta resulted mainly in a few more public buildings being isolated but not widespread use in the private sector. I'll just add to that, Japan I think is a special case. They have very restrictive requirements, far more than us, on seismic design and also the isolation manufacturers co-operate to a degree that would be illegal in New Zealand. It would be a cartel, and that influences the way isolation systems are supplied and used there. I'm not saying it's criminal or anything. It's just the different way their society works. Their construction companies co-operate very strongly in assigning different types of devices to different companies whereas Robinson Seismic will compete head to head with American companies offering the same devices on the same projects and driving down the prices.

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JUSTICE COOPER:

Is there any significance in the fact that so far as one can tell in terms of damage done to a central business district the February earthquake in Christchurch seems to be almost uniquely extensive. I mean there's a clean slate developing over a large part of what used to be central Christchurch and that area gets more extensive every day. What are the implications of that for new technologies?

MR KELLY:

The most sense would be to isolate the whole area and then build individual buildings on the same raft but again that would take a degree of co-operation and regulatory oversight that I don't think people would put up with. You can't rebuild all the buildings close together unless they're all on the same isolated raft, but they're all owned by different people.

DR DHAKAL:

I think there is something beyond awareness that needs to be gone there because if it is only because of awareness because weren't we aware that the old buildings need to be retrofitted. We were. But the building owners they didn't need to live there. They would put poor tenants over there and living themselves in some sort of safer building somewhere else. So what it needs to come down to is actually the incentive because I don't see a reason why the base isolated buildings and non-base isolated buildings need to pay the same premium for the same floor area because the premium, the insurance premium is directly, so directly (inaudible 17:28:34) to the risk they pose to the community and that doesn't make sense at all so unless those things are done otherwise there are others who will construct a bigger building and then sell it because that three percent of the cost out of 35 million that comes out to be like one million so it is only three percent of the total (inaudible 17:28:55) cost but that is probably as much as 100 percent of the profit that the builder is looking towards because the builder builds and sells it then basically out of 35 million the kind of profit that he or she might be looking at is three or four million which is a significant amount because unless there is a penalty for not using that technology which makes systems safer. Let's say that maybe when the builder builds it and tries to sell it that there is a penalty for not using that or there is an additional value of the building for having that system and that should be reflected throughout in the territorial authority's policy in the insurance premium otherwise apart from public buildings, the Government buildings, it'll never be done because most of the buildings even if they are long-term owners they are not going to live there and if base isolated building and non base isolated building that doesn't have any effect on the rent one can charge to the tenants. How is that going to appeal to investors? Like if I put myself in the shoes of the builder and investor I need to be convinced that there is a value of money.

PROFESSOR PRIESTLEY:

I think also there is a matter of familiarity with design and I think that perhaps one of the problems is that these new technologies, certainly base isolation displacement-based design, get brief mention in the University curricular but not in great detail. And this is partly a function of the fact that the amount of time that engineering students specialising in structural design actually get in structural design nowadays in comparison with what they did 40, 50 years ago when I (and, to a lesser extent, Richard you're not too far behind in age) did much more structural engineering. There was more time then to look at these sorts of things but this obviously will come up more in your education but I think that without the young engineers being familiar and enthusiastic about these things and being exposed to them they are not going to get used.

JUSTICE COOPER:

Well the other aspect, well it's a different aspect of it, but it may result in inhibition is that I understand from what I've been told that there isn't code provision for base isolation. Am I right in that?

PROFESSOR PRIESTLEY:
There is none.

DR SHARPE:
Deliberately I understand at the time, perhaps Commissioner Fenwick might remember, but I understand that it was deliberately not put out in detail in the 1170.5 because (a) it was difficult to codify and (b) it can be very restrictive and I think we've all anecdotally spoken about the American codes having killed base isolation by requiring so many belts and braces that it became too hard.

JUSTICE COOPER:
Surely there must be a happy medium?

DR SHARPE:
Well the principles of the code are all there. You can design base isolation using that principles that are very well in the code so you don't actually have to prescribe it but we're in a society which likes things to be prescribed because it's safe to follow what's written down.

JUSTICE COOPER:
If there were some code provisions would that help convincing the regulatory authorities that your design was acceptable?

DR SHARPE:
I don't think that's been a problem in New Zealand, regulatory authorities, and I would endorse what Mr Wilkinson said about using scaling factors that are a bit out of what the code people, the wise people who were sitting round the table at the time put into the code because I think we have enough knowledge to go outside some of those.

