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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TRANSCRIPT OF CANTERBURY EARTHQUAKES SOILS - GEOTECHNICAL CONSIDERATIONS: FOUNDATIONS ON DEEP ALLUVIAL SOILS HEARING HELD ON 25 OCTOBER 2011 AT CHRISTCHURCH

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MR ZARIFEH OPENS AND CALLS

MISKO CUBRINOVSKI (AFFIRMS)

(inaudible due to sound problems)

JUSTICE COOPER:

Q. Just a moment – difficulty hearing, yes.
A. Yes.
Q. Now –
A. Can I, okay is it getting better, okay, thank you.
Q. All right perhaps you could start again. It was just an adjustment that needed to be made was it.

ASSOCIATE PROFESSOR CUBRINOVSKI:

Okay, good morning everyone. I am going to present the report on foundations on deep alluvial soils written by myself and co-authored by Ian McCahon. Can I have the next slide please. And here is the scope of the report. We were asked from the Royal Commission to provide the general review of the alluvial soils found in the CBD of Christchurch to explain their performance and the effects in the recent Canterbury earthquake sequence and to focus on liquefaction and lateral spreading which had a major impact on the performance of buildings within the CBD and, finally, to discuss some general concepts that should be followed in the design of foundations for buildings on deep alluvial soils. There were a few other bullet points with more details but given this I thought that we really have to provide the general review and address these issues with some technical base but not going too much into technical detail so that a wider audience can appreciate the outcomes and the report itself. I would like to point out that in addition to the activities and what is presented here we source from the larger data and experience coming out of Ian McCahon's experience in designing and working in the area of Christchurch for many years, in particular designing foundations for commercial buildings among other things. Myself I have been involved in liquefaction studies for quite some period of time, including six years of research of Christchurch soils where we started a research programme

"Characterisation of Christchurch Soils for Liquefaction and Seismic Sites" about six years ago and then we were both involved in investigations of the impacts of the recent earthquakes which includes details on performance of various structures and performance of soils in the suburbs of Christchurch and the CBD itself. I'm not going to address any details because we thought that if we go into details the report is going to be very difficult to follow but I just want to acknowledge that there are some important details that needs to be addressed and maybe this submission is addressing one of those kind of details. Next slide please. So this is the outline of the report. There is an introduction section at the top and a conclusion section at the end, otherwise these are the major chapters. I will go through all of this briefly in this presentation and, first of all, I would like to point out that deep alluvial soils affect ground response and structures in two profound ways. The first is basically as the seismic waves travel through the deep alluvial soils, in the case of Christchurch we are talking about maybe a 20 to 40 metres of recent alluvial deposits sitting on very complex structure of interbedded gravels and sealed sand and clay layers so all of this structure could be considered as deep alluvial soil and as the seismic waves propagate through these soils they are going to modify the ground shaking. Some frequencies are going to be amplified whereas others are going to be de-amplified or reduced so that the ground shaking, in terms of frequency content, is going to be very different at the top of the soils. So this is a major influence that I am going to illustrate by showing some response spectra. In addition as the seismic waves propagate through the soils actually they deform the soils. This ground deformation is quite significant and in extreme cases can be so large that we are seeing, as commonly observed during these earthquakes, the major cracks and the formation of the ground distortion which affects structures. So this ground deformation is really the second very important aspect associated with alluvial soils and liquefaction and lateral spreading can be seen as extreme manifestation of that kind of deformation where we basically say that the soils have failed because the deformation is so large it cannot be tolerated by structures. So I am going to address these key issues or effects of deep alluvial soils on ground, land performance and structures sitting on top of them or built within these soils. Next slide please. So I have thought it is important for us to introduce the phenomenon of liquefaction because it is extremely complex. As we are all aware it was a major feature in these recent earthquakes and I think it is good to go through the process and explain the process of liquefaction because then the deformation associated with liquefaction becomes more apparent and it is easier to understand it. Next slide please. So let's consider this idealised soil profile and assume that the soil is uniform. We are having sand with a given composition and density, the whole soil is uniform, and we do have a water table indicating the upper surface of the ground water so the soil above the water table is going to be dry, whereas the soil below the water table is going to be fully saturated. If we consider sand we have to understand we are talking about the composition of grains which are in contact to each other but then there is a lot of voids, open space if you like, between these solids. So, in the case of dry soil, the voids are going to be filled with air, whereas in the case of saturated soil below the water table the voids will be filled with water. If all of the voids are filled with water we say the soil is fully saturated so we have only particles and water once we go deep in the soil beneath the water table. What you need to understand, which is quite important, is that the volume of voids in the soil is quite significant. So if you consider sand it can exist in a very loose state, not so compacted, or in a very dense state when it is well compacted. When it is loose the volume of voids is equal to the volume of solids. So in the total volume 50 percent of the volume is actually voids and 50 percent is solids. When the sand is very well compacted then about 30 percent of the volume is voids and 70 percent is solids. So when the soil is in a loose state there is plenty of voids within the structure of the soil. That means the soil is very compressible. We can compress it from the initially loose state to the compacted state by reducing the volume of voids and this is exactly what is happening during earthquakes. May I have the next slide please. We are going to consider now these two elements and what is happening to the soil during shaking. So at the bottom of this slide we can see a time history of cyclic stress, those are stressors acting on the soil element and are shearing, trying to deform the soil and, at the top, we have a simple diagram illustrating this deformation and the consequences in terms of volume change or pressure increase.

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So let's focus our attention on the top left part of the figure where we are considering the dry soil. As the soil is going to be shaken by the earthquake there is tendency for the soil to densify and this is a typical feature of any granular matter as a result of phenomenon called the latency which is basically when shaken the granular matter is trying to densify. It is the same as you have coffee in a jar when you shake it obviously you're getting some densification. The same thing is happening with the soils. If the soil is dry air is compressible then the soil will densify during the earthquake. Now imagine the same element everything is the same, the density is the same but all of the voids are filled with water. When shaken by the earthquake the soil has exactly the same tendency, it would like to densify but because water is uncompressible for the pressures we're dealing with in this case it cannot densify so you can understand that this tendency to densify the soil is never going to increase the pressure in the ground water so this is the mechanism behind the build up of pore pressure or increase in the ground water pressure during shaking. Next slide please. Now if we're looking into particles at the left-hand side we are having the state of the soil before the earthquake or before liquefaction and we can see that there is contact between the grains and some contact force and if this is a close look at five metres depth there is going to be five metres of soil sitting on top of these grains and all of that load is going to be transferred through this kind of contact forces. There is going to be some hydrostatic pressure in the water from the water table being close to the water surface so there is water, there is pressure in the ground water and there is higher contact stress between the particles. When the earthquake starts and this tendency for densification is going to develop we are getting increase in the pore water pressure and at some point that increase in pore water pressure can be so high then it can eliminate the effect of the gravity forces and it will practically separate the grains. So suddenly the soil from a solid matter will turn into some heavy liquid and the particles are essentially floating, they're in suspension instead of firm contact. So that is the state of liquefaction. Next slide please. This looks little bit complicated but I think it will really help to understand the process. So at the top on the left hand side at the top we have some idealised very loose structure of the soil before the earthquake and of course we have contact between the grains and we have some pressure in the pore water. At the lower plot where we have a time history of excess pore water pressure this is basically showing how the pressure in the ground water is increasing during the earthquake. So we can see with very few cycles of shaking the red line increases and approaches the blue line, the blue dashed line horizontal line. When the red line is going to meet the blue line basically liquefaction is going to be triggered and liquefaction is going to develop. At that point the grains are going to be separated and that is illustrated by the schematic figure shown in the middle part of the top part. So during liquefaction we are having separation of the particles, they are not in contact and suddenly the soil loses it's strength and stiffness, it loses the capacity to support any structure basically. So the first thing to notice here is this increase in the pore water pressure is very fast. If the soil is very loose and the earthquake shaking is very strong this may happen in a second or two or three seconds. All of this is developing very quickly. One point here to mention which is important is we're, with this behaviour we're now focusing our attention on the behaviour of soil at particular depth. So this is let's say what is happening at five metres depth we want to understand in detail what is the – how liquefaction is developing particularly at five metres depth. So once the red line has approached and reached the level of the initial effect of overburdened stress shown by the dashed blue line the soil has liquefied, we are getting lots of contact, it loses stiffness and strength. The shaking is still continuing so there is this effect of shaking and the effect of the latency. On the other hand we already because these pressures are in addition to the pressures of equilibrium a water flow starts. At some part of the soil the pressure in the water is going to be much bigger than in the surrounding parts of the soil so water flow starts in different directions and most of all towards the ground surface because the water pressure at the ground surface is basically zero. So we have upward flow of water to dissipate these pressures and the decline of the red line towards zero excess pore water pressure is actually indicating this dissipation of pore water pressure due to water flow. Once liquefaction is going to develop and even before it is developed there is water flow so this is why we are seeing plenty of water coming on the ground surface and quite often because the pressures are very high the water is going to take finer particles of soils like silt or sand and will bring it to the ground surface so this is why we are having this, this ground surface and roads and properties covered by water, sand and silt as a consequence of liquefaction. During the dissipation of pore pressures there is going to be – the particles are going to re-establish contact and the ground is going to re-solidify as a consequence of this dissipation and it is going to regain some stable structure as illustrated at the top right-hand side of the diagram. There is going to be some settlement of the ground because a lot of sand and water has come out and what is important here to mention is that we have to understand that this re-solidification or settlement or – is happening in a sense there's a consequence of settlement of particles in the water so it is very, in some sense, similar to the process of the original generation of the soil deposit if it is alluvial deposit transported and deposited in water environment, this is again some process of water sedimentation. So we're going to get similar events of this to the original state if you like and the fabric or the arrangement of particles between the grains is going to be relatively weak when this process, as a consequence of this process, so the density is not very large and also the structure of the soil is relatively weak. That is why we are seeing repeated liquefaction in areas that have liquefied before. Now if we consider what is happening in terms of stiffness and strength of the soil during this, during this liquefaction process we have to understand that when the excess pore water pressure reach about half of the level between the origin and the horizontal blue line the strength of the soil is going to be reduced on half and the stiffness is going to be reduced by 40/50%. Once you approach very close to the blue line the strength is going to be 10 times reduced from the initial strength and the stiffness is going to be two or three times reduced. So that is why once the soil develops liquefaction it becomes extremely deformable so the ground displacements are going to be very large. If we have a 10 metre deposit the movement of the top could be plus minus

50 centimetres relative to movement at the base of this 10 metres. So there is huge deformation within the liquefied layer. We have to understand that every soil irrespective of how stiff or strong it is it is going to deform during strong earthquakes. The key difference is that very stiff soils are deformed very little whereas soft soils and especially soils that are going to liquefy are going to suffer extremely, extreme deformation and lateral displacements as well as vertical displacements in forms of settlement. So there is some sort of continuum in terms of the formation liquefaction is probably one of the extreme manifestations of this deformation. Next slide please.

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At the bottom left part we have a diagram indicating how the excess pore water pressure increases with depth in a uniform and, as you can see, the pressure is much greater if larger depth and this is why we are having this upward flow of water towards the ground surface. It is zero at the ground surface. The pressures are very high then in this upward water flow is going to take some of the finer particles to the ground surface and we are seeing this typical sand volcanoes on the surface. Of course this manifestation depends on other factors as well, the depth of liquefaction, whether there is crust of non-liquified layer at the top or not and in some cases can be quite massive and widespread. In Christchurch we have seen areas where these kind of sand boils are actually covering 50 metres, that kind of areas, and very large areas and the emergence of the sand boils and very thick sediments of ejected sand and silt at the ground surface. Next slide please. Lateral spreading is associated with liquefaction and if we focus on the top part of the figure I will illustrate the mechanism of spreading using this simplified diagram. The white line is showing the ground conditions close to the river spreading typically across close to streams, rivers or in the backfills behind key walls and we're going to notice a gentle slope towards the river in the ground. If the soil is liquefiable then during shaking liquefaction is going to develop under the conditions that I've just explained and this gentle slope in the ground is going to basically produce a motion which is going to be biased. The movement is going to be larger towards the waterway so there is push of the ground towards the waterway because of this very gentle slope. Once the soil is going to liquefy then the resistance is very small so there is very large movement of the ground towards the waterway. This movement can be quite often above one metre, sometimes even tens of metres or even more so significant movement. The spreading quite often is going to affect area that is far from the waterway – 50, 100, 200, 300 metres, in extreme cases even more so a large distance from the waterway might be affected by spreading. Any structure sitting in this area, shallow foundations are going to be stretched because of the spreading displacements and pile foundations are going to be subjected to very large lateral forces due to the ground movement in addition to the vibration induced by the shaking. So this is why lateral spreading is an extreme form of land deformation.

JUSTICE COOPER:

Q. Is it possible to make any general statement about the depth of the lateral spreading. In this diagram you've used the label "The Liquified Layer" so does the lateral spreading occur over the entire depth of the liquefied layer. Is that the way it works?

A. That really depends on the particular certification but if it is a uniform layer there is going to be deformation which is going to be the largest in the top of the layer and this is going to gradually reduce towards the depth. Another scenario is if we have a particularly weak layer that is thin then the whole top of the soil may slide on that layer and move as a block so there are really different patterns of movement which are affected by the particular structure and soil conditions and the site and we have seen evidence of several different mechanisms in the area affected by the recent earthquakes. In the lower part of the slide we actually have a typical manifestation of spreading that was seen in Kaiapoi where a large chunk of the ground, large block, moved as a unit about 150, 200 metres of the ground moved as a block and the largest cracks actually opened way behind that block. In a more conventional or typical spreading these cracks are going to be largest close to the river and as we move away from the river and the cracks are getting smaller so the movement is largest close to the river and it reduces with the distance from the waterfront whereas in this case of block failure actually the largest cracks opened way beyond 100, 150 metres and unfortunately we've got some residential houses sitting in that area that was severely affected by the spreading so there are different forms. What is common is that the displacement and the deformation is always extreme and the impacts on structures, buildings, lifelines, is very large.

ASSOCIATE PROFESSOR CUBRINOVSKI:

Next slide please. I will summarise this part referring to liquefaction with conventional method for evolution of liquefaction. This is the simplified procedure, a state of the practice procedure and it has five steps to evaluate potential of liquefaction as will less potential impacts of liquefaction on land and structures. In the first step we assess the susceptibility of soils to liquefaction. Not all soils are liquefiable. Ground water soils such as sands, non-plastic seals and gravelly sand and sandy gravels are liquefiable. Those are cohesion soils. The derive strength basically through fictional resistance and contact stresses are really important so if we lose this contact stresses basically we are getting the liquefaction as a consequence. Clays, on the other hand, have cohesion and plastic seals possess this cohesion which is another binding agent if you like so complete separation of grains is difficult to achieve so in those soils this pore pressures are never going to meet the horizontal blue line. There is always some residual stress, residual strength and residual stiffness. Only in some extreme case that I'm not going to mention here can kind of liquefy so we are kind of considering two types of behaviour during strong earthquakes – sand-like behaviour where liquefaction is a possibility and clay-like behaviour where liquefaction is not possible. This is not to say that all clays are going to perform very well during earthquakes. They are not going to develop liquefaction but they are going to soften dramatically so the deformation also can be quite significant and damaging but the mechanism of that development and response is different from the sand-like behaviour where we've got liquefaction. So in the first step we just want to understand, are the soils in the site liquefiable or not? If not, we are using the assessment based on clay-like behaviour and, yes, then we are going onto step two. If the soils are liquefiable then the question is whether the site is going to liquefy under the design level of shaking or the design earthquake. This is based on the output of the Seismic Hazard Analysis where we have certain levels of shaking depending on the level of performance required and we are doing the simplified analysis to answer whether the site is going to liquefy or not. If yes, then we are going on step three and are trying to assess what are going to be the consequences of liquefaction in terms of ground deformation. What is going to be the lateral displacement, what is going to be the settlement of the ground and we are quantifying the land damage and ground deformation due to liquefaction. Once that is completed we go to step four in assessing the impacts of ground deformation and liquefaction on structures. What kind of global settlements, what kind of lateral settlements are going to occur, how are they going to affect the foundation, what kind of stresses the ground deformation and liquefaction is going to induce on the foundation of the building if we consider the building. And finally in the fifth step if the performance is not acceptable then we are going to either improve the ground in order to eliminate the possibility of liquefaction which is quite often difficult or at least to reduce the impacts of liquefaction to a point that it will be tolerable for structures and the structures and the foundations are going to perform as desired during strong earthquake so this is briefly a summary of the procedure that is followed in liquefaction evaluation and I have to mention that in addition to this there is an advanced procedure which considers dynamic analysis and similar tools where all of this is put together in some complex analysis where we have the building, the foundation, the soil and dynamic shaking imposed by a time history of accelerations we consider the response of the system and practically evaluate all these steps through some sophisticated analysis.

