HEARING RESUMES ON THURSDAY 26 JULY 2012 AT 9.30 AM

JUSTICE COOPER:
Well Professor Mander can you hear me?

PROFESSOR MANDER:
Yes I can Sir.

JUSTICE COOPER:
Can I just ask you to join in the formality of promising to tell the truth which I am just going to put to everyone here. We have Messrs Hyland, Jury, Bradley, Smith, Carr, Bradley, Davidson and Stuart here and now you and if I can explain that we have reached the point in discussing the NTHA analyses where we are hearing from individuals reading their statements on the interpretation of the new results and I hope you have your statement with you. Do you?

PROFESSOR MANDER:
Yes I do Sir.

JUSTICE COOPER:
Good. Well first of all then can I ask each of you by saying yes after I call your name to solemnly and sincerely truly declare and affirm that the evidence you give shall be the truth the whole truth and nothing but the truth. Mr Stuart?

MR STUART:
Yes.

JUSTICE COOPER:
Dr Davidson?

DR DAVIDSON:
Yes.

JUSTICE COOPER:
Mr Derek Bradley?

MR BRADLEY:
Yes.

JUSTICE COOPER:
Professor Carr?

PROFESSOR CARR:
Yes.

JUSTICE COOPER:
Mr Smith?

MR SMITH:
Yes.

JUSTICE COOPER:
Dr Brendon Bradley?

DR BRADLEY:
Yes.

JUSTICE COOPER:
Mr Jury?

MR JURY:
Yes.

JUSTICE COOPER:
Dr Hyland?

DR HYLAND:
Yes.

JUSTICE COOPER:
And Professor Mander?

PROFESSOR MANDER:
Yes I do.

JUSTICE COOPER:
Thank you. Now Professor Mander your statement about the revised NTHA analysis I think is one page and the first line is response regarding the overall findings of the NTHA panel?

PROFESSOR MANDER:
Yes that's correct.

JUSTICE COOPER:
Would you read that to us please?

PROFESSOR MANDER:
1. The NTHA process has come a long way since the original submission from the DBH. The first report was thought with so many problems, faulty assumptions and errors that in my view should not have been published in its present form. Nevertheless the first analysis and report provided a useful starting point for the second round the NTHA recently completed.

2. Main accomplishments in the second round have included
(a), modelling of the beam column joints,
(b) end on end analysis of earthquakes commencing 4 September 2010 through, well including 22nd of February 2011,
(c) the REHS motion is now included,

(d), and more realistic concrete strength,

(e), a PMM concrete failure surface for the columns largely due to the fibre element modelling used for the columns.

(3) Notwithstanding these improvements the second phase still needs further fleshing out for sake of completeness and to raise confidence in the comprehensiveness of the results.

(4) Specific modelling enhancements that remain to be done are:

(a) improvements in the beam column joint modelling. It is contended that the interior strengths assumed remains excessively high. Rational mechanics results show this, and while it may be possible under some rare circumstances to justify the chosen model capacities it is contended that at best this higher strength can only apply for the initial cycle of loading until the joint cracks. Subsequent cyclic softening is not adequately modelled but is more likely to be similar to that sighted in the Park and Mosalam paper. By the way I believe that's in the input file and record that Professor Carr put together.

(b) Another major problem with the joint modelling is the capacity is set based on an average axial load while in reality it will be a function of axial load should be permitted to fluctuate accordingly. Reinforcing bars within the joints particularly the pair adjacent to each side without a beam will be prone to buckle under a modest axial strain. This buckling effect is considered to assist the breaking off of the wings in the precast units leaving the joints completely exposed to rapid failure. Attempts should be made to model this phenomena and then capture its effect in subsequent analyses. The connections are based on post processing using a CD ratio process. This approach is archaic and given the power of the modelling should be avoided wherever possible. Instead the direct inelastic behaviour should be modelled and the member performance permitted to be whatever it needs to be without bias. In this way complete failure can follow through until collapse is observed. True geometric non-linear analysis is not handled directly, only inferred by a stiffness matrix modifications and

(5) with the above changes implemented it is likely to confirm that the interior frames on lines two and three were indeed weaker in the location of the collapse initiation.