COMMISSIONER FENWICK:

It's very pleasing to hear that you have been writing codes for these overseas countries. May I point out that from Wellington's point of view the South Island is an overseas country. So how about it?

PROFESSOR PRIESTLEY:

Can I can add a little bit to that because I'm not sure it is easy to design using seismic isolation to the current codes. If you're using displacement-based design as Grant has done it is very simple and straightforward but it is not such a simple fashion. How do you determine what is the appropriate ductility for a seismic isolation? I don't think you can get that. How do you do the necessary aspect associated with the isolated structure and the isolated system? These are things which are very simple in a displacement-based design but I don't believe that they are simple in a force-base with our existing environment and in fact the American codes essentially do use a form of displacement-based design where they use the period to maximum expected displacement as the stiffness of the system.

COMMISSIONER FENWICK:

Just a couple of questions I'd like to ask. They are very quick. First of all you mentioned that you measured the displacement on the Women's Hospital building as I think it was 40mm and 120mm in the two earthquakes. How did those compare with the predicted displacements? Presumably you've done some back analysis to see how they compared with what you might have expected?

MR WILKINSON:

I haven't been in a position where I've been engaged or able to actually do detailed work on the Christchurch Women's Hospital. Effectively these are just personal observations up close of the scratch marks and of course the scratch marks were only really detecting the movement in one direction only as opposed to any direction and, yes, I guess as being the principal designer of Christchurch Women's Hospital I was kind of disappointed that we were getting plus or minus 120mm when it was designed for effectively for a 2500 year event of plus or minus 420, but it certainly went well into that inelastic range in both earthquakes which is what we expected so, yes, like I say, and to be frank we haven't had time to explore that at a personal level with so much other work on the go.

COMMISSIONER FENWICK:

I look forward to a delayed answer. Another point was we're getting a slightly different message in the cost of base isolation which I wonder if the two consultants could compare with the Robinson Seismic values and I think if I'm correct that the Robinson Seismic values, your values, were the cost of the isolators and installing them. Well I think the consultants were talking about the cost of the isolators and installing them and the additional costs of stiffening up the building so there was a stiff building on top of an isolator which had a long period. Am I correct in that, trying to sort of line up the two sets of costs?

MR WILKINSON:

Well certainly from my point of view on the three base isolator buildings I have been involved in and also reflecting Trevor Kelly's comments there, that you effectively end up with more or less a conventional structure at a conventional cost on top of the isolation plane because if you follow the American standard for designer based isolator buildings you have to design for and allow for the building to remain stable and the isolators to remain stable in a once in 2500 year event. Even if the building is a normal building that would otherwise only be designed to remain stable in a one in 500 year event had it been a convention design. You cannot afford to have the building fall off the isolators and so and as well as that it's also that you have to keep the superstructure stiff enough that it doesn't react against the isolation plane. So you've got to have these two distinct periods of the buildings. So you've got the stiff building at the top and the soft isolation plane. If the two get closer together then you could get into some sort of harmonic effect or certainly effects that become really really difficult to quantify and design for. So, in the case of the costs I'm referring to, the principle costs, for instance, in St Elmo's our basement itself will cost the same, there is no difference, but we do end up having to build a much stronger ground floor which is suspended in the air. It's almost like building a suspended foundation and I've allowed for that in my thinking in terms of costs and the rattle-space cover plates, to do them well is costly. If they're particularly where there's pedestrian traffic over the cover plates or vehicle traffic over the cover plates it's relatively expensive and you typically have quite a long perimeter around your building. Then beyond that it's the services connections but they tend to be just one-off connections – it's a stormwater pipe, it's a sewer pipe, it's an electrical cable and a communications cable – so there's not a lot of expense in that and that's where the principle costs come into it but the value, I mean you're getting so much more value, so much more protection is the final result.

MS DEVINE:
I presume here that you're looking at the 3% versus the 5%.

COMMISSIONER FENWICK:
Yes, that's right yes.

MS DEVINE:
Each of the buildings that we looked at and compared for our figures, the 3%, they included all the installation costs as you said – the requirement for a basement or crawl space underneath, the seismic gap and generally with the seismic gap you can have, just as Christchurch Women's does in parts, the

metal plates or you can have sacrificial paving but, and I have to be a little bit careful here with Mr Wilkinson because he is in a position of putting a tender out for this St Elmo's and we, of course, are in a position of tendering for that job so the 5% is an estimate perhaps on his part. We'll talk about that with him next week I guess, but there are, of course two economies of scale. So with the provision of a seismic system Mr Kelly referred to then you get a prototype testing and things. If those sorts of costs are spread over a larger project it's proportionately smaller so.