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The simplified procedure is state of the practice, the advanced procedure is something that is recently suggested in codes that should be used for very important structures. It does provide huge input in the understanding of phenomena, however, it does require expertise by the user and it is very demanding in that regard so that is why the take-up from the, the uptake from the profession is quite slow. Not many people recognise its huge contribution. Next slide please. So this was all introduction so that we can easily understand what was happening in Christchurch and that will become even more evident once I describe briefly now the key features of the Christchurch soils. So this is a vertical section through the plains of Canterbury showing the typical structure – I apologise for the quality of the slide – in which we are seeing the soils at about 400 metres depth. The yellow area there is really showing the most recent alluvial deposit, about 20 to 40 metres thick beneath Christchurch, and they are overlaying a very complex stratification of gravel layers interbedded with fine grain soils such as clays, silt and sandy soils, and these gravelly layers are actually layers through which the water flows, as you can see from the top left part, from the alps, beneath the plains are then discharged at the seabed. These are layers with quite often with elevated pressures in the ground water which is a feature that probably contributed to some extent in the recent events to increase pore water pressures because you can easily imagine that if you have deformation in the ground then this elevated pressure there is going to be pathway for the water to penetrate towards the surface and contribute in addition to the contribution of the earthquakes in generating this pore water pressure. What is important here for us is we are going to focus on the yellow area, which is the top 20 to 40 metres of soils, and the layer immediately beneath that which is the Riccarton Gravel, the upper most aquifer. The composition that we are seeing here is really important for the amplification that I was mentioning or modification of the ground motion as the seismic waves propagate through these profiles. The 400, three to 400 metres of gravelly soils interbedded with fine grain soils have certain dynamic properties and they are going to amplify certain part of the motion and de-amplify other parts of the motion. The yellow part, or the recent alluvial deposit, is going to have similar effect but they are going to amplify maybe slightly different or the same part of the ground motion and de-amplify others, so there is very complex interaction in terms of how the input ground motion coming from below is going to be modified by these ground conditions that we've got and I'm going to illustrate that briefly through the discussion. Next slide please. Now this is a vertical cross-section in the east/west direction aligned with the direction of Bealey Avenue, so in the central part where you've got CBD that is exactly the cross-section along Bealey Avenue and then that extends to the west up to the Russley Road and to the east up to the coast and we are seeing now this yellow layer which is about 20 metres thick, which is the thickness of the recent alluvial soils been at CBD and that layer increases up to 40 metres near the coastline. There are two formations dominating in this layer. The (inaudible 10.43.39) formation, which is mostly fluvial deposits of sandy and gravelly soils which is predominant in the west part of the city and the Christchurch formation which has similar structure to the (inaudible 10.43.52) formation but those are influenced from marine sediments, lagoon sediments and swamp sediments which are predominant maybe in the eastern part of Christchurch. There are two meandering rivers, as you would know, the Avon and Heathcote, which also work the top superficial soils of Christchurch and the Waimakariri is known to have flooded Christchurch and significantly contributed to the shape and the ground conditions of Christchurch over a long period of time. The thickness of the alluvial deposit is quite important but also the specific soils and conditions of these soils and what we can notice here is that thickness is actually de-lined by the upper most surface of the Riccarton Gravel, this is a stiffer soil, older layer, about 14-70,000 years old so really competent in terms of foundations if you would like to specifically discuss buildings. This is not to say that there are no other competent layers and we are going to see more details in the later part of the presentation. The water table is also something quite important. From CBD, within CBD and to the east of CBD the water table is very high, about one metre from the ground surface, so we can say safely that all of the soils beneath one metre depth in Christchurch and within CBD and to the east of CBD are fully saturated. As we go, as we move towards the west the water table is lower so it goes about five metres depth maybe towards the western part of the Christchurch. That means we are having a five metre thick crust above the water table. This crust is going to provide some additional stiffness. You can think of this as a sort of bin so that is going to constrain the shearing of the underlying soil and is going to increase the resistance against liquefaction. The thicker the crust that contribution is going to be bigger. A crust of metre or two is not going to be significant in the case of very strong shaking because the crust is going to be cracked and that kind of resistance or stiffness is not going to be provided. Next slide please. This is an aerial view of CBD indicating streams in the central part of Christchurch as shown in the black marks of 1850s just illustrating that around the current flow of Avon there were a lot of streams and plenty of water on the ground surface which is saying a couple of things at least – that the soils are well saturated and also that the deposits are fluvial deposits and we can take off many old river channels which are very loose soils with conditions of high potential for liquefaction. I would like to point out that we should not expect to explain all the damage relative to the location of these streams because for liquefaction the history way beyond 150 years is really important so we may have some older channels three, four, 500 years ago that were dominant in these areas that we are not seeing in this 1850 map and that are still relevant in affecting the behaviour and the impacts of liquefaction. However, this is giving good understanding for the background of the development of these deposits and the overall conditions.

JUSTICE COOPER:

Q. Can you just orientate us on that diagram. Is that to the west, or left-hand side, is that Hagley Park?

A. Yes to the west is Hagley Park and we're, if we follow, this really runs from the, if you follow the river on the southwest at Hagley Park and rounds at the north-eastern part, which is coming very close to the Fitzgerald bridge. At the west, north-east corner, top right corner, we are roughly at Barbadoes Street and Salisbury Street.

Q. Right so the long street near the top going across this diagram is that Bealey Avenue?

A. No we are not seeing here the Bealey Avenue. The topmost horizontal street that we see here there is Salisbury Street.

Q. Salisbury Street.

A. Yes and on the right-hand side the vertical that would be Barbadoes.

Q. I see, right, thank you.

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ASSOCIATE PROFESSOR CUBRINOVSKI:
Next slide please. This is a vertical section or a cross-section across the soils along Hereford Street, it goes east/west and it is a generalised one just to show the dominant layers and soils throughout CBD and also to show the complexity. Now this is very simplified. If we actually go into great detail this (inaudible 10:49:37) is going to be extremely complicated to a point where we couldn't understand much what's going on so this is only trying to show the predominant layers through a cross-section. We're obviously showing only the uppermost 30 metres so if we look at the top left-hand side which is the west part close to Rolleston Avenue we're having a sand, and silty sand covering the top metre then we have a relatively thick sandy gravel about 10 metres thick overlaying loose medium sand and then dense, very sand – to very dense sand and we are meeting the Riccarton Gravel somewhere at a depth of about 23 metres or so. The profile is changing and if we see what is happening under Manchester Street then we are having silts and silt and sand and peat covering the top seven to eight metres, the sandy gravel disappeared in that part and from 10 or eight to 12 metres we have very loose sand and again dense sand so highly viable soils with different composition and different densities and this is the typical variability that we are seeing through CBD. From a foundations view point immediately one can start thinking and examining okay, which of these layers are really appropriate or combative for foundations? And you would think of the sandy gravel as a potentially good layer to put your foundation on and the deeper dense to very dense sand is certainly a layer that one would consider for putting piles on top of it and in the case of very heavy structures then the Riccarton Gravel which is quite deep at 22 to 25 metres would be the next layer to consider. However, we do have shallow foundations sitting practically on all sorts of soils. We do have to realise that the different sizes of structures require different foundations and impose different loads so it is not that the same layer is going to be the competent foundation layer for any structure it is structure

dependent but in general we can at least find three reasonably good layers of component that we would like to investigate whether we can put foundations on top of those. Next slide please. This is a plane view indicating the composition of soils in the middle part we see this red area which was the area that liquefied during the recent earthquakes and actually the red area is showing the manifestation of liquefaction during the 22nd February earthquake to be more specific. This is predominantly an area with mainly of sandy soils and that is why the liquefaction was a major feature there. In the lower left corner that is going to be the south-west part of the CBD shallow gravels are dominating in the, in the top seven to eight or up to 10 metres in some cases.

COMMISSIONER CARTER:

Q. Is that shallow gravel intended to convey the gravel near the surface or –
A. Yes.
Q. – what about the thickness of it?
A. The gravels here are reasonably thick between five, six to 10 metres that kind of thickness.
Q. Thank you.
A. And they are reasonably shallow two, three metres at most from the ground surface.

ASSOCIATE PROFESSOR CUBRINOVSKI:

On the right-hand side and bottom part of the figure we are seeing the soft silt and peat as predominant layers in the top seven to eight metres and a similar source I would say are at the top left corner which is the north-west part of the CBD again soft peat and soft soils dominating the top seven to eight metres. So I would say three-quarter of the city is basically having reasonably soft soils in the top seven to eight metres and that is obviously requiring some special attention in terms of the foundation design. Next slide please. So here view typical soil profiles it is actually maybe not the right word to say typical because they are so variable. Maybe just two soil profiles illustrating the Christchurch soils and their variability. On the left-hand side this north-west part of the CBD where we can see we have depth on the vertical axis on the horizontal axis is (inaudible 10:54:45) low count. That is a measure for the penetration resistance of the soil and we can see in the top maybe 10 metres very low penetration resistance indicating soft deposits of sandy, silty, and peat layers and then increase in the resistance once we hit the gravel and the underlying denser sand at depths beneath 12 metres. On the right-hand side we have the south-west part of the CBD where the gravels, the shallow gravel area that I was referred to on the previous chart particular so we can see the gravel starts from a depth of about two and a half, three metres and goes up to eight metres and then is overlying sand deposit. What is important to see, to mention here is that even though we have a reasonably thick gravel the penetration resistance is not exceptionally high, it is moderate I would say, indicating that even these gravels are deformable. You can see pore pressure is developing also in these gravels, the stiffness being reduced to some extent and the strength as well so we would expect some deformations to be happening in this layer based on the resistance we were seeing. Next slide please. This is similar profile but obtained from CPT which is more advanced penetration test. These are result of tests after the earthquakes done on Kilmore Street so it is within this zone that liquefied during the February event and we have here three penetration test Z1.4 to Z1.6. They are literally about 10 metres apart from each other so very closely spaced and we can see that they generally follow similar pattern, the – one of the key advantages of the CPT is that we get continuous record. Now you can see continuous resistance from the top up to 20 metres or so depth. If you follow the, on the left-hand side of the diagram the yellow line, the red line and the blue line, they increase at different depths. The yellow line increases at depth of 12 metres, the red one at depth of about 14½, 15 and the blue one at 17 metres, so within 10 metres distance from each other we've got part of the site where strong soil is encountered at 12 metres. If you move 10 metres away now this is at 14 metres and if you move another 10 metres then this is at 17 metres so this is giving you some idea about the variability of the soils even at a given site.

JUSTICE COOPER:

Q. Can you just explain for me in the left-hand diagram?
A. Yes.
Q. There's a box at the top left corner which has got 21 dash 4, 21 dash 5, 21 dash 6, or is that Z.
A. Z 1.
Q. Z is it?
A. Yes.
Q. What does that Z 1 mean then?
A. Ah –
Q. What does that signify?
A. Well after the earthquakes we have investigated the soils within the CBD in eight zones so this zone 1, Z 1.
Q. Right.
A. And test number 4, 5, and 6.
Q. So that's the test number?
A. Yes.
Q. Right.
A. And we have conducted maybe 10 tests in this particular zone around a couple of buildings so very densely spaced investigations to understand the details of the soil conditions around a couple of buildings.
Q. And are these different tests did you say they were typically 10 metres apart or does that vary?
A. They were spaced on the perimeter of the building.
Q. Yes.
A. And these are approximately 10 metres apart from each other. The tests that I am showing here.
Q. That was generally the case with the tests that you carried out?

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For each of the zones depending on the configuration of the buildings and our understanding of the soil conditions we had preliminary considerations how many tests to conduct and where to execute the tests so that is slightly variable but they are very densely spaced in all of the zones that we have investigated. We have conducted this kind of detailed investigations and we're also aware of the Tonkin & Taylor investigations which are trying to cover the whole CBD with the certain spacings of whatever is the distance between intersections of 100, 200 metres so that is going to give maybe some idea of general conditions throughout the whole CBD whereas we are looking at the very specific conditions around one or two buildings and how those are changing throughout the footprint of the building so we have a densely spaced programme of investigations and tests. On the right-hand side is a diagram illustrating the soil composition. It is a bit complicated but at the lower part, the label is IC and that is an indicator for the type of the soil. The low value would indicate gravelly soil, 2 IC, we can take it as sandy soil and IC of 2.6 is the limit separating between liquefiable and non-liquefiable soils. So if IC is greater than 2.6 we would expect that the soils are not liquefiable and this is just a rough idea, a rough index coming out of this CBD test trying to identify also the soil type. What we can see from here is that most of the soils are basically liquefiable. There is some, in the top two or three metres, there is larger presence of fine grain materials – silty soils. Some of those might have a certain amount of plasticity but basically sandy soils and liquefiable go quite deep at some points up to 20 metres or 17 metres.

Q. Could you just repeat those distinctions between the layers again for me, sorry the coloured bands.

A. Based on IC?

Q. Yes.

A. That is an indicator for the grain size, a small value, let's say about 1.4 will be where gravels are going to start and as we move towards larger number the soils are getting finer so sand soils will be about two. Of course this is a continuum depending on the grain size and then as we go towards values of 2.6 we are hitting the limit between liquefiable and non-liquefiable so soils with values smaller than 2.6 are considered liquefiable and soils greater than 2.6 are non-liquefiable and I would stress again that this is just a rough measure based on this test.

Next slide please.

The green size composition is really important because we've mentioned that sands, silty sands and the gravelly soils are liquefiable. We're having here a couple of plots on the vertical axis we have an indicator of percentage of what is the percentage of certain proportion or a green size in the total composition of the soil and on the horizontal axis is the green size. This is a green size distribution curve so if the curve falls within the yellow area then the soils are with high potential for liquefaction based on composition. On the left diagram we have green size curves for soils that were ejected in the area of Christchurch and we can see that they are mostly within the yellow band. On the right-hand side are results from our study on soils from the Fitzgerald Bridge site and FBM-10 is the regional soil with the diamond symbol and we can see that that is straight in the middle of this yellow band so by composition the soils that we have got here in areas where the predominant layers are sands and non-plastic seals are with high liquefaction potential. Next slide please. At the location very close to where I was showing the CBD results we have conducted undisturbed sampling of soils and here is just an illustration of that process with the drilling rig. We are trying to secure samples from throughout the depth of the ground and we have taken samples from three metres up to 13 metres depth and the whole point is to take the sample outside of this site without disturbing it, preserving the density of the soil and preserving the structure of the soil.

Q. Can I just ask was the greatest depth 30?

A. 13, one, three.

Q. One, three, thirteen?

A. Yes, 13, yes. So we're targeting, based on what I have described, we have identified critical layers and the suspicious layers for liquefaction and we are targeting those layers to understand better what was the density of these layers and what is their liquefaction resistance so in addition to the procedure, simplified procedure, that I have described which is based on field tests another approach is to take undisturbed samples from the ground and test them in the laboratory and examining in detail their resistance which is a very complex and quite often very expensive procedure so that is why it is not state of the practice but it will be used for very important projects. We had in our research programme this in mind well before the earthquakes and we were ready to go and do this just when the earthquakes happened. This is done in corroboration with Japanese University and Japanese company but we acquired the technology, the (inaudible 11:05:48) sampling which is an alternative way of collecting samples and it is not so expensive so these are the first samples coming out of the CBD soils somewhere in July and August and at the bottom right part of the figure you can see the sample in the laboratory and we have already conducted about 20 tests on the samples from soils at this site and we were happy to report those findings in the next couple of months or so. This is an alternative approach to evaluate liquefaction resistance in addition to the simplified procedures that I have described.

Next slide please.

So finally the age of the soils is another important factor. Ageing is producing increasing the strength and stiffness of soils in several ways by creating some bonding between particles, some sedimentation or small re-arrangement of the structure is really increasing significantly the strength and stiffness of soils so as the soils get older if they are not disturbed by major events such as earthquakes they get stronger with time so in that sense it is important to understand what is the age of the soils in Christchurch and this is that kind of plot where we have the depth on the vertical axis and then on the horizontal there is the age of the soils and the data is basically showing that in the top 10 metres of the Christchurch the soils are 4000 years old or less and in some cases a few hundred years old and by definition these would be soils that are susceptible to liquefaction. They are young soils from a geotechnical viewpoint and young soils the younger they are the more susceptible to liquefaction they are.

Next slide please.

So this is a brief summary of the CBD soils. We've got 20 to 25 metres of recent alluvial soils overlying 300-500 metres gravelly deposits. The top 20-40 metres are really fluvial deposits transported by rivers. We've got also some swamp and marine sediments. They are mixtures of gravels and silts. We've got peat as well. They are highly variable horizontally and vertically which is simply illustrating the process of deposition and the previous history and there are certainly areas in which sands and non-plastic silts are deposited in a very loose state. The water table is very high meaning that the soils are fully saturated and most of the soils in the top 20 metres are reasonably young or very young and when we put all this together they are all leading towards high liquefaction potential.

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Q. Can I just ask about the thick gravelly deposits at which you arrive once you get down to below 25 or 30 metres.

A. Yes.

Q. You say here that that layer is 300 metres to 500 metres thick. How is that knowledge arrived at?

A. Well this is really an estimate and in the discussions last week during the hearings there was some indication of the volcanics basement, you know those kind of data or deep wells or bore holes are really the only source we've got at the moment.

Q. So these are wells that have been drilled commercially to extract the water basically.

A. Yes, yes.

Q. And from that information there's a discernible pattern or sufficient information available for you to infer a statement such as that which you've made.