JUSTICE COOPER:
Thank you very much Professor Mander. Mr Rennie, is there anything else you would like to lead from Professor Mander at this point.

MR RENNIE:
Not at this point Sir no.

JUSTICE COOPER:
All right well then I think we should hear next. If you will just wait Professor Mander we'll come back and there'll be a discussion about what you've said as well as what others are going to cover. We heard from Dr Hyland yesterday evening with his statement. Have you read that?

PROFESSOR MANDER:
Sir I've seen about four of them. Ms Kelly Paterson advised me to look at several pieces. Dr Hyland's was one and I've seen. I guess I've seen all of them except Professor Carr reading the statement but I've read that so I'm familiar with what's on.

JUSTICE COOPER:
All right. Well you're up with the play then thank you. Now I think we go to you Mr Smith next please.

MR SMITH:
I'm referring to my letter dated 23rd July.

JUSTICE COOPER:
That's correct.

MR SMITH:
Now starting with the interpretation of the, or the joint report and the item that I identified as an item of disagreement. I possibly wouldn't call it disagreement but I felt it needed further clarification and so I'm going to the bottom paragraph of the first page. The first section is just a repeat of the questions that were asked.

JUSTICE COOPER:
Thank you.

MR SMITH:
The matters under item 6A were addressed in a joint report in relation to interpretation of the second Compusoft NTHA prepared by Professor Carr on 22nd July 2012. Under areas of disagreement item five in that joint report I recorded a disagreement as follows and that is the revised NTA collapse scenario appears to be and under a) Mr Smith comments that there was failure of the line F columns. Some records predict mid to upper level columns to exceed their ultimate compressive strain prior to actual load carrying capacity being lost in the level 1 columns. The reason for my disagreement here was to clarify that ultimate compressive strains were well exceeded in some of the line F columns at mid to upper levels and after having looked further at the results it's actually, could be lower levels as well but all on line F. Prior to the ground level column actual failures listed in table one of the joint report. This means that either column bar buckling or significant concrete spalling may have occurred in the line F columns at those earlier times, to the extent that those line F columns could not support their axial force as explained in the agreement on uncertainties, item K of the joint report. And for example now I would like to refer to these figures if I could?
0943
JUSTICE COOPER:
Yes.

MR SMITH:
And I'm not sure whether to use the first draft to the second draft.

JUSTICE COOPER:
I think if they're in both, use the second one.

MR SMITH:
Use the second one? So the references I gave in the written report were actually the first draft. It becomes figure K21 in the second draft.

JUSTICE COOPER:
K21, now do you know where we might find that?

MR SMITH:
I'll just have a look for it.

JUSTICE COOPER:
Does the "K" indicate it's an appendix?

MR SMITH:
Yes it is, appendix K, it's the reference at the top right, BUI.MAD249.0552.273

JUSTICE COOPER:
273, okay. Now Professor Mander are you able to see that?

PROFESSOR MANDER:
Not at the moment Sir. I'm seeing you.

JUSTICE COOPER:

Well there's something rather more pretty that we want you to be able to see. Will he be able to? No. Do you have the Compusoft amended report with you?

PROFESSOR MANDER:
No, no I don't but let me try and see if I can get it online because this I believe is streaming live and I might be able to see it that way if it's being broadcast live.

JUSTICE COOPER:
Right, okay.

PROFESSOR MANDER:
I have my computer in front of me so let me try.

JUSTICE COOPER:
Yes.

PROFESSOR MANDER:
Thank you. Sir this may take me a couple of minutes. I suggest that you continue on.

JUSTICE COOPER:
Proceed, all right.

PROFESSOR MANDER:
I will…

JUSTICE COOPER:
Mr Smith?