COMMISSIONER FENWICK:
Thank you, there are now a couple of points quickly. First of all we know that the response spectrum changes with the distance of the earthquake and probably with it's duration, I'm guessing a bit, if you probably build up resonance effects in the ground. So how would the, presumably you've checked the base isolation against an Alpine fault type event.

MR WILKINSON:
Yes.

COMMISSIONER FENWICK:
And how did that perform in that, in terms of comparison with, say, the Christchurch earthquakes.

MR WILKINSON:
I can't comment directly on that but my engineer has done that and the spectra that was selected took into consideration the Alpine fault event as well and, of course, the process is an averaging process in the standard that allows us to average some of the results according to the standard, the, to 1170 in terms of how to deal with the outcome of the various time history analyses across the isolation plane.

COMMISSIONER FENWICK:

But the 1170 doesn’t have the hump we see in the displacement demand between the two seconds and three and a half seconds. So how is that catered for?

MR WILKINSON:

Well we’ve been, like I say, our brief to Dr Brendon Bradley was to actually cover all of the earthquakes that are likely to occur, including the Alpine fault, specifically the Alpine fault and he has given us that full range of seven earthquakes and scaling factors. I think we’ve got scaling factors as high as 3.5. I think one was actually 5 actually to mimic the effect of a large earthquake at some distance.

COMMISSIONER FENWICK:

Now we heard from Trevor I think the problems of base isolation on soft soils and that sort of raises the question of where can we actually use this in Canterbury CBD knowing the type of soils we’ve got and the, I mean it sounds as though your particular structure is mounted on fairly good foundation conditions which won't be true through other areas in the CBD so is this system not suitable, for instance, on sands or areas that potentially may liquefy even though they have piles or are there limits there in terms of base isolation systems?

MR WILKINSON:

I would certainly expect there to be limits there. If you’re looking at soils that are prone to liquefaction then it may well be like Trevor Kelly was saying, that it’s a bit like the, was it the Orange Hotel in Tokyo where the very soft ground provided a base isolation effect. Of course you couldn't have that reacting against the formal base isolators as in a lead rubber bearing type system and if you were to be dealing with a site like that you’d be having to have some sort of deep piled type structure as well. So, yeah, effectively doing base isolation on a site, a very soft soil site or deep profile soft soil would involve quite a degree of very very careful consideration to see whether it was possible at all.

JUSTICE COOPER:

Mr Kelly do you want to add anything to that?

MR KELLY:

No I don't know enough about the local geology here to (inaudible 17.43.35). I'm a little concerned, I share Nigel's concern it might not just be the soil type but also the basin as well –

JUSTICE COOPER:

Basin effect.

MR KELLY:

– and I don't know enough about the seismology to be able to rule that out but Grant's looked into it more than I have obviously because he's got a project underway in the area which I don't have so….

COMMISSIONER FENWICK:

Right thank you.

JUSTICE COOPER:

Mr Wilkinson do you know whether there was liquefaction in the vicinity of the hospital site?

MR WILKINSON:

Sorry, was there liquefaction on this site, no.

JUSTICE COOPER:

And there wasn't in the vicinity of the St Elmo's site either.

MR WILKINSON:

Sorry on the…?

JUSTICE COOPER:

My first question was in relation to the Women's Hospital site.

MR WILKINSON:

I'm not aware of there being liquefaction in the vicinity of the Women's Hospital site and at the St Elmo's site we had a good look and checked that there was no liquefaction in the general vicinity of that site.

Could I just add to the question that was asked earlier on in terms of why there's a difference in take up of base isolation in New Zealand versus some other countries. And I think it's worthwhile just reflecting on what's happened with the recent earthquake in Italy, L'Aquila, and what's happened there in relation to the take up of base isolation following that earthquake, and I'm not an expert at all in this and I can see at least two other experts sitting in the audience that know a great deal more about this but my understanding of the situation is that they have a different government type structure and their equivalent of CERA has actually got on and actually built base isolated platforms, many many of them, very quickly after the earthquake. I think they talked about having the first one up and running about three months after the earthquake and these platforms are one storey up in the air which provide for carparking underneath and then the platforms are then provided to industry to actually build on top of apartment buildings or other buildings on top of them and I understand the numbers of these are sort of in the order of about perhaps 30 –

PROFESSOR PRIESTLEY:

It accommodated 18,000 people in six months.