A. Yeah I would say probably a more accurate statement to say at least 300 to 500 metres because we haven't established very precisely the base rock as relates the layers at large depths but I would say maybe that is not dramatically important from a geotechnical viewpoint because those layers are very stiff and they're not going to contribute dramatically and they're not going to change the ground motion significantly and they're certainly not very deformable so, in that sense, they do not contribute in the two most important effects that deep soils are contributing. As we're approaching the surface the soils are more deformable and they're developing or affecting the ground motion more significantly. So if we are even doing some very sophisticated analysis from a geotechnical viewpoint I wouldn't go deeper than 300 to 500 metres because the velocities are so high that they're not going to produce any serious strengths or deformations there or modify the ground motion.

Q. Did you say the velocities are so high?

A. Shear wave velocities.

Q. Yes okay.

A. Those are the velocities of propagation of seismic waves.

Q. Yes.

A. And they're indicating how strong (inaudible 11.11.37) the soil are. Since they are very high we understand that the soils are very stiff and strong.

Q. Right, well, leaving on one side how deep that layer is but once you get down to between 25 and 30 metres you consistently reach this competent sort of gravelly –

A. Yes Riccarton Gravel yes.

Q. – deposits and that, is enough known to infer that that's a reasonably consistent substratum throughout the CBD once you've got down to 25 to 30 metres?

A. I think it is quite consistent but, of course, the thickness of that layer is also variable and not necessarily is covering all of the Christchurch as well so in that sense where we have it it is a competent layer, it has a large stiffness, shear wave velocities that I have seen are certainly above 3-400 metres per second, which is reasonably high, and they're drastically higher than the shear wave velocities that we are getting in the upper layers at 20 to 40 metres. So there is a sharp increase in strength and stiffness. Obvious that that is an aquifer to which there is ground water flow and one of the issues that we have got is when building on that layer foundations issues like construction issues and especially contamination issues of the groundwater will be quite serious because the water flow will be going vertically from that layer upwards as well as from the upper groundwater towards that layer potentially contaminating the ground water supply. So these are the issues that needs to be accounted for when trying to use that layer as a basis for foundations. It is not unachievable but it is something to really consider.

Q. To be careful about.

A. Yes.

Q. So the aquifer is typically at what depth?

A. Within the CBD I would say 22 metres, 23 metres, that kind of depth typically.

Q. Okay now we've sort of reached the end of a subject matter here so I'll just ask whether Commissioner Carter do you have any questions that you'd like to put at this point.

COMMISSIONER CARTER:

Q. Just dwelling for a moment on that age profile and the relatively young materials in the top five to 10 metres and that those, that's where the fine grain soils seem to be concentrated.

A. Mhm.

Q. I'm just trying to understand why there would not be fine grain soils deposited earlier on in the geological sequence. Was there something different taking place in the way materials were brought down off the mountains in those earlier years?

A. Well it really depends on which part of Christchurch we are focusing. In the eastern part of Christchurch actually this fine grain deposits are quite deep, they go 30/40 metres, up to 30/40 metres depth, and there are at least two global deposition environments that we can consider, one was the fluvial environment bringing and transporting lots of different materials and we have also marine sediments that are approaching the CBD as well which is related to the movement of the coastline which was going as far as the CBD or even slightly west of CBD at certain times. So it is really a complex product of several different environments that were changing over time and creating all of this.

Q. The density of those finer grain deposits will increase because of the overburdened pressures on them?
A. The density?
Q. As you go deeper, still (inaudible 11.15.58) fine grain deposits –
A. Yes, yes.
Q. – do they get increasingly more dense?
A. Yes as we go deeper just because of the sheer fact of gravity loads they are getting steeper and stronger.
Q. So their stiffness and their shear wave velocities, etc, will increase with depth even while there might be layers of fine grain materials at a greater depth?
A. Yes, we expect gradual increase in the shear wave velocity and stiffness and strength with depth.

MISKO CUBRINOVSKI:

Next slide please. Okay so we are now going to focus on the manifestation of liquefaction during the recent earthquakes and also see several response spectra observed within CBD. Next slide please. So this is a liquefaction manifestation map derived through reconnaissance conducted immediately after the 22nd February earthquake indicating areas of liquefaction by red, that would be moderate to severe liquefaction, yellow is low to moderate, magenta is liquefaction predominant on roads and less of that on properties, red symbols are indicating areas of where traces of liquefaction were evident but we judge that that is not damaging liquefaction for structures and blue lines without any other symbol are indicating areas of no liquefaction. So the first thing to notice is that along Avon River and in those suburbs like Avonside, Dallington, up to Bexley, liquefaction was really of moderate to severe intensity and many of us have seen those pictures and we know what we are talking about, massive sand boils and amounts of silt and sand covering streets and properties. We can see horizontal east/west oriented stretch that runs through the CBD and I'm going to focus on that later on. One thing to mention here is that this map was defined for purpose of general understanding distribution of liquefaction, there was no intention here to or no detail to define this on property basis so this should be taken as just for general information about the extent of liquefaction and coverage throughout Christchurch. Obviously we didn't cover the western part of the city because it was two of us doing this in seven or eight days so and there wasn't much of liquefaction happening there so we really concentrated on the areas that we thought are something that we need to colour.

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We need to understand that even within a single area like the red zone there is huge variability in terms of liquefaction manifestations. This is not to say that all of the red area the liquefaction manifestation was equal. The variation from moderate to severe could be really very large but this is at least some attempt to quantify the manifestation of liquefaction. Next slide please. Here three maps actually indicating the areas that liquefied with the white contours or areas that liquefied in the September, 4 September event 2010 then the black one is the areas that liquefied in the June 13th event and the other colour like red and yellow is the previous map indicating the liquefaction during the 22nd February earthquake. Now this is complete we never covered the whole area but it is indicative of several things. Obviously there is areas that liquefied repeatedly, clearly showing very high potential for liquefaction and those were deposited soils that for the size of the earthquakes that we are getting here can, can really easily liquefy. There are areas within Christchurch that liquefied four or five times and even very small aftershocks were causing liquefaction in some cases like aftershocks of 4.7 or 4.8 would cause a liquefaction that will affect very few properties. While the liquefaction was quite severe it was extremely limited so I would say that those kind of events while may affect few structures in general they are not damaging so we have to take that kind of interpretation from engineering view point is it not whether soil or, sorry water is going to come on the surface and whether liquefaction developed or not but what we really care about is whether the consequences of liquefaction in terms of ground deformation and effects on structures, so those kinds of small magnitude events the consequences are very low and we can tolerate those. Next slide please. Now another important feature is that when evaluating liquefaction we are basically comparing two things, one is the liquefaction resistance of the ground and the other is the severity of ground shaking produced by the earthquake. The simplified matter that I explained provides one easy way how we can compare two earthquake events and in terms of which one was more damaging or contributed larger loads for triggering of liquefaction. Based on that kind of comparison the conclusion from the, based on the acceleration records is suggesting that the yellow area indicated in this light was dominated by the 22nd February earthquake or in other words in this area the ground shaking produced by the 22nd February earthquake was the largest, whereas the green area in the green area the largest shaking considering all the earthquakes that happened up to date was largest by the 4 September earthquake. So in that sense we can compare now whether the liquefaction manifestation is consistent with this kind of interpretation and we can see that there are some white areas in the green, in the green belt suggesting that indeed the liquefaction that occurred during the 4 September earthquake kind of is consistent with the simplified interpretation because the seismic demand or the ground shaking produced by that event was the strongest for that area whereas for the yellow area obvious there is the 22nd February earthquake was the most significant the largest produced the largest shaking but also other earthquakes may have produced ground shaking strong enough to liquefy at least some of the loosest and very weak soils. Next slide please. As I mentioned before there are two prominent effects of deep alluvial soils and one was the modification of ground motion and I think this response spectrum plot really illustrate this effect quite well. These are response spectra. They are from the observed ground motions at Lyttelton, two stations sitting very close to each other about one kilometre apart maybe. The red one is sitting basically on a rock site whereas the blue one on a soil site with soft soil deposit of at least about 30 metres. These are response spectra for the same earthquake 22nd February earthquake and the difference that we are seeing here is the effect of the soil and how the soils modified the response spectrum so you can see on the red one there are very high peaks and low periods those are high frequency vibrations that were completely dumped or not completely but significantly reduced on the soft soil site so the blue line is well below the red line for short periods of .23 seconds. On the other hand in the range of long periods one, two, three seconds the blue line is well above the red one meaning that this soft soil deposit amplified this long period motion. So having one story house which will have a predominant period let's say of .1 second is that kind of house is going, building is going to experience very high forces if it is sitting on the rock whereas the forces are going to be much lower if the same building sits on top of the soil site. On the other hand if we have 10 or 20 story building on both sites then obviously the one sitting on the soil site is going to experience much larger inertia loads because that component of the ground motion has been amplified so there is drastic effect in terms of how soils modify ground motion. Now we would really love to understand these kind of effect for the Christchurch soils but as you understand from the previous week there are a number of factors contributing to the observed ground motion including the fault rupture propagation, basin effect and site effects is one additional contribution to the final ground motion that we experienced or observed. Liquefaction is going to be part of this site effects but certainly good understanding of site effects within Christchurch and for any city actually on deep alluvial soils is key requirement for a successful design and planning in terms of height of the structures because that is going to tell you which periods, in which periods actually the motions are going to be amplified. We have to be aware though that the features of the earthquake or the type of the rupture and the magnitude and distance are going to also change the features of the ground motion in the base rock if you like so the input that is going to vary with different earthquakes and then different earthquakes are going to be amplified in different ways but site effects are certainly something that need to consider very, very seriously for foundations in deep alluvial soils. Next slide please. These are response spectra within CBD at the Botanic Garden and Resthaven and the red ones are showing the spectra observed during the 4 September earthquake and the blue ones during the 22nd February earthquake as discussed before the 22nd February earthquake produced motions way above the design level and that was, the spectra are quite consistent within CBD. There are of course certain variations but they are not very far apart that kind of variability is well within the expectations and it is not really significant. In the report we are showing though also the spectrum for the Riccarton High School which is a good comparison because it is not so far from the CBD. It does have these very deep alluv- gravelly layers of 300 metres or so but it does not have very thick recent alluvial deposits and you will see because of the distance from the soils but also because of the characteristics of the soils beneath the station it looks quite different and maybe the Riccarton station is one good reference point in addition to the records that we have got at Lyttelton for further exploration of site effects throughout the city of Christchurch and that is our intention.

JUSTICE COOPER:
Q. I think you said that the diagram on the left-hand side which is from the Botanic Gardens -
A. Yes.
Q. – station is typical when the, shows a typical response within the CBD?
A. Yes.
1129
Q. How many stations are there within the CBD?
A. Within the four avenues there are four stations that are official but I understand there are a few other more records available so I think we probably have more, at least four records and potentially more.
Q. Yes, well in saying that this was a typical response were you reflecting your own knowledge of what was reported at the other stations?
A. Yes we have compared the, if we plot the response spectra from the four stations within CBD for the February event, for example, of course there is some variation but that is, in terms of response spectra, not very significant. The difference that we are seeing here is actually probably the largest, the two motion, if we compared it to blue motions, blue spectra for the 22nd February.

Q. Yes.

A. Of course there are differences there if you consider a specific period of one second there is dramatic difference but, overall, I would describe as those being similar. That kind of variability is always expected once you move away a kilometre or two because you're changing the distance from the source, the arriving seismic waves are different and also some local site conditions are going to be different so that kind of difference is to be expected.

Q. Yes all right, thank you.

JUSTICE COOPER ADDRESSES MR ZARIFEH – RE VIDEO LINK

ASSOCIATE PROFESSOR CUBRINOVSKI:

Next slide please. So I'm going to briefly describe the typical causes of failure within the CBD and when I say failure here I do not necessarily mean of collapse. We describe failure in soils always when the deformation in the ground is excessive and beyond what can be tolerated by structures. So if they are not meeting the performance requirements we would quite often refer to failure of soils. Next slide please. Because of the highly variable ground conditions there are different types of foundations that have been used within the CBD. There are shallow foundations with isolated footings and some tie beams. There are more robust shallow foundations, or mud or slab or rough foundations, under the whole footprint of the building, and then we've got pile foundations either using shallow piles or, in other cases, much deeper piles reaching more competent layers at larger depths and, eventually, we've got some hybrid foundations where there is some sort of combination between part of the building is on shallow foundation, the other part is on deep foundation, or you are having piles of different lengths. Then columns two and three are just summarising the typical building types in terms of the number of stories and the foundation soils. When I say foundation soils these are the soils immediately beneath the foundation. Those are the soils that are taking the loads from the structures, so these are the typical foundation layers depending on the type of the foundation used. So quite a range of different foundations within CBD. Next slide please. This is the liquefaction mud that I showed before so we can see the Hagley Park and here are the four stations indicated with not so visible dots and acronyms for the stations. On the upper part the red line, the red area indicates that area, the area that manifested moderate to severe liquefaction in some parts and the yellow one is low to moderate liquefaction and the red part going east to west we can see one solid black line as well as another black portion beneath that on the southern part, those are areas indicating ground with a particular weakness where it was noticeable when observing on the ground the damage that there was some feature like maybe old river channel or infill that created part of that area to be particularly weak so we could see large cracks on the ground and patterns of movement in the down-slope direction along that line so there was the larger damage and quite big difference when you consider what was happening to the structures on the elevated part, and when I say elevated I'm talking about one metre higher or half metre higher, and once you go on the opposite side which was lower. So there are those kind of zones of particular weakness in the ground and I think those are very important to be identified. The earthquakes that we've got here produce so severe shaking that pretty much what we saw on the ground is telling us where the weakest spots are so that is why it is so important for us to document what has happened here because that is really an excellent indicator about ground performance in relative terms – which parts of the city or blocks are stronger than others – and this is exactly the reason why we are doing all of this. Next slide please. So we are going to check the performance of a few buildings within this area that liquefied. This is one typical manifestation of effects of liquefaction where there is a large lateral deformation and displacement of the building in the order of about 15 centimetres and tilting of the building as a result of differential settlements. At the front part of the building, which is to the right on this figure, that is looking towards north, there was very severe liquefaction in that part of the building and on the street and, obviously, the whole building moved towards this liquefied area and because of loss of bearing capacity, loss of strength and capacity to support the building, that end of the building settled quite significantly.

JUSTICE COOPER:

Q. Could you tell us where that is?

A. This is in the corner of Armagh and Madras and this is one of our zones of intensive investigations within the CBD.

ASSOCIATE PROFESSOR CUBRINOVSKI:

Next slide please. So we are still at depth, the previous building is at the left corner below so just across the street on the other side of Armagh Street we have this building again showing the large differential settlements, the parts where you have the 26 centimetres of settlement. At that corner there was huge sand boil beneath the footing and obviously the liquefaction and the soils just beneath the foundation caused the differential settlements. This building is on shallow foundations so the isolate footings connected with tie beams were sitting on top of the liquefied layer and because this liquefaction was quite non-uniform throughout the footprint of the building it resulted in differential settlements. Differential settlements are very damaging because they propagate through the superstructure and create stresses and obviously that is going to be very difficult to expect anything but poor performance in the superstructure under this kind of stresses imposed by differential settlement.

JUSTICE COOPER:

Q. But those figures in white are the stages along the way to 26 centimetres are they –

A. Yes.

Q. At the different points of the building indicated.

A. Yes, so if we take the northern foundation as a reference point where we have zero then two, five, eight, 17 and 26 are showing the relative movement of those foundations to the northernmost point. So those are differential settlements relative to that reference point.

COMMISSIONER CARTER:

Q. Have you correlated the soil structure beneath that building yet? Do you have the soils information beneath them?

A. We have conducted here densely spaced (inaudible 11.38.37) probably at least 10 in this exact area and across the street we've got another 10 to 12 so and we've done all (inaudible 11.38.48) sampling so we have collected the data. We have not made the interpretation but definitely in the top six/seven metres there are very very soft soils.

COMMISSIONER FENWICK:
Q. Can you give us it's location too please?
A. Sorry?
Q. Location of the building.
A. Armagh Street and Madras Street.
1139

JUSTICE COOPER:
Q. At the side of the intersection?
A. Yes.

ASSOCIATE PROFESSOR CUBRINOVSKI:
Next slide please. In many cases we observed so called punched through settlement or sinking of building within the liquefied soil. This can be illustrated by the diagrams I have shown. On the left-hand side there is a simplified plot of uniform free-filled deposit or level ground without any structure on top of it before liquefaction and then as a consequence of liquefaction we would expect some settlement of the ground. This settlement can be quite significant. It really depends on the thickness of the liquefied layer. It could be 10, 20, 30, 40 centimetres. Now if we have a building on top of it and the red dust line is indicating the foundation depth then once liquefaction is going to develop in addition what you can observe now in the right-hand most diagram is that in addition to the settlement of the ground there is going to be sinking of the building within the soil because the liquefied soil simply cannot support this heavy structure so we have settlement of the ground and additional punch through failure or sinking of the building in the ground and this will be maybe typical manifestation of building on shallow

foundations beneath which the soils have liquefied. We've got evidence for this kind of behaviour for several buildings within this area.