MR SMITH:

So I'm referring to the diagrams on the right-hand side in this figure. So we're looking there at the time histories for column at grid intersection F3 with level 1 at the bottom, level 5 at the top. So now what I'm looking at is what is occurring at the time of four seconds on that plot and I'm looking at the blue lines extending below zero on the vertical axis at the time of four seconds. If we can, I'm not sure if we can make that out at that scale but…

JUSTICE COOPER:
I can see it.

MR SMITH:
Okay, now the thing to, the other point is there is an orange dotted line horizontally across on each of those plots and that is drawn at the point where the concrete is exceeding its ultimate compressive strain, and so we're seeing some of those blue traces extend well below that line which indicates that the concrete is exceeding its ultimate compressive strain at that, at that point. It's happening at levels 1, 2 and 3. At levels 4 and 5 it does not extend below the orange line, so we're seeing excessive strain at levels 1, 2 and 3 for that column at four second point.
And I want to compare that to, if we can now go to figure K23 on page 275, so it's two pages further on. And in particular I'm looking at the figure on the bottom right-hand corner which is for the interior columns at grid, or column at grid C2 intersection at level 1 and in particular we're looking at the green line on that plot which is the failure criteria reported in the joint report at the time of I believe 5.82 seconds. So what we're seeing is the, at that time of 5.8, just prior to six seconds that column is losing axial capacity and that green line is dropping off. But the point of difference that I want to make is the earlier plot I showed you was occurring at a time of four seconds, so well prior to that we are exceeding ultimate compressive strains in the grid F columns. Where the also, the other point that hasn't been modelled explicitly is potential for the column bars to buckle prior to that also. So there's two failure modes that are

mentioned in the joint report that are potentially occurring prior to this time that's been highlighted. That's really just what I wanted to clarify there. Now do you want me to continue through the?

JUSTICE COOPER:

Yes, yes.

MR SMITH:

So really that, well let's read it out.

The reason for my disagreement here was to clarify that ultimate compressive strains were well exceeded in some of the line F columns at mid to upper levels, and as I said I've looked at the level 1, line F column also since writing this. The point about the level 1 column, it was drawn at a much smaller scale so it wasn't immediately apparent to me but it's just the scale of the plot. This means that either column bar buckling or significant concrete spalling may have occurred in the line F columns at those earlier times to the extent that those line F columns could not support their axial force as explained in item K of the joint report.

I will skip the next paragraph because I have explained that by referring to those plots.

Regarding the questions under item 6B and 6C I have attached modified versions of the DBH report conclusions, so rather than trying to write something else I relooked at the conclusions that we had in our report and the summary of findings that I presented at my last visit and just to see if those statements, I felt, were still applicable. In both those attachments I have highlighted in yellow the points that I believe have been verified by the second Compusoft NTHA. So the point is I've highlighted some sections in yellow that are relevant to the analysis, other sections were determined by other means other than the analysis, so I'm highlighting in yellow the applicable portions and then I'm reviewing that wording.

In attachment B I have shown a revision to the wording that I used in relation to the north core diaphragm connections which takes –

JUSTICE COOPER:

Perhaps we could display that?

MR SMITH:

Look at that so it's –

JUSTICE COOPER:

550.5. BUI.MAD249.0550.5

MR SMITH:

So I'm looking at the bottom paragraph, so which is the only one that's had an alteration because it's shown in red. So I'll just read it out as it is the amended form.

"It appears from the collapse evidence that these east face columns may have failed prior to diaphragm slab or drag bar disconnection at the north core. However the NTHA indicates that slide, that diaphragm slab or drag bar disconnection, or at least partial damage to those connections was likely prior to the column failure, and so that certainly remains a possibility."

So I'm, I guess it's linked into our scenarios 1 and 2 in the DBH report. Scenario 2 was the internal low level column failure which was indicated on the green line in the plot that I showed and certainly what we're saying is did something happen prior to that to cause failure elsewhere and looking at the evidence, the other evidence we had we felt that something did occur on grid F prior to that, so just giving the basis for stating that.