MR WILKINSON:

There you go so I mean talk about take up.

PROFESSOR PRIESTLEY:

All three storey buildings, about 18 apartments I think from memory per platform.

MR WILKINSON:

So there's an example of take-up where the lead has come from the central government in a different sort of organisation, different government structure and political structure than what we have in New Zealand and I'm not sure whether they acquired the land or whether they owned the land beforehand but I mean they've just got on and one it.

PROFESSOR PRIESTLEY:

But, essentially, it was right from the very top down. It was Berlusconi saying to Professor (inaudible 17.46.38) you sort this out within six months and then any issues on the side would just be dealt with. They were pouring 5000 cubic metres of concrete per day at the height of this extraordinary construction progress. But that would be difficult in New Zealand.

MR WILKINSON:

Excuse me, do you know whether they actually had a design in the drawer ready to go or whether…?

1747

PROFESSOR PRIESTLEY:

No, the concept was developed after the L'Aquila earthquake. It was done in the weekend afterwards with Professor (inaudible 17:47:17) and a few other people in the pub here.

MR WILKINSON:

That really is earthquake recovery isn't it?

PROFESSOR PRIESTLEY:

Yes it really was.

JUSTICE COOPER:

Yes well our terms of reference don't extend to the political system. Sir Ron I think you had another question?

COMMISSIONER CARTER:

I'm just wondering if there's anything in any of the panel members' minds about the matter of balancing the advice that you're giving for us to draw conclusions from in respect of the two earthquakes that are most frequently referred to, that is the September earthquake and the February earthquake. The September one being something close to the design level earthquake and the February earthquake being closer to the maximum considered event. When we come to make our recommendations we're going to have to be very clear about what we're aiming at in terms of the sort of situation that we believe New Zealand should develop its design methodology to satisfy. I'm just conscious of the fact that we heard some numbers today that quote all up costs related to the whole of the event which includes suburban losses, liquefaction effects, et cetera. We're trying to concentrate here on commercial building improvements, you know the shutting down of the business sector of Christchurch and should we be aiming more directly at an event closer to the September earthquake when we start to talk about consequential secondary damage rather than the MCE earthquake which of course has really devastated Christchurch and devastated a lot of people in the process so I'm just looking to hear if the panel members have any concerns about the fact that we might be trying to solve the wrong problem?

DR SHARPE:

Well I'm prepared to say that I think we should be aiming to stick with approximately with what we've got now which is meaning that we should be aiming more towards the 4th of September earthquake than winning Powerball on top of first prize.

PROFESSOR PRIESTLEY:

I agree with that in general but I would say that there should be more consideration of the MCE in terms of displacement capacity for example the stairs displacement capacity, survival of people which perhaps has not been done to the extent that it could be, so I think that probably the force level that the intensity sorry of the earthquake that we're designing for at the 500 year return period is possibly okay. I think we can do better in terms of the design to minimise damage at that level than we're doing at the moment but I do think that more consideration for survivability needs to be done in terms of at least looking at the displacements associated with the maximum credible event.

MR PETTINGA:

Just to add something to that that would start to bring us in line with what's happening on the West Coast of North America where they're designing to a code level which is a 500 year return period earthquake but verifying against the 2500 year return period earthquake and it's exactly those critical, critical pieces and life-safety or collapse prevention that you're starting to target more than, more than anything so…

COMMISSIONER CARTER:

I think if I could just make sure my point is clear I'm sort of trying to focus on the secondary damage aspect rather than the life-safety I think we've got the point well and truly that one wants lives to be saved at the most severe earthquake but just a question how far one goes in terms of setting the earthquake actions when one's looking at the, the limiting secondary damage.

DR DHAKAL:

My opinion is as I mentioned actually we have not been using those three requirements explicitly in our design we currently use only two and third is automatically assumed to be satisfied which needs to be made explicit and the secondary damage the, which is damage minimisation or loss minimisation should be checked for the design basis earthquake and the life-safety should be checked for the maximum credible earthquake.

PROFESSOR PRIESTLEY:

My feeling is that the only way to do that is by displacement based design where you design for the level of damage that you will accept and that's not inherent or possible with the current design approach.

JUSTICE COOPER:

Well then this will conclude today's hearing. Our thanks go to all of you for the important contributions that you've made to our work. Thank you all very much, so we'll adjourn now until 9.30 tomorrow.

COMMISSION ADJOURNS: 5.52 PM