COMMISSION ADJOURNS: 11:41 AM

COMMISSION RESUMES: 11:58 AM

PANEL DISCUSSION

JUSTICE COOPER:
Professor Bray, good morning from New Zealand. I am Justice Cooper. I am the Chairman of the Royal Commission. On my right is Sir Ron Carter who is a retired civil engineer and on my left is Associate Professor Richard Fenwick who is a retired academic structural engineer. Before we hear from you I will just go through a formality which is an affirmation.

JONATHAN BRAY (AFFIRMS) (VIA VIDEO LINK FROM CALIFORNIA)
IAN MCCAHON (AFFIRMS)

MR ZARIFEH:
Q. Professor Bray, as discussed with you, you've conducted a review of the report that was prepared and presented to the Royal Commission by Professor Cubrinovski and Mr McCahon and you've prepared a report dated 18 October, I wonder if you could just take us through the main points of that report, perhaps dealing with your primary review comments and relevant opinions and just summarise those for us and then we can have Professor Cubrinovski and Mr McCahon comment on those and perhaps engender some discussion about the points that you raise. So can I ask you to conduct that review please.
A. Yes, and although I'm not an expert in this area I would like to start off by congratulating you on your World Cup victory. My son and I watched the game over the weekend and it was the most tenacious defence that I've seen in terms of rugby, the most exciting I'd say last 20 minutes of the game where a score did not occur and although the kicking in parts was a little disappointing until Donald kicked one through, I think that the defence just was amazing in terms of their tenaciousness so I'm a great fan of rugby and definitely you're the experts when it comes to that and

now the world approves of that, knows that the All Blacks are the champions in world cup rugby.

JUSTICE COOPER:

Professor Bray, those comments have gone down very well here I assure you.

PROFESSOR BRAY CONTINUES:

I know from being in Christchurch before the earthquake and walking across Hagley Park around 1700 hours it's filled with rugby games. Seven on seven touch and it's a phenomenal game and it's something I got my son excited about when he was young and he's sorta gotten me back and excited about it. It's just a wonderful sport because it mimics life and it mimics all the struggles that a person goes through so again sincere congratulations on that. To a thing that I know a little bit more about is earthquake engineering and its effects on the built environment. I would like to summarise very briefly the report on the foundations of deep alluvial soils and I think the primary directive was to ensure that the findings sit well with accepted international best practice and so I'll cover an overall assessment then talk about some primary review comments and then talk about a few relevant opinions that I'd like you to consider. The first is the overall assessment. I believe the geotechnical earthquake engineering characterisation, the analyses, the findings presented in Court do reflect the state of the practice and best practices internationally. It conforms to accepted international practices. The reasoning is sound. It presents key issues on seismic site response, soil liquefaction, foundation performance in an excellent manner. It's well organised and I believe the information is insightful and the findings are well supported by the interpretations that are made of the existing information and in essence the foundations are sound. There are a number of very important points made in the report. I think the investigation and description of the soil, the historical aspects, the deposition or history is so important to understanding soil, understanding the age of these soils, it being upper 10 metres but only about, it's less than 4000 years old and many of the soils are only a few hundreds to a thousand years old I think is very insightful. I think the map that depicts the areas that had issues in terms of liquefaction from the September and February earthquakes are clearly the areas that are most prone to liquefaction in a future event. A liquefaction re-occurs where it's occurred in the first place. A documentation of the strong shaking during those events I think is well done and the assessment of the likely effects of an event on the Alpine Fault, although it's preliminary and submitted to be preliminary and I think there's additional information that needs to be investigated and brought forth, provides a very good oversight into what you might expect for a significant event on that fault. The recommendations made are useful and I see it as a very fine report. In terms of primary review comments, I think it's difficult to find things to critique but a couple of things that I think would be useful to add to the value of the report is, the report does make an emphasis on the value of the cone penetration test and I think that the Commission is probably aware of this test. It is a more modern standardised test. It's relatively inexpensive and the great thing about this test is it's nearly continuous in terms of showing the penetration resistance and some other parameters through the profile and so I think it would be useful for example in Figure 7 which shows some typical ground moorings and shows the typical ground conditions to have included some of those because sometimes I find, at least in US practice, the true variability vertically and horizontally of the ground conditions are not captured by borings with SPTs that are often, Standard Penetration Tests that are often spaced metres apart. The other thing that I think I'd like to make a point and this is more of a question that needs to be addressed by the Commission is, What's the definition of important structures? There's a use in the report in s 7 several statements are made with regard to work that should be expected for important structures and the definition of that is not made in that report and it's not clear what an unimportant structure is.

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In fact one of the things that we're wrestling with, and this earthquake has kind of brought that to life, is many times the building code has been developed looking at one building in isolation but if several buildings are required for a functioning city and we're looking at true resilience, you could argue that unimportant structures are important when you aggregate them and look at them in terms of the soul of the city, the heart of the city in terms of the central business district and so I consider most of the buildings in the CBD having seen it several times before the earthquake and now seen it twice after the earthquake I would say that many and if not most of those buildings are important in terms of their vitality and the economic sustainability of the city. In terms of relevant opinions I think that there are a number of things that we've been wrestling with that maybe you're also wrestling with and one is I look at the CBD and its buildings, its roads, its utilities, the other infrastructure components as a system and beneath that comprehensive integrated systems approach and this report is well done and it focuses on the effects of geotechnical phenomenon such as strong shaking and liquefaction on building foundations and the resulting impact of these phenomenon on the building foundations. It's not clear to me from my perspective on how this report fits within maybe the larger scheme that the Canterbury Earthquake Royal Commission's looking at because for example I'm not sure who is addressing seismic performance in utilities and other lifelines that are buried, entities like utilities and gas lines and water lines and liquefaction clearly and ground deformation from liquefaction can clearly affect the performance of these and the performance of these systems affects buildings and how well the buildings function after an event and so I would like to know how that is being addressed and how the overall resilience of the city is being addressed in terms of other things other than building foundations such as utilities. The other thing that I wanted to comment on is I think a strong case is made for the value of geotechnical engineering to discern the geotechnical conditions for a proposed project. If you engaged geotechnical engineers that are well trained they can adequately characterise the subsurface and with that information you can develop robust foundation designs and they can be appropriate for the ground conditions and the ground shaking levels that one might see in Christchurch and there's tremendous variability in the soils in Christchurch but I don't want that to be taken away as that there is chaos and there is things that we cannot know. We can know it, it's – there's not the – that's not knowable or unexplainable there is variability in the soils due to the fluvial environment and I think the geotechnical engineers and civil engineers who are working with you recognise that. I want to make sure that other people realise that there are, there's a key to the good site investigations are a way of unlocking that mystery and those patterns and so I think with these variable ground conditions that should even motivate you even more to do site specific investigations to develop well-designed foundations that are appropriate for the ground conditions. And we had that same problem, we still have that problem in California, United States, there's always a desire to control upfront costs and to try to limit development costs to allow development to go on and when faced with these same competing interests in California we often found that sometimes the geotechnical investigations were not as comprehensive as they should be and it took a earthquake for us, we knew this, but it took the 1989 Loma Prieta earthquake, the chair of the Seismic Safety Commission, a good friend of mine, Lloyd (inaudible 12:11:05) was chair at the time. He recognised the opportunity to turn that disaster into a positive thing to improve our resilience and so that led to the 1990 Seismic Mapping Act and very importantly the Act requires a geotechnical investigation in the seismic impact zones so maybe the entire area is not a seismic impact zone but there's areas that are most prone to a hazard like liquefaction or landsliding are zoned and it doesn't mean that they necessarily have an issue but it requires an investigation and another important component of that Act was that it requires peer review. Peer review has elevated the state of practice in California because an engineer knows that their work is going to be reviewed by another engineer and it's just brought up the state of practice. The last thing I'd like to comment on in my overall relevant opinions is I also want to show you that although this earthquake in terms of when it occurred and the ground, the fact of its close proximity to Christchurch may have been a bit of a surprise, the effects from the earthquake are not a surprise in most cases in the sense that the ground conditions simplified techniques that we have. Well calibrated simplified techniques for evaluating liquefaction hazards could point to these things happening. There's always a need to do site investigation and compliment that maybe cone penetration testing with some drilling, some sampling. For important projects there is often a need to do some laboratory testing and some advanced analysis but much of the issues in terms of liquefaction occurring and some of the impacts of liquefaction those are discoverable by using simplified liquefaction evaluation procedures that have been well calibrated against imperical evidence in conjunction with these comprehensive site investigations and so I think that that should motivate us to then require that collection that data, employ those techniques to provide that initial preliminary assessment and then that provides a good basis for going forward in terms of assuring that robust designs are conceived and implemented. And I'd be happy to answer any questions relative to my report.

JUSTICE COOPER:

Q. I'd like to ask about the Seismic Hazards Mapping Act, are you close enough to the practising profession to know how that's worked out and is it seen as a beneficial reform?

A. Yes I am close to that from two ways. One when the State was required to implement that Act they actually came to the professors at the University of California, Berkeley to develop a training programme to train the regulators so that they could properly implement the Act and so we went through several training sessions in northern and southern California to train the regulators on how to review reports and then very importantly we then went through I think a series of about eight short courses where we trained practising engineers. In fact they were motivated to go to that short course because they realised that we had trained the regulators and so the practising engineers wanted to know what we had told the regulators to look for. The thing that is done and I've worked as a practitioner in seismic impact zones in may seem a sounding for a place like California that has had the 1907 earthquake and 1971 San Fernando earthquake that it took another earthquake the 1989 Loma Prieta earthquake and the damage from it to force us to do this but there were people developing land without a good geotechnical investigation, without looking at seismic hazards such as liquefaction and landslide and this Act then now requires it and so if you're in these zones you have to hire a geotechnical engineer. You have to do the appropriate study and see guidance is given on what that level of study is and then very importantly somebody's hired by the State to do an independent peer review to make sure that the standards of that report are sufficient and it is brought up the state of practise significantly so that now work is done that wasn't done before and that work is done in a satisfactory manner, there's a check and balance there. I don't think it's increased the cost of developments significantly and in fact it will save us many in terms of retrofit when we do have an earthquake and these facilities have been designed with consideration of liquefaction and landsliding and other seismic effects.

Q. You said it was a system that was run by the state, it's the State of California that administers this Act?

A. Yes the State of California.

Q. Yes.

A. Yes sir.

Q. So it's not something which in California is left to what we call local authorities, councils?

A. The State of California implements the Act but they do have local jurisdiction actually do the day-to-day working so if you were working down in Los Angeles county, Los Angeles county would have a reviewer who then would review the report, ensure that a local geotechnical engineer was hired as a peer review so there, it is decentralised and handled at the city and county level within the state of California.

Q. I see, yes thank you.

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COMMISSIONER CARTER:

Q. The question that I have is I heard you say that if you were in the zone so have they zoned the state in a way that requires this application of the sand. So certain areas would have to apply it and other areas would not, is that correct?

A. Yes sir. What they have done is they have focused primarily in Los Angeles area, Southern California, because of the information they have there from the north ridge earthquake in 1994 and they've also focused in the area of Northern California that was affected by the 1989 Loma Prieta earthquake and they actually have maps where they've painted out zones, green zones or zones that could potentially have a liquefaction hazard and blue zones are zones that could have a landslide hazard. There's also a separate Act that looks at surface fault rupture effects. These zones are not supposed to be 100 percent encompassing. The idea that the state is shooting for is about 85 percent. They realise that, they don't want to paint the entire state green and blue the idea is that within these zones if there is a liquefaction event from an earthquake 85 percent of the observed liquefaction will be in these zones and then just because you're in a zone doesn't mean that you necessarily have a liquefaction hazard it just requires, says you have a more higher likelihood of liquefaction so if you're in those zones then you have to perform the site investigation and evaluate that hazard and the entire state has not been zoned yet. They have started in Southern California then they've added Northern California so that the more urbanised areas are zoned but much of the state hasn't been evaluated yet. It's an ongoing process that will take decades.

Q. The one other matter that interested me is your comment, observation, around the alpine fault earthquake and the likelihood of that to cause liquefaction and you noted that you were satisfied with the approach within the report but suggested more work be done. Could you just amplify a little on what more work you are contemplating.

A. I think that the basis of that was on the observations from these two earthquakes as well as some of the other earthquakes that have occurred recently in the series and you have an ongoing study to do site investigation work with the cone penetration test and standard penetration testing (inaudible 12.19.48) so you're collecting information that obviously once you get that information you can take advantage of that and do a more comprehensive study to look at the effects of the alpine fault and so I think an updated seismic hazard study that brings in the most recent information as well as bringing in all this amount of geotechnical data that you're collecting over the last year after these earthquakes, any study that takes advantage of that more recent data will be able to go beyond this preliminary assessment.

Q. I interpret that you're comfortable with the Seed and Idriss classification technique suggesting the levels at which liquefaction might occur and you're suggesting more specific work on the actual soil characteristics in the Christchurch area to apply to that method of assessment.

A. I am very comfortable with the Seed and Idriss simplified method. It's been around for a number of decades now, it's gone through several different developments and it's based on a significant amount of data and especially for the soils that you have in Christchurch which the soils that I have seen and the soils that I've read about in the literature and the reports are largely non-plastic silts, fine sands, some sands and some fine gravels. These materials are the materials that are largely the databases developed of. It's on clean sands and some silty sands as well as some coarser materials and so it's the Youd et al method of 2001 which is based on Seed et al 1985 which is based on the Seed and Idriss 1971 relationship is well founded and well calibrated and so I think that technique for use in terms of the cone and the SPT procedure is solid for the soils that you're looking at and I think that the updates by Seed et al 2003 as well as by Idriss and Boulanger 2008 provide even a stronger foothold in that area so I think the key aspect is understanding the ground shaking which an updated seismic hazard assessment can bring that in and then most importantly is the ground conditions and all this work that's being done and where their putting in cone penetration testing just about at each intersection within the CBD that offers an incredible opportunity to really do a refined zonation of the city of Christchurch and the surrounding areas. So it's the data I think that's most important to be able to extend the assessment of what might happen with the alpine fault.

JUSTICE COOPER TO ASSOCIATE PROFESSOR CUBRINOVSKI:

Associate Professor Cubrinovski are there any questions that you would like to raise or points of discussion that you would like to embark on with Professor Bray?

ASSOCIATE PROFESSOR CUBRINOVSKI TO PROFESSOR BRAY:

Q. Well good afternoon Jonathan.

A. Good afternoon.

Q. I don't have any questions I just want to thank you Jonathan for the positive review and for the review comments that you have just elaborated on.

ASSOCIATE PROFESSOR CUBINOVSKI:

I will briefly go to each of the points that Jonathan mentioned. The first one was with respect to finding some additional information about CBD tests and data within CBD and as you are actually aware we've got plenty of data and even in today's presentation I've showed some CBD results from the Kilmore Street so this is definitely something that we will easily include in the report and provide further documentation where that kind of data is going to be available for further analysis and interpretation. I certainly agree that CBDs are providing great information that we should be using on a consistent basis. In terms of the definition of important structures I agree that most of the structures within CBD are important. I intentionally didn't put the definition because I thought that is more than a purely subjective definition it is really a definition for society as a whole and engineers are certainly significant profession in defining what is important or not but I wouldn't, I didn't want to pre-empt what is important and what is not important and for different people maybe different things are important but I agree with you that most of the buildings within CBD are going to be really important structures. Going on the next one I completely agree with you that we have to look into the performance in terms of building performance and the consequences of those performances, not only for an isolated structure but for the city as a whole or for the community. In that regard we do have a very brief comment in our report that basically the current design philosophy of design of buildings is really focusing on the performance of the single individual building and is not really considering the potential consequences when we do have effect on a large number of buildings and as it was the case with Christchurch where the CBD was severely affected to a point where we practically lost the CBD for a quite significant number of months up to this point. So this global impact and the impact in terms of behaviour of the city as a system is something that we really have to recognise and reconsider the building code in that regard. Maybe we are providing what is needed for a single building, that may change as well but certainly we have to consider what is the more global impact in terms of the city as a whole and within the same context the utilities and lifelines are certainly a very significant component. Fortunately myself and Ian are actually involved in a project with the Christchurch City Council looking at the impacts of liquefaction on lifelines and, as I've mentioned in the introduction, we do have a lot of information that is not included in the report. Certainly we do have the information on the performance of the lifelines, in particular the water system and waste water system and I completely agree that this should be looked upon as one integrated or a set of integrated components that's really created the city and the life of the city so in that sense we have to again consider now not only the building stock but also utilities and lifelines in particular. The next your strong support in terms of site specific investigations and you're belief that geotechnical engineers can really contribute to greater resilience and better performance.