0953

JUSTICE COOPER:

So when you say there so that remains a possibility you're referring to disconnection of the diaphragm slab or drag bar occurring prior to column failure?

MR SMITH:

That's correct. We do see this from the analysis.

JUSTICE COOPER:

And then is there anything, no, there is no other red changes in that attachment are there?

MR SMITH:

That's correct, the thing I highlighted was just the portions that were relevant to the analysis.

JUSTICE COOPER:

Then you're going on to respond to each specific questions. You set out questions in italics. Is that right?

MR SMITH:

Okay, I'll respond to each of the specific questions as follows.
The first one about, do the results of the second Compusoft NTHA change any opinion expressed in relation to the response of the CTV building to the earthquake at 4.35AM on 4th September? My response is no, refer to attachment A paragraph 3 which I can read it out. "Although there was some scope for interpretation of the reported building condition the estimated response of the building using a September ground shaking records, and the assessed affects on critical elements are not inconsistent with observations following the September earthquake."
Under section 2, is there, do the results change my opinion on the response of the CTV building to the earthquake at 12.51PM on 22 February? The answer no.
Item 3, has my opinion changed on why the CTV building failed on 22nd of February, and I guess, well I've just explained no. Refer to attachment B for clarification of wording I would use in relation to the northward core diaphragm connections which I read out just a minute ago.
Under item 4, has my opinion changed about the sequence of failure of the CTV building, as I explained no. A number of collapse scenarios were evaluated. Variability and uncertainty in physical properties and analysis processes do not allow a particular scenario to be determined with

confidence. That's the wording from our report. A likely scenario and the scenario that appears most consistent with the collapse evidence and the eye witness reports was initiated by failure of a column on the east face of the building at mid to upper level and as explained, I would also count level 1 as a possibility having seen this latest analysis. This column failure would have been caused predominantly by north-south direction drifts but could also have been influenced by east-west drift or vertical seismic loads, spandrel panels or low concrete strength. Also from attachment A increased demands due to diaphragm slab separation from the north core or at least damage to those diaphragm connections may have contributed to the collapse.

So I just want to explain further about the predominantly by north-south direction drift. That is also borne out by the results of these plots that I refer to that at the time prior to about five seconds in the record we're getting predominantly north-south direction drifts and that is also the case for the September earthquake and it's only at around five seconds plus that we get major east-west drifts which cause the problems for those internal columns.

So item 6, the experts are to address if the results of the second analysis change any opinion, in what way has the opinion changed? Okay, my opinions are covered in the attachments A and B which are again, the conclusions from our report and my summary of findings.

In summary I consider that the second Compusoft NTHA has been a worthwhile refinement of the initial NTHA that was carried out for the Department of Building and Housing. It is verified that the overall response of the building was accurately predicted by the first Compusoft NTHA. The sequential analysis using ground motion records of the 4th of September followed by 22nd of February appear to have had little effect on the overall results and conclusions. The second NTHA has investigated potential column and beam column joint failure mechanisms in greater depth and this has been helpful in achieving agreement by the various parties on many aspects. My conclusions about the various vulnerabilities and the likely collapse scenarios remain largely unaltered.

JUSTICE COOPER:

Now I think we'll hear from you Dr Bradley?

DR BRADLEY:

So from my individual submission it reads: this document provides written comments on the interpretation of the joint report in relation to the interpretation of the second Compusoft NTHA. The capability of the NLTHA model to simulate the likely failure mechanisms of the CTV building has been significantly improved from the first version which was included in the H-S report. However there remain acknowledged limitations of the current model which must be forthright in one's mind when considering the seismic response of the CTV building predicted during several earthquake events. These include:

(1) the beam column joints are now modelled with moment rotation springs while previously they were considered as rigid. However regarding the constitutive of model for the moment rotation of the beam column joints, there is significant uncertainty as to the peak capacity of the joint and the constitutive model does not consider degradation of strength over successive cycles of loading which will occur following peak joint capacity being reached as a result of cracking.