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I one hundred percent agree with that and I thank you for the support in that regard because that is one of the key recommendations that actually I would like to highlight coming out of our report. And finally I also think your information about the seismic hazard mapping practice is quite important especially because I see in New Zealand if I can briefly summarise the professions dealing with earthquakes we have seismic hazard specialists and geologists and we have structural engineering community and we have geotechnical engineering community and if I would like any of those three to get more strength I would say that is the geotechnical communities so what you are proposing here is maybe one way of providing that kind of support to the community in order to bring forward some really important issues that are going to increase the resilience through planning and better design and execution of engineering projects. And finally on the simplified procedure I also adopt this procedure as something that is providing very insightful and useful information on liquefaction. I would always use it and I would always be aware what it does provide, what is the background of it, and I would hope that engineers are going to apply that kind of scrutiny because if properly applied I really think it provides great information for designing considerations. I would also say that further attempts going beyond the simplified methods is something that I would like to do as a sort of complimentary effort that is going to better explain the outcomes and provide even more in-depth information about the performance of ground and structures so all in all I would say that I completely agree with your comments and the intention was to reflect those in the report and maybe some of those are not strongly emphasised for different reasons. Thank you.

JUSTICE COOPER TO MR MCCAHON:

Q. Mr McCahon is there anything that you would like to say?

A. Very little really, just that some of the points you make have been going on and are going on in Christchurch for the last 20 years but the earthquake, of course, has brought this together and my hope is that some of these strands will be brought together in a more coherent fashion. For instance the zoning arrangement in the Seismic Hazard Mapping Act the Regional Council here in Canterbury has done a similar zoning of the whole of the Canterbury Region as part of their hazards requirements so the data is there, the zoning is there but it needs perhaps some instrument to bring that to the fore and to be used more comprehensively. But other than that I don't think I have anything too significant to say. Thank you.

JUSTICE COOPERTO PROFESSOR BRAY:

Q. Professor Bray, it seems that this largely agreement between you and those who are advising us in New Zealand, is there anything further that occurs to you to say?
A. I think that the challenges of earthquakes are well recognised in New Zealand. Some of the top earthquake engineers in the world are from New Zealand, they practice in New Zealand, I've interacted there over a decade and have been impressed both with the professors and the practitioners, you have the talent and they recognise it. Your GNS is well recognised world-wide in the structural engineering community and the geotechnical engineers that I've worked with and so we just had a mention of the data, that the data for zoning is there and so you have all the components and I think in many respects the same thing could be said about California in the late 80s. We had all the components. We had great universities and great practitioners and we had a lot of data. We just hadn't put it into policy. We hadn't been able to convince the politicians to enact a law that would require some minimum level of investigation in zones that we thought might have issues in terms of liquefaction and land sliding and in fact a bit of history is after the '71 San Fernando earthquake in the Los Angeles area there we actually an Act that was to go into practice that was going to look at four hazards – surface fault rupture, ground shaking, liquefaction and land sliding. As this Act started going forward the developers and the lobbyists started hitting away at it and they took around ground shaking. How can you say that this one building on this side of the street is in the ground shaking impact zone and this other building on the other side of the street is not and then, well, you know, liquefaction do we really know enough about liquefaction so by the time it had gone through that process the only thing that came through was surface fault rupture and so the El Questro Act 1972 was enacted. It said we are going to allow you to build right on top of an active fault because we actually had developments that had been built right on top of the San Andreas Fault, the equivalent to your Alpine Fault and liquefaction and land sliding

were put on the back burner and it wasn't under the leadership of Lloyd Clough, and unfortunately an earthquake, a very damaging deadly earthquake in 1989 were we then able to take something that we tried to get done after '71 and get it actually implemented into law and it did get implemented and it took some time to get it going but now it's been in practice for about a decade and a bit and it has really improved the state of practice and so if you have the zonation, you have the tools and you're collecting all that information from this post earthquake investigation you have a great opportunity to enact some legislation that would require some basic level of site investigation to allow us to address this issue in a very reasonable and really not a costly manner. It's when you (inaudible 12:33:54) the total cost I think some site investigations it pays off in terms of improving the resilience of a city.

JUSTICE COOPER:
Professor Bray, thank you very much for the advice that you've given us which we are very impressed by and I think unless anybody has anything further to say we will bring this part of our hearing to a close. Thank you very much.

Can I just say that we are very grateful to Richard Bishop from Asnett Technologies for the smooth way in which the videolinks have been conducted.

COMMISSION ADJOURNS: 12:35 PM

COMMISSION RESUMES: 12.39 PM

ASSOCIATE PROFESSOR CUBRINOVSKI:
Thank you, so I'm going to continue with –

JUSTICE COOPER:
Q. I wonder if you could just go back to the previous slide before you go any further?
A. Yes.
Q. That diagram on the –
A. Yes.
Q. Right-hand side, the red line indicates the foundation depth?
A. Yes.
Q. In the before liquefaction situation?
A. Yes.
Q. What are we to infer by the position of the red line in the right-hand side post liquefaction settlement –
A. Okay.
Q. – situation, I mean this is probably obvious to everybody else in the room but I'll ask the question. When the building settles does it take the foundations with it, does it push the foundations further into the ground?
A. Yes, so the red line is referenced to the ground surface.
Q. Yes.
A. And if it is let's say 50 centimetres depth of embedment in the deformed ground after liquefaction that line still sits at 50 centimetres from the ground surface post liquefaction ground surface.
Q. Oh I see so it matches, it goes down to the same extent?
A. Yes.
Q. As the ground has gone down?
A. Yes. But now what we see is that the building relative to that red line is actually went down, settled so that indicates that in addition to the

ground settlement there is sinking of the building in the ground so there are, the absolute movement of the building is going to have two components because of the overall settlement of the ground.

Q. Yes.

A. In addition to that because of the sinking of the building into the ground.

Q. And I take it, well is it correct that in some cases the settlement of the building and the ground is accompanied by the adverse effects on the foundations themselves, would there sometimes be in other words physical degradation of the foundations themselves?

A. Definitely. And especially so we are distinguishing between two types of settlements, total or global settlements and differential settlements.

Q. Mhm.

A. So differential settlements are particularly damaging and they always go with some sort of strains and stresses in the foundation that may cause significant damage beyond point of yielding it which is a quite large deformation, so I would say that these differential movements are always, always more problematic than, than global movements. If everything moves together and a structure moves as a rigid body there is not much additional stresses in the building or in the super structure because of the foundation movement. However, once these movements are different then immediately this is creating strains, stresses in the foundation itself and then these are propagating also upwards in the superstructure.

Q. Yes. All right thank you

ASSOCIATE PROFESSOR CUBRINOVSKI:

Next slide please. This is another illustration of the performance of high-rise buildings within the CBD. The building to the right is on shallow foundations. The building to the left is on a hybrid foundation, part of it is shallow foundation and beneath part of the building there are parts. Now there are two important aspects here. The first is when we having building on a hybrid foundation where obviously we are having shallow and deep foundations combined it is very difficult to anticipate what will be the performance of this foundation during the strong shaking. As I mentioned before liquefaction or ground response is developing spatially and temporarily throughout the foundation soil so different parts of the soil are going to attract different deformations and forces and in addition because of the hybrid foundation the loads are going to be transferred to this soil from the building in a very non-uniform fashion which is even difficult to anticipate during strong ground shaking so the interaction between the soil and the building is such that it is difficult to control it. This is similar to the performance of buildings themselves, I mean it has been mentioned that regular buildings have performed better than irregular buildings in terms of the distribution of mass and stiffness. We can say the same thing for foundations and for underlying soils. So hybrid foundations are stimulating non-uniform response and are attracting forces and deformation at particular components whereas others may not be contributing much. The overall goal is to have a foundation that works as a unit and redistributed stresses so that we get more uniform deformation and better performance so in that sense hybrid foundations are very challenging for the design to achieve that kind of performance because we don’t really understand well the load itself. In this situation one additional important aspect is that we have two adjacent buildings high-rise buildings and one is on shallow, the other on deep foundations and intuitively you can expect that the foundations are going to work in a very different way because of deep and shallow foundations and also because the buildings are going to have different vibrations so any response of the building is going to affect the soil and this soil is going to affect the adjacent building as well so we have structure soil, structure interactions with these adjacent buildings. If you’re designing buildings for very strong earthquakes and sitting on relatively soft alluvial soils then even this kind of interaction should be considered and we have to anticipate what does it actually mean for the other structure that we are designing. How is the adjacent structure going to modify the response and affect our foundation? And think about that in the design. So –

JUSTICE COOPER:

Q. Can you just tell me the addresses of these two buildings please?

A. This is again, this is Victoria Square on Armagh Street.
Q. That's the building on the left I think isn't it?
A. Both buildings are on Armagh Street.
Q. Both are on Armagh Street?
A. Yes.
Q. The one on the left is the building known as Victoria Square?
A. Yes. Victoria Square yes.

ASSOCIATE PROFESSOR CUBRINOVSKI:
So quite often when we've got two buildings of like size we are seeing that one of those is leaning towards the other because we can see that the middle part beneath these two buildings the part of the soil that is having the largest stresses and enlarges the deformation so that kind of tilt for a building on shallow foundation is intuitive. Next slide please. These are just two solo profiles beneath the footprint of the single building illustrating that the variability in the certification so the gravel layer of three metres on the left-hand side is actually eight metres thick so even between the footprint of the single building there is significant difference in the thicknesses of the layers and obviously each of these soil layers are going to respond during an earthquake in a different manner affecting different deformations so this by itself is going to generate some differential settlements of movements. In the foundation hybrid and the irregularity in the structure is present then we are going to have additional differential movements and once the differential movements are going to initiate then there is bias in the load so you are attracting more and more movement and deformation towards that bias direction. Next slide please. This is a building on piled foundations. I would say we can put those in two categories. Buildings on pile foundations that reach strong and competent layers at the end of the piles and buildings on pile foundations where the tip of the piles or the end of the piles is really sitting in some medium dense layer and in some cases even a layer that liquefied. In the first case when the piles are reaching competent soils this differential movements were much less than in the case of shallow foundations. This is a building on Kilmore Street and we can see that on one end, that is the north

side, the settlement of the ground relative to the building was about 30 centimetres whereas on the south side it was 17 centimetres.

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By the way the CBD data that I was presenting, presented before is from this site and, as you can see, at the bottom right corner the pile foundations are connected with strong beams connecting the top of the piles so all of this is creating some relatively rigid foundation which is transferring the loads at the end of the piles and for that reason the overall movement and differential settlement of the building was not as significant. Since the building is not moving vertically the surrounding soil that liquefied settles relative to the building and that settlement was 30 centimetres after the February earthquake. I've measured it after the June earthquake and it was 50 centimetres. So the June event created additional 20 centimetres of settlement at this particular site so overall movement is half a metre at least.

JUSTICE COOPER:

Q. But if 50 centimetres would be where the 30 centimetres is currently shown.

A. Yes and that was the total settlement or the settlement at that point after the 13 June event. Actually there were a couple of earthquakes which is quite important. On 13th of June the first one was 5.5 and that probably generated some more pressures and liquefaction and then 80 minutes later we've got the second one so there is a cumulative effect of these two earthquakes for liquefaction and that is why we are seeing some quite large settlements as a combination of these three events, especially in areas of liquefaction.

COMMISSIONER CARTER:

Q. (inaudible 12.50.58) foundations reached?

A. These piles are, I think, approximately at about 15 metres depth and as you will see from the CBD profiles the depth of that stiffer layer is variable so ideally during construction you're trying to identify whether you've got to that point or not and the piles are going to be maybe a metre or so variable in terms of length depending on the exact position of the stiffer layer or denser layer at depth.

ASSOCIATE PROFESSOR CUBINOVSKI:

We've measured the lateral spreading displacements within the CBD so this is one diagram showing a summary of spreading displacements at nine transects so on the vertical axis we have lateral ground movement and on the horizontal axis is distance from Avon River. In general the spreading displacements within CBD were on the order of 10 to 30 centimetres and there were a few locations where we've got larger displacements of about 50 to 70 centimetres. They are much smaller than the lateral spreading displacements along the Avon River that went in the eastern suburbs which were quite often a metre or two. So I would say limited area within CBD was affected by spreading and in those cases we can see that the spreading displacements propagate about up to 50 or 100 metres, 120 metres in some cases from the waterway so any building sitting within those 50 to 100 metres distance from the river is going to be subjected to some sort of stretching of the foundation and these forces can be quite large and pile foundations are going to be pushed toward the river due to this lateral spreading movement. Now, bearing in mind that in addition to this lateral movement due to spreading we still have this cyclic phase of the deformation during the shaking as well as vibration of the superstructure. So we are having combined effects of cyclic, shaking as well as spreading. This might be happening at the same time or there might be slight delay in the spreading depending on how quickly the soils have liquefied.

JUSTICE COOPER:

Q. Those sites that are identified as CBD one through to nine is it possible for you to tell us or let us have the locations of those sites?

A. Well I am very happy to provide additional information with exact location of those transects because they are basically along Avon River, maybe starting from the Colombo Street bridge going towards east and it would go as far as the Fitzgerald bridge so I will provide the exact location of those transects.

Q. Well I think that would be helpful wouldn't it. Yes, yes if you would please.

A. Sir.

ASSOCIATE PROFESSOR CUBINOVSKI:

Well summary on the CBD building foundations. Well basically buildings on shallow foundations supported on loose to medium dense soils that liquefied suffered significant differential settlements and residual tilts whereas the pile supported structures, in cases when the piles reached the deeper competent soils, suffered less differential movements and impacts of the foundation performance on the superstructure. There were a number of buildings, high rise buildings, on shallow foundations sitting on shallow gravels and they showed mixed performance. Something that I didn't mention in detail is that because the soils are highly variable quite often buildings are sitting in transition zones where the soils are changing from bad towards much better soils and in those cases obviously the ground supporting the structure is very different beneath different parts of the building. The thickness of the gravelly layer might be changing dramatically over the footprint of the building so all of this is contributing for larger deformation that you would have if the soils are highly uniform. So this is, I think, one of the significant contributing factors for seeing some deformation and differential movements of buildings sitting on shallow gravels. Hybrid foundations, as I pointed out, performed relatively poorly and that was expected to be the case. We have identified the zones of ground weakness throughout, especially the portion that liquefied. Parts of that area that liquefied is showing clear evidence of larger ground deformation, distortion and with really very severe effects of liquefaction we have to account for in future considerations and, finally, we have quantified the lateral spreading which is isolated and limited to areas in some parts along the Avon River and in those areas they have affected a number of structures both on shallow and pile foundations.

JUSTICE COOPER:

Q. Can I ask how many, approximately how many buildings have you been able to study for foundation performance in the CBD?

A. I think we have probably studied in greater detail about 10 to 12 buildings within the liquefied zone.

Q. Yes.

A. And we have covered to a lesser extent another 20 buildings or so. We do have preliminary reconnaissance report on those buildings that we are just about to complete so I'm happy once we complete that report to pass it for your interest.

Q. Well what's your time frame?

A. Well I certainly hope that by the end of the year we are going to have that report in terms of this is a report summarising our building inspections with particular emphasis on geotechnical aspects so with greater detail about ground deformation around buildings and measurement of tilts and description of the behaviour.

Q. Is that work that you are carrying out for an organisation or is this just academic research.

A. No this is, this is academic research with the (inaudible 12.57.43) students and I have a number of projects and one of those projects is focusing on CBD buildings and the impacts of liquefaction on CBD buildings and we do have also very comprehensive site investigation and lower artery testing programme which is maybe a group of three or four PhD students are working in areas related to CBD.

Q. All right, thank you. You're going to go on to discuss the liquefaction impacts from an alpine fault earthquake is that so?

A. Yes.

Q. Would this, this would perhaps be a logical point for us to stop for the luncheon adjournment.

A. Yes I think that would be

COMMISSIONER CARTER:

Q. (inaudible 12.58.26) I'm just interested in the question of driven pile. We've been informed that there has been a reluctance to use driven piles in central Christchurch because of the disturbance that they can give, both noise and vibration to adjoining buildings. I'd be interested in your comment on that but also do driven piles driven to some recognised driving record still become subject to liquefaction or if you're getting a pile driven well into a supporting material does that generally indicate adequate resistance to liquefaction in that layer?

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A. Well I will answer the second question first. Definitely if the pile tip is reaching the competent end bearing stratum then, which is not going to liquefy, that obviously is going to minimise the differential movements in terms of settlements and vertical movement of the foundation. The piles are still going to attract a lot of deformation if the surrounding soils are going to liquefy. Those deformations are quite significant. We've seen a number of piles damaged in the Kobe earthquake during the very strong ground shaking, similar to what we've got here, and some of those piles were beyond repair but many were not so actually performed reasonably well. So they attracted damaged but still preserved the vertical carrying capacity. So, in that sense, well designed piles that reach competent stratum at depth are going to perform well. In terms of the specific construction practices I would really invite Ian McCahon to answer that question because he would give you much better answer and information about those practices rather than me guessing or –

JUSTICE COOPER:

Yes he might have been the source of the information that we start with.

COMMISSIONER CARTER:

Q. Just a little bit of amplification on the likelihood of liquefaction at depth even if the characteristics of the soil are suggested could be liquefiable can we carry on getting liquefaction at considerable depth?

A. Yes liquefaction in the profession generally is accepted to be developing up to 20 metres depth. So there is no question, so if you're doing investigations of liquefaction, potential and (inaudible 13.01.10) you should cover at least 20 metres depth. For some structures like earth dams actually it could be much deeper because the body itself is producing a response which is quite different but if we are discussing native soils or pre-filled level ground conditions then we would go at least up to 20 metres depth. Now the severity of ground shaking is going to influence what will be the thickness of the liquefied layer. In the case of the Kobe earthquake in which most of the liquefaction developed in artificial islands, very plain soils where the deposits were 20 metres thick, they liquefied up to 20 metres depth. The piles there were going up to 40 metres depth to reach the competent layer so it was very deep. The buildings performed exceptionally well. The island did completely liquefied and settled for 30/40 centimetres. High rise buildings of 20, 30, 40 stories performed well and they are still there and the piles performed well. So in that sense you can design sometimes you really have to go deep to find the layer that is competent enough and the piles are going to be significant in dimension but it can be achieved.

COMMISSION ADJOURNS: 1.02 PM

COMMISSION RESUMES: 2.16 PM

ASSOCIATE PROFESSOR CUBRINOVSKI:

So I think I have a couple of topics to cover, the first one is the effects of liquefaction of the potential Alpine fault earthquake. This was also one of the requirements and topics to address. May I have the next slide please? First of all I would like to say that what we are showing here is a preliminary result based on a simplified analysis only and we have stated basically in the report that this applies to triggering of liquefaction only and should be restricted to such use so this is no [*sic*] any attempt to indicate what will happen to different types of structures during an Alpine fault event but specifically what will the level of triggering of liquefaction by an Alpine fault event. I am going to describe now the approach taken. In general ground shaking is described by three parameters for engineering purposes and one is the amplitude of shaking which shows that the magnitude of movement whether displacement, velocity or acceleration. Another is duration and I say duration here we engineers think of duration of significant cycles or significant shaking. We are ignoring the small bits of shaking at the beginning and at the end and finally is the frequency content. What kind of frequency predominate in the ground motion, is it the short frequency or long frequency and what are their amplitudes? If we consider the behaviour of a given soil element at the given depth then two of those parameters are critical whether liquefaction is going to develop or not and that is the amplitude of shaking and the number of significant cycles. We use the peak ground acceleration or PGA as a measure for the amplitude and we use the magnitude as a proxy for the duration or number of significant cycles. With those two then we can calculate and estimate the potential for liquefaction and triggering of liquefaction in particular. So the concept is summarised on this slide and to the left you're seeing the correlation between the number of significant cycles and the earthquake magnitude as defined by Seed and Idriss. The first thing I would like to point out that there is a line passing through this point and this is just an average relationship. Actually if you plot many earthquakes you are going to find that there is significant scatter between different earthquakes and this relationship. But anyway there is a clear decay suggesting that if we have a magnitude 8 earthquake then we're expecting to have about 20 to 22 significant cycles of shaking whereas if you have a magnitude 6 earthquake we are going to have only five cycles so there is clear correlation between the size of the earthquake in terms of magnitude and the number of significant cycles that will be in the ground motion. And then to the right I have two time histories of now simplified interpretation of what would be magnitude 8 event with 22 cycles and let's say and peak ground acceleration of .1 g. And then below we've got a magnitude 6.3 which has only seven cycles but a very high acceleration. The reason why I have got large acceleration in the second case is because now the source is much closer to the site so we're getting very high PGAs whereas at the top figure magnitude 8 is let's say very distant earthquake 100, 150 kilometres from the site so that is why we have lower amplitude. In essence what I am trying to compare here is a hypothetical magnitude 8 Alpine fault earthquake happening some 130 kilometres or further away from Christchurch with effects of the recent earthquakes in particular the February earthquake of 22nd where the magnitude 6.2 was recorded. Next slide please. So what we did was we used the estimates from calculations for the ground motion that will be caused by an Alpine fault event first at bedrock level and those peak ground accelerations according to the GNS analysis are quite low, .02 g to .04 g which is just 2 to 4% of rapid acceleration. Next, in some of their analysis they do have, they have included the effect of site amplification, site effects for deep soils but anyway in our case what I did was if that was .04, .05 we multiplied those by a factor of two to account for amplification effects so we came to .1 g, so .05 was bedrock multiplied by two, we got to .1 and then we applied plus minus one standard deviation which is multiplying by 1.7 or dividing by 1.7 so we have got .17, .06 to .17 g as expected range of PGAs in an Alpine fault event and we are covering for different effects like site amplification, basin effects in a very approximative way and so that is the – at the right-hand corner of the plot, the blue diagram and the arrows are indicating the lower limit and the upper limit of PGA predicted for an Alpine fault event. The yellow band on the other hand is showing the equivalent PGAs from the February earthquake and we can see they're way above the Alpine fault PGAs. So for in order for the Alpine fault event to have similar triggering level of liquefaction and effects of liquefaction the blue part in the diagram will have to be comparable with the yellow one or of similar PGAs. We clearly see that the PGAs triggered by an Alpine fault event are predicted to be below the February event so based on this reasoning we can say that the simplified analysis is suggesting that the level of triggering of liquefaction is going to be well below the 22nd February event and based – yes.

COMMISSIONER FENWICK:

Q. Just a point of clarification. You've taken the Alpine fault plus or minus the standard deviation but you're using the actual measurements from the Christchurch earthquakes is that correct?
A. For the yellow one the yellow one is based on the actual measurements from the 22nd February earthquake.
Q. The peak values you recorded in the four different sites or…?
A. Yes. Four different sites within CBD so this strictly applies to the CBD alone.
Q. Thank you.

ASSOCIATE PROFESSOR CUBRINOVSKI:

If we did the similar analysis for the Darfield event from 4th of September and we can see that that is indicated by the green area and arrow we can see that again the Darfield event the area triggered the liquefaction area is going to be slightly larger than expected in an Alpine fault event so I would say that the Alpine fault event is going to be similar or lesser event than Darfield event in terms of liquefaction trigger. Now having said this I would make a couple of important comments here. One is that since the Alpine fault event is magnitude 8 we are going to have very long duration okay so the 22 cycles is a proxy to represent that long duration. When you have that kind of long duration it is much easier for structures to get into the resonance mode and that is to say you are going to excite particular periods of vibration within the structure which is going to amplify their response and this has been observed in several earthquakes and I'm going to point out two significant events. The first one is 1985 Mexico City which was affected by an earthquake 320 kilometres away. The ground motion at the base rock was very low, .04 g or something like that. That was then significantly amplified through very soft clay deposits and the amplification happened to be in the predominant period of high-rise buildings, 10 to 20 storeys so there was double resonance effect through the soil and then through the building and there were many collapses so this is where the long duration is really important for engineering structures because there is long enough time for structures to enter into a specific mode of vibration which is going to bring large forces and problems.

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The second important point when we are comparing the 22nd February and the Alpine Fault event is that because it is far away source of large magnitude it is going to be probably quite reaching long period motion. This long period motion, unlike the short period one, is going to excite larger depth of the deposit so if you like to think in terms of Christchurch maybe in February the top five, seven, eight metres moved back and forth as a body. In the case of an Alpine Fault event this is going to go deeper so we may get 15-20 metres of deposit moving in the same period so that means that the shearer stresses deeper in the ground are going to be larger so in relative terms that kind of event is going to induce larger response in the deeper layers as compared to the shallower layers when we compare the 22nd February event and the Alpine Fault event. So having these two in mind is a good way of addressing the additional analysis on top of what we are seeing here based on a simplified analysis. I am a strong proponent of using whatever is available in terms of advanced analysis to understand better what is going on so in addition to the simplified analysis that we discussed during the discussion with the reviewer, I think we are going to do also some advanced analysis. We do have models that can simulate liquefaction in a very sophisticated dynamic analysis and see how Christchurch soils are going to respond to that kind of event, what kind of complications we are going to get and then in the next step how is this going to affect the different buildings and I think it is good to also conduct that kind of more advanced and sophisticated analysis at least to understand what will be the impact of this kind of features of ground motion coming from an Alpine Fault event far away but which is different in character. So on top of the simplified analysis I'm advocating to do some additional analysis, advanced analysis, just to get a better understanding. I'm not suggesting that this should be part of the practice but part of the understanding of what does this mean for structures so that would be my recommendation in that regard.

COMMISSIONER CARTER:

Q. When might you be able to go through with this added work?

A. That is a difficult question to answer having in mind all the tasks that we've got but at least in some preliminary form we can have that in the next three or four months I hope, completed, which is going to be indicative of type of ground response that we will be getting from that kind of event.

COMMISSIONER FENWICK:

Q. This is an issue which has come up before with the seismicity section. We didn't get an answer then but I think sounds as though you can supply, the response spectrum at the moment has that bump –

A. Yes.

Q. - between two seconds and three seconds which is unfortunate because sort of 18 storey buildings can drift into it –

A. Yes.

Q. – and excite. Now the question I had before was with the Alpine Fault would you expect, because of its longer duration, would you expect that relative increase in the long frequency to give you a very much higher response in that sort of two, three second period than you'd get, say, that we saw in the September earthquake?

A. Well I would say that increase in the response spectrum at longer periods can have many contributing factors and one contributing factor can be the source itself. I mean the way the fault ruptures it may generate that kind of long period motion. There is no question though that if you have a long duration earthquake with presence of those components that there is significant time through which soil can amplify that response so I would say that in terms of site amplification and site effects probably that part is going to be amplified in the Christchurch soils. Now that amplification comes at least through two sources when we discuss site amplification – one is the deep gravelly deposits and the other is the top soils of 20 metres or so. It may happen that both are amplifying the ground motion within the same range of period because they have similar predominant periods. This one because it is very thick – three, four, 500 metres, and this one because it is very soft and it may even liquefy so both may amplify that particular part of the spectrum. I have checked the records for this feature and it is interesting that you don't see that bump in all the records which makes things complicated. If we see it everywhere we can see well this is source effect and maybe the gravels effect but actually in some records you see it, in others you don't so we need more systematic analysis in order to figure out what are the factors really contributing for this elevated part in the spectra.

Q. Now the GNS analysis they took the worst case, as I understood it, to get the highest accelerations which was the fault starting to fail in the south and travelling north so that you've got reinforcement but now of course if you zip it the other way I thought that would be better but it's not is it because you've got a longer duration earthquake, you may get slightly less shaking but now you'd increase the duration. Would that be correct?

A. Yes so it's really the combination of the two – of the amplitude and the duration and what I've said here with 22 cycles that is just one approximation so this is why I think we need, in the simplified analysis, to go with some parametric variation to cover for uncertainties and then on top of that maybe conduct a more sophisticated dynamic analysis. Basin effects and all those discussions that you've been dealing with the last week are going to come into the picture again and they are going to again influence those ground motions but now in different ways because they are propagating far away from here and incoming from different directions so I would just say that this is still an important issue to address. In terms of triggering, simplify the analysis, is suggesting that it should be not close to what we have experienced. My personal experience is supporting that kind of outcome. I'm happy with what I'm seeing there but I still think there are a few more things to check and especially to check how different structures are going to respond to this kind of earthquake. The second event that I would like to mention is the 2003 Tokachi-oki earthquake that occurred in Japan and I was on a reconnaissance mission of that earthquake. In that event an earthquake about 330 kilometres away from Sapporo affected the tank farm of oil tanks. Very low ground motions were amplified and sloshing of the oil which has very long periods of four or five seconds, not very far from the periods we are discussing here, ignited fire and they've lost two of the tanks and it was actually critical to lose a huge oil tank farm so this is another example where large earthquakes producing very low amplitude motions at the base level got amplified through the site effects and created significant motions and even inflicted damage to structures so I would think that this has to be checked. I don't think it is going to be critical. Everything is pointing out that in general it will be much better. I would see some cases that will be worse and this should be checked.

ASSOCIATE PROFESSOR CUBINOVSKI:
Next slide please. I am just going to briefly conclude now with the typical foundation methods that we are recommending and it is just a brief summary in two slides so first we realise that there is significant uncertainties both in the hazard which was dealt with last week in these hearings and in the soil behaviour. Fortunately enough for the soil behaviour if we have a good investigation programme, good analysis and interpretation we can reduce those uncertainties to a large extent and that is why it is so important to really understand the soil composition, the stratification, the in situ state of the soils and how are they going to respond during an earthquake, whether liquefaction

is going to develop. If yes, what kind of deformations are going to be caused by the liquefaction and how all of this is going to affect structures. So in this process if we are dealing with important which would be any building I think, especially if it is three storey or more, we would like to understand how the system behaves and when I say 'system' I think of the building, foundations and soil how this system is going to respond during the earthquake because that is when structural engineers are going to really understand what is the contribution of ground and foundations in terms of forces, loads and behaviour of the superstructure and I think this exercise is particularly important when you are dealing with soft soils because, in that case, the effects are the largest, the effects of the soils.

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And, finally, this obviously requires site specific investigations and design which is going to be structural specific as well. I'm not advocating here and now unreasonable numbers of field tests but certainly a decent number of tests which is going to be variable and different depending on the particular site conditions, dependent on our level of understanding, if the uncertainties are bigger and consequences of failure or poor performance are greater then obviously we will have to conduct in depth and detailed investigations followed by appropriate analysis and interpretation. I would like to emphasise the analysis and interpretation here because having the results is not enough. You have to make good sense out of those results and discuss it with the structural engineer and understand how the system is going to behave and then really you will provide the critical feedback and input from this process to the designer. That is what I think is critical in this exercise. The final slide please. So in order to achieve improved performance we are not recommending any particular type of foundation. I think we can achieve good performance both with shallow and deep foundations. For shallow foundations, since the soils are quite weak, quite often we will have to do some ground improvement simply to reduce the impacts of liquefaction and pore water pressures in terms of stiffness and strength degradation of soils and the foundations will have to be robust, they will have to be strong and stiff enough to work as a unit so that we minimise differential movements and negative effects of the foundation, of soils in the foundation on the superstructure. For deep pile foundations it is really critical to reach competent ground at a depth because what we understand in deep soils we should not rely on frictional resistance and shock resistance because that will be lost quickly during the strong earthquake so you really really rely on very good end bearing resistance of the piles. So that is why it is critical to reach those. For both foundations details are extremely important. Good connection and strong foundations at the top that are ensuring the foundation to work as a unit as (inaudible 14.38.43) individual foundation members that have certain level of ductility. Piles, even when damaged, they have to preserve the vertical carrying capacity, that is critical and there are those kind of a number of detailed requirements that we have to satisfy. All this is basically calling for a site specific and structural specific considerations. The level of detail is going to be different depending on the importance of the structure, depending on the particular site conditions and, finally, as I mentioned, we need to integrate the soil foundation and structural considerations and have this as a system in order to see the interaction between different components sometimes you have even to (inaudible 14.39.29) the interaction with the adjacent structures of significant size and we anticipate that they are going to change the way our structure is going to respond. With this I would like to conclude my presentation. Thank you.

JUSTICE COOPER:

Q. Can I just ask what kinds of activities are included within the expression 'ground improvement', your point one there?

A. Well these are different fill methods trying basically to increase the stiffness and strength of the soil. Those are methods that are going to densify the soils by a range of densification methods. They can solidify the soil by injecting cement or other binding agents that are contributing to stiffness in different ways, by building walls in the ground and connecting the network of walls which is going to confine the soil, so even if that is liquefiable the walls are going to prevent the formation. In order for liquefaction to develop strength the deformation must happen in the ground. It will restrict that movement of deformation in the ground and pore pressures cannot build up, even when shaken, so that kind of confinement is going to increase the resistance to liquefaction. There are matters also based on improved drainage so we put drains so that pore pressure development is going to be much more difficult because once the pressures are going to get elevated the drain is going to provide an easy part for the water to dissipate the pore pressures. So there are a range of methods. Some are less effective for strong earthquakes others are more effective and, of course, effectiveness is quite often associated with cost as well.

Q. And are some of these techniques more expensive if they are adopted on a site by site basis rather than by an area by area basis where you can address the condition of a number of properties in a block say. Is it more expensive to carry out these measures on a site by site basis?

A. Well, in principle, on a site by site basis will be more expensive but then it depends on the size of the site and if we have a large building with a large footprint then basically that site alone is large enough to justify the cost of a certain level of improvement. Those methods have been applied and they work well. They certainly reduce the impacts of liquefaction. What we have to be aware is that by making the soil less deformable actually less of the energy is going to be dissipated in the soil, more of the energy is going to enter the building. So, in those cases, obviously the superstructure will have to be well designed and should use all sorts of dissipation mechanisms and damage control mechanisms to achieve the desired performance but what we will achieve in this case is that any differential movements, large movements of the foundation, are going to be reduced and limited and controlled to a certain extent.

Q. Have measures such as those you're discussing been adopted in the Christchurch CBD historically?

A. Not really. There are several sites in which a ground improvement have been implemented but I would say those were methods which are somewhere at the low end or mid range in terms of effectiveness and, of course, this is related with cost and this should also be associated with the hazard that was anticipated before this earthquake which was much lower than the earthquakes we have experienced. So, in that sense, to a limited extent there has been some improvement by methods that are more effective for moderate earthquakes than for very strong earthquakes.