The second point is particularly important because such degradation effects are considered in the fibre modelling of the column elements and therefore this impairs the ability of the analysis to allow for beam column joint failure prior to column failure. There are several alternative beam column joint models which could be considered and given the identified vulnerability of the beam column joints it is prudent to examine the sensitivity of that NLTHA to this uncertainty in beam column joint modelling.

(2) The beam column join model also does not consider the time varying effective axial load which is known to be significant as a result of significant vertical acceleration.

(3) As a result of a lack of confinement in the joint, rebar buckling is likely to occur at small axial strains, particularly the pair adjacent to each side without the adjoining beams. This buckling effect is considered to assist in the breaking off of the wings in the precast units, leaving the joint completely

exposed to rapid – that should say deterioration. Attempts should be made to model this phenomenon and then capture its effect in the subsequent analyses.

(4) There is debate on the uncertainty in concrete strength. Initial analyses used a strength of F prime C plus 2.5 megapascals while the revised analyses have used 1.5 times F prime C. As several potential failure mechanisms are not directly related to concrete compression or tension capacity, then the variation in concrete strength may result in a different sequence of local failures leading to the global collapse mechanism. As such the sensitivity analysis on concrete strength should be examined.

(5) I consider it somewhat a violation of the principle of consistent crudeness that such complexity was given to modelling of non-linearities in the structural details of the CTV building but the effect of sufficial soils were modelled simply as linear springs with tension gapping. At the least a sensitivity study should have been considered and in that sensitivity study I note that larger variability than the plus or minus 20 metres per second shear wave velocity used by Sinclair should be considered. Soil non-linearity occurs at infinitesimal strains and therefore plastic deformation of soils is essentially always occurring. While high frequency vertical loading of the foundations such as that which may result solely from vertical ground motion results in relatively small strains and therefore soil stiffness remains high. The lower frequency vertical loading which has transmitted through the foundation from translational sway of the structure likely resulted in larger strains, during which time the equivalent strain stiffness of the foundation soil would have been less than the small strain stiffness. I note that this occurs even under (inaudible 10:02:47) conditions which Tim Sinclair noted in his testimony is a reason for the high stiffness values that he provided. In the Compusoft report there is a citation to Carr 1994 in reference to it being more important to model gapping of the foundations and the underlying soil than modelling non-linear soil response. It is important to note that since the publication Carr 1994 the awareness of the importance of soil structure foundation interaction has been considerable and it would generally be accepted now that such a comment is only valid for dense soils which it is argued that the CTV site does not possess or does not

have. The explicit modelling of soil non-linearity would result in the ability of the foundations to have differential settlement during the response to ground motion. This would result in re-distribution of forces in the structure which may be significant in leading to additional distress at several critical elements.

(6) Drag-bar strengths used in the revised non-linear time history analysis are likely in upper bound as noted in the joint panel report. BECA provided information on their view for drag bar strengths. Professor Priestley's evidence also notes that the current drag bar strengths are based on drag bar bolt shear failure whereas his calculations suggest that failure will occur as a result of flexural failure. Given that the analyses for the 4th of September 2010 earthquake illustrate that failure of the drag bars was likely to have occurred, not to mention failure early in the 22nd of February 2011 ground motion then the use of a lower and arguably more realistic value for the drag bar strengths will likely indicate a greater performance, a greater predominance of drag bar failure than their current analyses already illustrate.

(7) Beam bar pull-out was not explicitly modelled in the revised non-linear time history analysis. Post processing of the analysis results suggests that beam pull-out demands exceed their capacity. As a result, this failure mechanism should be modelled explicitly in any revised analyses to allow for the effects subsequent to this failure to be considered and to understand whether it is important in the global failure of the CTV structure.

Finally, (8) – The analyses utilised small displacement theory with a P-delta adjustment, hence geometric non-linearities are not explicitly considered which are likely significant given the high axial loads on vertical load resisting element. Differential vertical deformations as a result of foundation settlements and/or initiation of beam column joint or column collapse will lead to a redistribution of loads which may overload other elements and are not currently considered. It is noted that the neglect of large deformations does not allow for the possibility of buckling type failure that is postulated as a possibility in the evidence of Professor Mander.

JUSTICE COOPER:

The word in your text is "displacements". You read "deformations".

DR BRADLEY:

Sorry, Sir. In this context.

MR JURY:

I have read the report prepared by Compusoft Engineering dated July 2012, that's the first draft issue, and I have compared the results presented with those given in the report described in the first Compusoft Engineering Reported dated February 2012. I am aware of the additional results that are in the second draft issue and I don't believe my opinion changes as a result of seeing those. It may be obvious but I just noted that I wasn't a member of the expert panel that directed these new analyses but have really just been reviewing the results.

I will respond, as others have, to the particular questions requested by Counsel Assisting the Royal Commission. They were dated 17th of July.

In relation to the matter of a response to the CTV building to the earthquake on 4th of September 2010 – Although the latest analysis results might arguably suggest slightly higher levels of damage than the original analyses, I still agree with the view expressed in the DBH expert panel report that the reported damage after this event, and quoting, "appeared to be relatively minor and was not indicative of a building under immediately distress or having a significantly impaired resistance to earthquake shaking." In reaching this position I am putting greater weighting on the observations that were reported after the September event, necessarily on either the first or the second NTHA results. I also note the NTHA carried out the 4th of September 2010 and 22nd of February 2011 shaking in series or in sequence indicate that the damage following the 4th of September event had little influence on the overall performance of the building during the 22nd of February event. In relation to the matter of the response of the CTV building to the earthquake on 22nd of February 2011 – Notwithstanding the refinements made to computer model the levels of storey drift predicted by the second NTHA are comparable with those predicted by the first NTHA. This is also after conclusion of the results using the Rest home records. This confirms to me that although there are obvious changes in the elemental force envelopes and the order in which elements reach particular levels of stress, that the overall levels of performance of the building predicted by the two analyses are not dissimilar given the accuracy of the analysis procedure.

In relation to the matter of why the CTV building failed on the 22nd of February – The second NTHA analysis confirmed that the most likely reason for the building collapse was the failure of one or more columns. This is consistent with my view and that of the DBH Expert Panel as expressed in its report. Joint failure was not predicted in the latest analyses but I do recognise the joint model, although refined from earlier analyses, may still not model performance of the actual joints during this earthquake. It would appear that this issue is not likely to be resolved in the short term and I would also add that I still believe that the loss of stiffness of the joints is perhaps more important, if not equally important, to the loss of strength.

In relation to the matter of the sequence of failure of the CTV building on the 22nd of February – the second NTHA provides strong support for the DBH Expert Panel collapse initiation scenarios 2 and 4. They related to the failure of the internal columns and in scenario 4 also affected or were influenced by the failure of drag-bars. In my evidence to the Royal Commission on the 10th of July of this year I indicated that collapse initiation scenario 2 was the most likely in my view at that time. The latest NTHA results provide a stronger case for this view in my opinion than the earlier analyses but do not rule out the possibility of failure of the floor diaphragm to north wall connection on a number of floors, and that's collapse initiation scenario 4. There still remains the possibility that these connections did not fail based on the observations made following this event and as diaphragm connection failure is not necessary to predict column failure, I am still comfortable to retain my view that collapse initiation scenario 2 is the most likely of those that have been proffered. It is at least possible in my view that the NTHA predicted diaphragm connection forces that are of such short duration that the magnitudes of the forces on their own are not sufficient to indicate failure and that relates to the high frequency nature of the forces that are predicted in those elements. Based on the above or the opinions that I've mentioned earlier, I confirm that in my opinion the second NTHA would not lead me to change the opinions that I expressed to the Royal Commission on the 10th of July.

JUSTICE COOPER:
Now Professor Carr you said that your separate statement was largely repetitive of matters in the agreed statement, is there anything you'd like to raise at this point?