DISCUSSION BETWEEN COMMMISSIONERS

COMMISSIONER FENWICK:

Q. You've talked a lot about liquefaction, the effect on buildings, you've also referred to fairly soft material in the upper levels. Are you aware of, I mean I'm aware that a lot of the buildings have small tilts in them which are probably acceptable. I gather even that some that Professor Pettinga measured came back straighter after the June earthquake. But are you aware of any serious problems that have occurred in the foundations where this liquefaction is not a feature?

A. Well I think that there are buildings that were affected by differential settlements even in areas that are not manifesting severe liquefaction or even moderate liquefaction and I think that wouldn't be surprising for the level of ground shaking produced by these earthquakes and the types of soils that we've got in the top 20 metres. We would expect these soils to deform under such strong shaking and given the variability of soils under small distances then this deformation is going to be highly non-uniform. So this in itself is going to contribute to some differential movements so any additional irregularities in the structure and in the mass of the structure and stiffness of the structure is going to additionally contribute to those kinds of movements so I wouldn't be surprised at all to see that kind of differential movements because the earthquakes were extremely strong, producing very strong shaking, and these soils are mostly compressible, more or less depending where the building is sitting on.

Q. When you design a foundation, I'm sorry this is probably more directed at your colleague, I take it you're given the design forces or the surfacibility limit state, the ultimate limit state and the capacity design actions on the foundations, do you have any oversight into those values, I mean they are just given to you is that right? Or do you have any involvement in their calculation?

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A. Well I don't know what is the exact practice but I would hope that there is some involvement of geotechnical engineer but in essence we are getting the loads from, from the structural engineer.

Q. Yes.

A. But then there might be an iteration where if we anticipate that the performance will be difficult to achieve acceptable performance that we may influence that the designer if possible for changing loads but in principle those are given to the designer.

MR IAN MCCAHON (ON FORMER AFFIRMATION)

COMMISSIONER FENWICK TO MR MCCAHON:

Q. Do you want me to repeat that or do you…?

A. One of the recommendations we've made is that there must be, there should be a much greater interaction between the structural and the geotechnical and my experience is that very often I don't actually know anything about the structural configuration or the loads and I have some idea because of experience in terms of what the loading might be on the structure, on the foundations but then I basically say here's a range of foundation sizes, these are the sorts of loads that they can sustain that goes back to the structural engineer and in 90% of cases that's the end of what I hear so that there's very little interaction between the geotechnical and the structural.

Q. I guess my main concern is I think you are dealing with soils and you know how variable they are. I'm not sure that structural engineers know how variable their structure is, how it behaves but perhaps ought to return to that one later on because I know that Kevin I think has a sort of question which leads into that. When we look at the foundations I happen to ask one geotechnical engineer about the factor of safety that the .9 of average material strength say assumed for over strength and I asked this geotechnical engineer, he said, "Oh that could be justified on the basis of the speed of loading", which surprised me because I would have thought that the speed of loading would have had a big effect on clays but I'm surprised if it had much effect on sands and gravels. Now would you like to comment on what influence does the speed of loading have on the strength of soils given that we're looking at two second period would this be, the loading have an appreciable influence there, on the behaviour of those soils?

ASSOCIATE PROFESSOR CUBRINOVSKI:

Well the rate of loading is an important issue and there is probably quite often quite often misunderstanding there. The rate of loading for certain soils for clays for example the speed of loading is going to increase the strength but as I have discussed here if we have poor pressure development obviously for sands the strength is going to be reduced. One consideration in the geotechnical and foundations engineering which is reducing this factor is the difference between a constant and continuing gravity load as opposed to a short term load because quite often we are using this pseudo-static analysis with the constant load then the consideration is that if you're applying dynamic load as a pseudo-static load then it should be reduced because the time of application of that load is limited. Well this is general consideration for a pseudo-static analysis and for certain members if think that logic is relevant but for varying capacities and differential settlements it's certainly not relevant so maybe we're sometimes mixing things there and some improvement is definitely needed.

COMMISSIONER FENWICK TO ASSOCIATE PROFESSOR CUBRINOVSKI:

Q. I take it then that you would say that the speed of loading in terms of settlement and the seismic oscillation would not have a big effect for gravels and sands.

A. Well it will have negative effect but if pore pressures are induced it will have negative effect -

Q. Yes.

A. – to the loading because under static conditions those pressures are not generated by a static load.

Q. Yes.

A. But by an oscillating load elevated pore pressures will be generated.

JUSTICE COOPER TO ASSOCIATE PROFESSOR CUBRINOVSKI:

Q. I'm sorry what's a negative effect? It reduces does it, what do you mean by negative effect?

A. Okay, negative effect it will increase the deformation of the soil and the foundation so settlements will increase so the impact on the foundation performance will be negative.

COMMISSIONER FENWICK TO ASSOCIATE PROFESSOR CUBRINOVSKI:

Q. Just leading on from that the vertical excitation of course is very high frequency and that's something of course which will have been ignored by the structural engineers in general?

A. Mhm.

Q. Would the vertical excitation have had an influence on the behaviour of those foundations, given its very high frequency?

A. Here we have to distinguish between liquefaction and behaviour of the system. On liquefaction vertical acceleration is not going to have significant effect because it effects both, the pressure and the ground water as well as the solid component or the particles so those two effects are going to cancel out so there is no net effect of the vertical acceleration on the pore pressure development. On the other hand obviously this vertical acceleration is going to change the loads of the building at the interface with the foundation and that oscillation on itself it is going to contribute development and softening of the soil at the immediate interface with the foundation so the vertical acceleration will have some effect. Any rocking is going to make that effect non-uniform and I would say that vertical acceleration should be considered especially for buildings where we anticipate significant rocking contribution.

Q. Damping effects I probably don't need to pull up the slide for it but the damping effects, structural damping has a very big influence going from 2% to 5% it has a very large influence on less responding structures probably less on ductile structures but is – when you've got the deep alluvial soils with piles in it do you have increased damping as a result of the pile in the alluvial materials, is there some way one can assess how much additional damping one might get from this type of behaviour?

A. The damping in soils that deform significantly is very large. If soils are going to liquefy and are developing strains in the excess of 1% which is certainly the case when soft soils are subjected to strong earthquake or liquefiable soils we are talking about damping on the order of 20, 30, 40% extremely large damping. Now it's interesting that liquefiable soils when liquefaction is going to develop the stress plane relationship is butterfly loop relationship, very specific which actually reduces damping to a large extent but again that damping will be 10, 15, 20% so whilst deformation in soils comes with damping so some of the energy's going to dissipate it in the soils that's is why I was suggesting that if we improve the ground more of the energy will actually enter into the building.

Q. One percent deformation sounds rather high for soils doesn't it?

A. It is.

Q. Do you think with 1% deformation you might have rather a lot of tilting in your building for it?

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A. Yes but for these kind of very strong events I would expect even in relatively good soils to have .01, .02 percent of shear strength in the ground. Point 4/5 will be for moderate soils and then weaker soils will certainly approach one percent and liquefiable soils will go up to two, three, four percent. So that is why we are seeing this tilt actually because of those kind of strengths in the ground.

Q. I guess that sort of leads onto my next question. I mean there are two schools of thought about whether you should plan to dissipate energy in the soil rather than in the structure by (inaudible 14.56.44) reinforcement. I know that Professor Taylor at the University of Auckland, many years ago, was quite an advocate of dissipation of energy in the soil rather than in the structure but he, I think, was more talking about clay rather than sands. But have you got any, I mean the obvious point is if you can dissipate in the soil you transfer less forces into the structure.

A. Yes.

Q. But then you've got the problem of rectifying any permanent deformation in the soils which may or may not be harder than rectifying any permanent deformation to the structure –

A. Yes.

Q. – depending on the structure type. Would you like to comment on that sort of interface problem?

A. Yes, I think, in concept we can certainly think of these two mechanisms, whether you spend more of the energy in the soil or in the structure itself. When we discuss strong earthquakes the problem is that if you would like to dissipate more energy in the soil it is very difficult to achieve that in a controlled manner. It is much easier to achieve dissipation within the superstructure in a controlled manner and for that viewpoint I would rather go for the second option because then you can engineer against it in a more meaningful way. For static conditions or small earthquakes I think maybe this idea of spending, or providing more dumping and dissipation of energy in the soil might be a good and easily achievable but for strong earthquakes that will be a very challenging task and I would rather go with doing something with the superstructure in terms of dissipation because of the controllability.

Q. Yes the idea of relying on the soil always worried me because I wondered how the soils properties might change over time, especially with clays where you've got consolidation. Now Professor Bray referred to it and I think you briefly referred to severe strength loss in soils, not associated with liquefaction. Can you just describe?

A. Once again severe strength?

Q. Strength degradation of soils –

A. Yes.

Q. – but not associated with liquefaction.

A. Yes, well …

Q. Can you just sort of outline to me exactly what goes on there?

A. Well I would mention two scenarios. One is clay soils and there are strong earthquake excitation, actually there is going to be degradation in stiffness and strength for clays and so we should be expecting reduction in stiffness and load carrying capacity even in clays so more deformation during earthquakes and they may be showing a strain hardening behaviour but simply we're going to see a drop in stiffness and strength and consequent deformation.

Q. Does this depend on whether the clay is sensitive or not? Does that come into it?

A. No even if the clays are not sensitive. Now if we have sensitive clays, so this is especially soft and weak soils, they can also show strange softening behaviour where you have large drop in strength and the residual strength level is extremely low so they're carrying capacity is extremely low. So that would be an equivalent case to liquefaction if you like. So if there are those kind of highly sensitive soils or very soft that is also an issue that has to be addressed.

Q. Thank you. This is for your colleague I think.

COMMISSIONER FENWICK TO MR MCCAHON:

Q. I want to know if, for instance, the Z co-efficient, seismicity co-efficient, increases from say .22 to about .37, 67 percent increase, what sort of increase would this cause or lead to in terms of the cost of foundations? Can you sort of give a rough assessment, a feel for how significant that would be on the cost of foundations?

A. Um, there are perhaps two aspects on this. One is, first of all, the, with the triggering of, the onset of liquefaction we're finding a great many sites that would need a PGA of perhaps .1, .12, we start getting liquefaction triggered. By the time we get up to a PGA of .18, .2, we'll have liquefaction over 90 percent of the liquefiable layers, so that there's very little increase. So an increase of that magnitude for the Z factor of the ultimate limit state is not going to mean we're dealing with a much greater depth of liquefiable material, we've already liquefied it at lower earthquakes. So in terms of that increase it's not so significant. It has a greater impact down at the surfaceability limit state level where we suddenly find ourselves having to design for significant liquefaction on virtually any site which has got liquefaction potential and so this is pushing the buildings into having to be piled whereas previously, a year ago, many sites we could get by with shallow foundations. Now, with the increased seismic hazard there are many sites where we can't, we can't meet the serviceability limit state criteria so we're pushed into piling and this clearly adds cost to the job. Undoubtedly the increase in Z factor will mean that there's greater seismic demand on the building and there's greater seismic demand on the foundations and so there will undoubtedly be an increase in cost in the foundations as well. We might end up with, I don't know, 30 percent more piles or something like this but I haven't had, I haven't thought about this in detail so I can't be more specific than that, but, yes, it would be a flow-on cost in terms of additional foundation requirements.
Q. But I take it the 30 percent increase in piles wouldn't lead to a 30 percent increase in cost.
A. Um, 20 percent increase in cost perhaps.
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JUSTICE COOPER TO MR MCCAHON:

Q. Twenty percent increase in cost of what?
A. Of the foundations, not of the whole building.
Q. And typically what percentage would they be of the cost of the building?

A. I, again, I'm unsure. This is one, perhaps again a symptom of the split between the geotech and the structural that I don't often get privy of the overall budget of the thing. I'm guessing here. Maybe 10 percent.

ASSOCIATE PROFESSOR CUBINOVSKI:

If I can make a comment because this is related to the discussions last week when the Z factor was discussed and the impact of minimum magnitude considered in the hazard one issue that needs to be addressed is what does this mean for the serviceability limit state because I was presented the results from two independent analysis and the outcome of those was very very different and with huge consequences for design so in that context, actually, we have to consider the hazard together with the engineering solutions and the uptake of the hazard in order to really reach the appropriate level and this level should be, this should be considered both for the ultimate limit states and for the serviceability limit states and this would be also structure dependent. I don't want to go into details but this is something, a very serious issue to consider.

COMMISSIONER FENWICK TO ASSOCIATE PROFESSOR CUBINOVSKI:

Q. We probably need to talk a bit more about that at some stage.
A. Yes.
Q. I mean the 80 percent increase in serviceability limit state was very high. I think they might be backing off that a bit, I hope so but that’s, we’ve got to wait and see what happens there.

COMMISSIONER CARTER TO ASSOCIATE PROFESSOR CUBINOVSKI:

Q. Just on that last point about cost increases obviously this is a subject of some interest to those who will study this earthquake and what should be done about it. I just contemplate the possibility that having looked at foundations that have not performed well under buildings for which details are available, one could reassess what type of foundation may have sustained no damage or significantly less damage and that might be a way to get at the increment of cost that would have occurred between what was a previously adequately thought design and what would now be implied by the greater knowledge that we have of the performance so perhaps we could discuss that with you further later on how one might tackle that problem because you do have some knowledge of building foundations that have displayed failure characteristics and perhaps that might lead to what alternative could have been employed and the cost consequence of that.

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A. Certainly. I would like to make just one general comment which is providing maybe some idea about the cost. The foundations are obviously a small part of the total cost of the building and we can say ten percent for example. Recently we have been doing the so-called loss assessment analysis basically calculating what is the total economic loss of an earthquake affecting a building and what is interesting is that eighty percent of the loss is actually not the physical damage to the structure itself but it is damage to contents and components, non-structural okay so this is eighty percent so only 20 is the structure and the foundation is ten percent of those twenty percent so in that context the contribution, the financial contribution for better foundations is really small so we have to consider it in that kind of context to quantify the size of this increase and economic –

Q. Yes that's a very relevant matter to consider. Thank you.

JUSTICE COOPER:

Mr Elliott, do you have any questions for either of these two gentlemen?

MR ELLIOTT:

No thank you.

MR ZARIFEH TO MR MCCAHON:

Q. I just have a couple of things I want to ask to clarify. One that was touched on was this integration of the geotechnical with the structural analysis for the engineers. I think Mr McCahon touched on it. I don't know whether you want to expand at all on that because he said that one of your main recommendations in the report there should be more integration I think was your point.

A. I think in the past the system has worked obviously well for normal loading conditions and the issues that have really come to the surface are to do with the strong earthquake shaking and it's there that there's clearly a need for greater integration between the geotech and the structural sites because we're not just dealing with a static model with a load imposed on it. We are now dealing the deformation within the ground, the ground shaking coming up through it and the response to the structure and the response of the structure then imposes different loads back onto the foundations so that a change in the structure can have an impact in terms of the foundations. As an example one project I was involved in where we had shallow gravel and my recommendation was to found it on shallow foundations so that was accepted. They got well into the design and came back and said, "We're having trouble because we've got overturning. We need tension capacity. We need something to hold the building down when the earthquake shaking comes. We need some pile capacities." And so the design changed at that point but it wasn't integrated to the degree that we stepped away from shallow foundations to a piled foundation which in hindsight we should have done and we ended up with a hybrid foundation which didn't perform particularly well in the earthquake so at that point there was need for actually greater interaction between the geotech and the structural to really look at the implications of what the structural engineer was coming back with in terms of what that really meant with the performance of the foundations.

Q. Do you have any comment on how better integration could come about, what could bring that about?

A. First and foremost, education. There's clearly a great deal of knowledge that's coming out of these earthquake events that the engineering fraternity needs to learn about. I'm not sure how you can legislate for something like that but I certainly think that there are ways and means in terms of the professional institutions pushing this. There are guidelines for design et cetera. This sort of thing should be more prominently pushed.

Q. For geotechnical issues am I right in saying there's no code, no formal code?

A. There is no formal foundation code, this is correct. It's touched on in the Building Code has a Verification Method and there's a section there on foundations. That's the closest we come to a formal code but it's quite general. There was at one stage a Code of Practice for the Design of Foundations. It was a draft Code. It disappeared. It never became formalised.

Q. And the fact that there isn't one is that problematic or not in your experience?

A. I haven't found it problematic provided there is sufficient guidelines to assist me in following accepted practice. Without anything at all then you're very much reliant on text books which might be from overseas. One of the problems with Code of Practice for Foundations is that we're dealing with a wide range of soil and foundation types, conditions, so it's hard to be concise in the way that you can be in a structural steel code because you're dealing with materials which are very much more variable.

Q. And who is responsible for the guidelines you spoke of?

A. The New Zealand Geotechnical Society has recently (last year), the year before published a module in terms of geotechnical design for seismic conditions so this is the Society of Geotechnical Fraternity in New Zealand so they took it on themselves to produce something to act as guidelines.

Q. Thank you.

MR ZARIFEH TO PROFESSOR CUBRINOVSKI:

Q. The only other question perhaps for Associate Professor Cubrinovski, the last paragraph of you report you talk about potentially liquefiable soils not being unique to Christchurch and you spoke of the study that you've personally been involved in, in the Canterbury soils. Do you know what other types of studies are being conducted or have been conducted in relation to other parts of New Zealand.

A. Well I'm not aware of any substantial studies in this regard and I guess they are conducted on a project basis. I haven't heard of any significant research studies trying to identify specific responses of soils but it is easy to understand that this kind of environment is not unique only to Christchurch, that we can see a presence of alluvial soils in many other parts of New Zealand and internationally as well. In fact many many cities are built on fluvial plains and face similar problems in terms of earthquake and liquefaction hazards so I take the outcome from this investigation and recommendations are going to be hugely beneficial to other parts of New Zealand. I would like to mention a couple of things, one to add, that in addition to alluvial soils, maybe artificial soils, reclaimed soils or fills are also very prone to liquefaction and those are areas to be given also special attention and these are prevalent in (inaudible 15:15:00) areas and those are especially vulnerable liquefaction and lateral spreading. And if I can make a comment on the previous discussion related to the issues that geotechnical engineering and the profession is facing when dealing with this kind of evaluation well we understand it basically in the design process and in the assessment we are kind of having three components – one is the hazard component that is involving seismologists, tectonics, evaluation and so on. Then we have structural component and then we have geotechnical considerations. The uncertainties are quite different in these three different domains and the source of uncertainty is quite often different. The uncertainties in geotechnical engineering really come from poor knowledge of stratifications, soil conditions, soil behaviour. So this is why geotechnical investigations are important in order to reduce these uncertainties.

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The level of codification is very different so there are certain hazard procedures for hazard analysis, there is a very well-defined procedure to follow in the structural design and there is very little guidance in the geotechnical design and this is common to many countries because geotechnical engineers face a lot of uncertainties and we don't really know the materials we are dealing with. It is very difficult to be prescriptive and give numbers in the code but it is easy probably to provide guidance in terms of what is the right way to do intensive investigations approaches and dealing with issues so in that sense we are more using guidelines and the recently developed guidelines on assessment and mitigation of liquefaction hazard which was a product of co-operation of the Geotechnical Society and DBH with a group that worked on preparing these guidelines we have now recognised that after this event maybe it is a good time to revise these guidelines and to incorporate as much as input as possible which is going to inform designer foundations as well as other structures and when I say designer foundations I'm referring both to residential and commercial buildings and we here the engineers present are really of the opinion that this needs to be done quickly so any support in that regard in terms of execution of this kind of effort is going to be greatly appreciated. Finally within this context I would like to comment on the position of Geotechnical profession in New Zealand as compared to other countries that I have been working with and Japan in particular. When I was involved in significant projects I did talk to the structural engineer and did, they did, really wanted to hear my opinion and we would collaborate for a couple of weeks just to understand what was going on how certain foundation was performing. The specific position of the geotechnical engineers here has to be put in the context of very strong geological presence and knowledge and also extremely strong structural engineering presence and knowledge. We didn't, this context, the geotechnical engineering is quite strong but very small group and we don't have quantified approach in our design to in that sense there is no really an institution representing our branch of engineering because DBH is a department for building housing, GNS is responsible for the hazard and I cannot mention any equivalent institution for the geotechnical engineering. We do have the New Zealand Geotechnical Society who is a professional association and it provides great support but it can do as much as it can and it is certainly not the same level of the government organisation as the others so I think that we have to look into the solution by realising that this is the status and maybe making for adjustments if we think we can find better ways of dealing with this recognising that this is an integrated approach where we need to bring disciplines and professions together.

Q. In the review of the guidelines that you mentioned what's the timeframe for that?

A. Well we are just shaping up the details around that. We would like to go as quickly as possible. Hopefully within the five, six months or something like that we would like to have at least the guidelines into just one.

MR ZARIFEH ADDRESSES JUSTICE COOPER

JUSTICE COOPER TO MR MCCAHON:

Q. Just one question arising out of that really, supposing you are building in central Christchurch would it be the case that in applying for a building consent the council might require to be satisfied that a geotechnical engineer had designed the foundations, that would be a typical requirement presumably is that right?

A. For a major structure.

Q. Yes.

A. They would normally expect a geotechnical report.

Q. That's right.

A. To be produced by a geotechnical specialist.

Q. Yes. And that would set out what was known about the subsurface conditions and also explain the design approach to foundations, would that be typical?

A. Yes typically the geotechnical report would outline investigations done for the site, describe the model for the conditions found.

Q. Yes.

A. And from that interpret what sort of foundation systems are appropriate and through to design parameters.

Q. And then as far as the above ground structure was concerned it would have to comply with the building code and one of the approved means of compliance say with a standard is that right?

A. Yes.

Q. But there'd be no regulatory requirement that would have satisfied, or designed to satisfy the council that there had been a holistic approach to the, what Associate Professor Cubrinovski and you have called the system, the surface, the foundation structure, a holistic approach to that, there would be no regulatory requirement to that effect, am I right?

A. I don't believe there's any regulatory requirements in that respect no.

Q. Right thank you very much, and may I on behalf of my colleagues thank you both very much for a very valuable contribution to the Royal Commission's work. Can I say that we're very interested in any further that your work achieves between now and when we have to report. I think you're furthering your study of the investigation of the buildings in central Christchurch. We'd be very interested in a report back on that were you in a position to give it to us before we have to end our work. Thank you both very much.

MR ZARIFEH ADDRESSES JUSTICE COOPER

COMMISSION ADJOURNS: 3.23 PM

COMMISSION RESUMES: 3.42 PM

MR ZARIFEH CALLS
KEVIN MCMANUS (AFFIRMED)

MR ZARIFEH:
Sir, I will get Mr McManus to speak to the written submission that he's filed and to briefly summarise it in doing so.

DR MCMANUS:
Your Honour I'm going to speak this afternoon about a sort of a particular technical issue, if you like, regarding the practice of foundation design in New Zealand, especially relevant to earthquake design and I'll just give a very brief summary of the issue, it's quite a simple issue in a way. I've given a reasonable amount of, hopefully, technical backup in the paper and perhaps if I skip most of that and you can ask any questions or clarifications. Basically the issue that I want to raise is that under the New Zealand Building Code and Regulations, and people have already referenced Verification Method VM4 as a almost pseudo code for foundation design. Under certain load cases during earthquakes the Verification Method permits a very low factor of safety to be applied for the design of building foundations and that factor is as low as a factor of 1.1 and that's specific for buildings that are under capacity design, in other words they're designed to yield under the earthquake loading and that yielding of the members imposes a certain load onto the foundations and, in that case, the factor of safety of 1.1 effectively applies. That's, in my opinion, a very low factor of safety and this is specific to the vertical loads. In other jurisdictions and traditionally a factor of safety of at least two would normally be applied to such foundations and there may be some, using capacity design the design is, in some senses, trying to limit the forces within the structure and by allowing them to yield and some people may argue that one way of doing that would be to allow the foundations to yield but my submission is that by permitting soil yielding the foundation performance is so variable that it would

be very difficult to predict the way in which the building itself would then deform. So, in my opinion, it's far more satisfactory if the foundations remain resilient, in other words do not deform excessively and allow the structure to perform as designed. The, typically the, as I mentioned, the factor of safety of two would normally be used in geotechnical design. The reason for that is the capacity of a foundation is very variable, it's quite difficult to predict. It's not like a structural element made of concrete or steel where the properties of the materials are known rather well because they're tests extensively, make in quality control factories. The soil itself is very variable and very hard to exactly predict from place to place what the strength is and Dr Cubrinovski has been alluding to that through most of the day and it is for that reason that we tend to use much higher factors of safety, not because we're trying to build in conservatism but we're trying to account for the actual variability so that if for any given foundation there's a reasonable degree of reliability that it will achieve the target strength and I think, historically, there's been some misunderstanding around that and, again, I think it comes probably because of the different ways that structural engineers and geotechnical engineers look at design issues. I think it's probably fair to say that the low factor of safety is really a result from a committee of structural engineers who wrote the loadings code at the time and just the way these things go that kind of misunderstanding seems to have been replicated in all subsequent codes. In fact it's dropped out of the loadings code but it's been picked up in Verification Method VM4.

JUSTICE COOPER:

What was it VM4?

DR MCMANUS:

So in terms of, and I've actually spoken to the author of VM4 so he assures me really it's just a repetition of what was in previous codes. So my recommendation is, sir, that the very high strength reduction factor referred to in VM4 specifically, as high as 0.9, which is equivalent to a factor of safety of 1.1, is inappropriate. Many foundations so designed will receive over-strength loads exceeding their capacity and leading to excessive, which really means excessive plastic deformation. The high variability of soil properties and foundation performance ensures that the overall behaviour of the structure will be unpredictable and, most likely, undesirable. Premature failure of some foundations is quite likely, just because of the variability. So my recommendation is that the selection of strength reduction factors for foundation design should be based on a proper risk assessment procedure, and I've given a very good example there of the Australian Pile Design Code, AS2159. The objective of this proposal would be to ensure that foundation performance is reliable under all load cases, including the earthquake over-strength case. There seems to be no basis for treating capacity designed buildings as a special case where unreliable foundation performance should be acceptable. In terms of, the reason that there is a problem, I think, is that its now reflected in this Verification Method 4 so it's a kind of pseudo-code an it then puts pressure on people to actually use that as a design basis because it's very difficult to explain perhaps to your client why you're not using it if it's permitted in the code, even if you might perhaps disagree with it. So I'll stop there but be happy to answer questions.

DISCUSSION BETWEEN COMMISSIONERS

COMMISSIONER FENWICK:

Q. Do you wish to add to, I did ask the two authors before about the influence of rapid loading on shingle and sand type foundations and the influence of vertical excitation, vertical ground motion on the forces, clearly the gradual loads get increased by the vertical excitation. Would you like to add any comment about the effect of these on the gravel and sand foundations?

A. Yes I'd like to make a couple of comments, if I may, about other relevant issues that came up during that conversation. First of all, whether or not there are such dynamic effects those might properly be taken into account by the geotechnical engineer in calculating the foundation capacity. I don't see that the factor of safety is the proper place to sort of try and scrub up a few other side issues but I agree with Dr

Cubrinovski's conclusion that, generally speaking, earthquake loading is reducing the capacity of a foundation through a number of mechanisms. For instance, clay cyclic loading of the interface of the piles is a known problem that that degrades the cyclic loading. There are pore pressure increases and there's also a reduction in strength generally on clays with the cyclic loading so there's a whole range of factors that are actually acting to decrease the foundation capacity and those are certainly not covered in the M.4 for instance and the extent to which designers may be accounting for those is uncertain which is one reason why we'd like to get some guidelines out there to try and get that tidied up. So in general I think the –

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JUSTICE COOPER ADDRESSES DR MCMANUS – SOUND PROBLEM

A. As a general comment Commissioner Fenwick I don't believe that those dynamic effects are an excuse for having a low factor of safety.

Q. This raises another issue of course because the structural designer is working to a series of load combinations and though vertical earthquake actions are considered for a few elements they are not considered for the structure as a whole except under exceptional circumstances so they will not have actually worked out what the, they would not have added in the vertical excitation effects from the ground motion because that just doesn't appear in their equations at present so it's probably a lack of education partly on structural engineers as well.

A. But it's another argument one would say for having a higher factor of safety to account for miscellaneous sort of second order effects that we may not be accounting for explicitly, just, could I make one other observation. You asked Dr Cubrinovski the question or it was more of a statement that as a geotechnical engineer you know you suggested that the structural designer would be giving load values for the serviceability limits state, the ultimate limit state and the over-strength case and speaking for myself that, I've never observed that, that generally we might be given if you're lucky the loads from the serviceability limit state and the ultimate limit state. I've never had a structural engineer explicitly state a load case for the over-strength case as a separate number and it does leave me sometimes to wonder whether structural engineers fully unders – are in some cases actually applying that very low factor of safety for the ultimate limit state not just for over-strength capacity.

Q. If that's the case it's pretty alarming, the 1.1 factor of safety on the standard seismic actions but the over-strength actions can be twice as high easily or more so if you're not getting the over-strength actions then that's something that certainly needs to be addressed between the structural engineers and the geotechnical engineers?

A. Might I also comment on the discussion about the integration if you like of the structural design and the geotechnical design. In my experience in practice that in New Zealand we see the whole range of that. I'm involved in specific projects where there's a very close co-operation between the structural engineers and the geotechnical engineers in a very beneficial way but at the other extreme you get the structural engineers who just want you to write a geotechnical report give two strength parameters for the soil and a soil stiffness, whatever that is, for them to use in their structural engineering programmes. Personally I refuse to get involved in those project s.

JUSTICE COOPER:

Q. To the extent that there is a more collaborative approach it's all voluntary in the sense there's no, there's nothing in the regulatory environment which requires it is that right?

A. No there's no, there's no, there's no regulations that go into that depth as I understand it. It's basically whatever the territorial authority will accept in terms of design statements and so forth.

Q. The territorial authority may not be the best judge of whether a more collaborative approach would have been desirable in a particular development?

A. No that's correct yeah, it's basically up to the judgement of the engineers involved.

COMMISSIONER CARTER:

Q. Just on that particular point, you I think were present when Professor Bray was talking this morning were you, you might have noted that he had a suggestion of peer review being part of a process that could be looked at with – do you have any observation on that or how that might be applied in practice?

A. Yes I was present for that and I thought Professor Bray's comments were very useful. In my experience that certainly for most significant projects, large buildings and so forth, most councils in my experience require some sort of peer review and I think a lot of engineers also seek peer, a peer review for their own comfort but almost exclusively in my experience that peer reviewer is appointed either by the designer or the client and this creates a difficult tension if you like between a sort of commercial tension that if let's say you are reviewing someone else's work that if you're perceived as being too difficult or conservative that you're, you know there's market pressures that then would say that you may not get any more work. I think for that process to work effectively and I think it can work effectively it's probably the best you know professional peer review is probably the best way of assuring quality but I think it needs to be directed through the regulatory authority. In other words the chain of instruction should come from the regulatory authority, whether or not the developer or the client actually pays for that in the end but you can see Your Honour I'm sure that there's a subtle twist in that and I'm pretty sure in the California situation that that would be the case.

JUSTICE COOPER TO MR MCCAHON:

Q. Mr McCahon do you care to comment on anything that Mr McManus has told us?

A. I thank Dr McManus actually for putting this paper together because it has clarified some issues for me in terms of how we've got to where we are and I basically support his recommendation that this should be addressed and in terms of the peer review yes peer reviews are done not necessarily routinely it depends very much on either the client deciding that it would be useful to have a peer review before they submit for building consent to streamline the process or on occasion by the local authority requiring a peer review so both, I've experienced both, both systems here.

JUSTICE COOPER TO ASSOCIATE PROFESSOR CUBRINOVSKI:

Q. Anything from you?

A. Well I would like to support the submission as well and I think it's a very important issue that needs to be addressed and there might be other details that we need to address including the hazard interpretation, especially under the new circumstances but specific to foundation design this is certainly something that needs to be carefully looked at.

JUSTICE COOPER TO DR MCMANUS:

Q. Dr McManus I just want to make sure I'm understanding what you're recommendation is. At one stage in your paper you've referred to a traditional safety factor of two but then you've addressed I take it what you consider to be more sophisticated approaches such as is found in the Australian Piling Code so that your recommendation is in the last paragraph of what you write, "the selection of strength reduction factors for foundation design should in all cases be based on a risk assessment procedure such as that in the Australian standard". Do you think that's the better way to go than to go back to the blunter, what I would describe as a layman, as a sort of blunter approach of a factor of safety of two?

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A. Yes Your Honour. The advantage of going to, basically let's say roughly you're going back to a factor of safety of two but there's quite a bit of variance on that depending, so obviously the benefit of having the variable risk approach is that it provides quite a strong incentive to use improved practice. In other words, improved design procedures, things like load testing which give you a more certain understanding of the behaviour of the foundation and you get rewarded by that by being able to use a lower factor of safety let's say which has a direct financial consequence in having say fewer pile foundations so I see that as a much superior option but it's within limits. It certainly doesn't extend as far as the factor of 0.9 currently.

Q. No, and also I think probably inherent to what you've just said but it would also require a detailed site investigation to apply it I take it, or an appropriately detailed site investigation?

A. That's part of the risk assessment. So again the thoroughness of the site investigation is taken into account and if that's a very limited investigation you have to use a low strength reduction factor. In other words you have to size up the foundations to take care of that extra risk so it is encouraging you to de-risk the situation by doing more investigations, more thorough testing and so forth so it really drives things in a desirable direction.

COMMISSIONER FENWICK:

Q. Are there any other aspects you'd like to comment on while you're there related to foundation design?

A. No. I think I support most of the discussion that's been going on previously. Hopefully we've started a process perhaps with the New Zealand Geotechnical Society to address this issue and some even more technical issues surrounding seismic design of foundations but ultimately for this specific issue there will need to be a change, if you like, of the approved document so it can't just be handled in the sense of guidelines.

JUSTICE COOPER:

Thank you very much indeed for taking the trouble to come along and speak to your submission. We are grateful to you.

COMMISSION ADJOURNS AT 4:05 PM

COMMISSION ADJOURNED UNTIL 7 NOVEMBER 2011 AT 9:30 AM