PROFESSOR CARR:
Not particularly. I wasn't initially intending to give evidence on the first report and therefore any changes were not relevant in that sense and so I feel again there's nothing more that I add at this point apart from the fact that there are uncertainties and maybe they need to be thought about at some point.

JUSTICE COOPER:
Right, thank you. Now Dr Davidson and others from Compusoft there have been some criticisms of the second analysis. Is there anything that has been raised to which you'd like to respond? Yes, Mr Bradley.

MR BRADLEY:
I guess there are some obvious areas where some improvements could be made, notably the beam column joints but again we have noted that there is no real data with which to calibrate any model or recognised model that we could use. We believe that the analysis that we've got adequately projects or shows the trends in building behaviour and any enhancements that we would do would, you know, help narrow down some things but I don't think there'd be any fundamental change to the performance in the building from what we've shown to date.

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JUSTICE COOPER:
Yes. Dr Davidson, anything you'd like to add?

DR DAVIDSON:

I support what Mr Bradley has said. It seems to me that the criticism is primarily in the modelling of the beam column joint and we sort of feel that as not particularly being experts in concrete behaviour we would need some good support from other people as Professor Mander to try and resolve that and that what not be resolved in the timeframe I suspect the Royal Commission wants to act in.

The other point is the north wall connection and the forces and this high frequency, whether it would disconnect or not. It could be modelled for sort of almost historic reasons the way the modelling process took off, that has not been modelled as a possible disconnection feature and it could be redone but that would be a few weeks' work and that might resolve that issue.

I think the third point, and may be the only one that we think is possible and feasible, is to investigate may be whether more damage was done during the September earthquake, was whether the masonry shouldn't be modelled as sort of the expected value. So in other words, most of our model in the main has been taken using expected values. I know there's a discussion on the drag bars but at the moment we have no masonry in the sense of a structural element in there. It was not intended to be structural in the design we understand but it could be modelled with gapping elements and so on so we believe it would interfere and alter the behaviour during the September earthquake to some amount of and of course we've tried to bound it in the original analyses by having it in completely and not having it in, but as we have discussed today and yesterday, these things do interfere with the dynamic behaviour and it may alter some damage that would have occurred in September. We don't believe it would make any difference when we come again to February.

JUSTICE COOPER:

Yes, well Professor Mander, you've heard what's been said about the difficulties of modelling the behaviour of the beam column joints. What do you say to that?

PROFESSOR MANDER:

I don't have the numbers here but I believe you probably have them. It's in Professor Carr's paper on the inputs and in that I did an analysis on the strengths of the joints based on a rational mechanics approach plus using what is in FEMA documents. Now I will concede that the FEMA document is conservative but when we actually discover this other paper by Park and Mosalam from the University of California, Berkeley, it seems that this is the most recent (inaudible 10:17:07) work that's been done and then modelled that is properly relevant for our situation but the main issue is that most of the model was calibrated for a low level of axial load and it calibrates very well but interestingly there is data there but which they kind of have parked and felt that it was too difficult to deal with because they couldn't get their heads around what was the influence of axial load on the performance. So they essentially parked that but they do plot the results and it essentially shows that it seems to support the earlier, I might say very arbitrary it would seem, position that has been held in the United States that beam column joint capacities is somewhat invariant of axial load. Now this is somewhat of a surprise to a lot of people but I think what we're really talking about here is there are two issues.

The first issue is how does the joint behave on the first cycle and I do believe in some respects that is what we have modelled and so therefore one might argue that is enough about.

Then what happens after the very first cycle, it cracks and then it settles down and behaves and it's going to get further in-cycle deterioration but it's after that first cycle I believe that these experimental joint performance that you see by Park and Mosalam is really what they are modelling. And then when you apply their latest formulation, this kind of came up yesterday indirectly through some questioning from Commissioner Fenwick but it's to do with the aspect ratio of the joint. The joints are taller than they are wide and so this actually gives a little bit more capacity to the joint in this particular case and when you do that, when you run the numbers on that, it turns out that the capacity as per this latest information comes out to be remarkably close back to what the FEMA recommendations were. Now in the – I think one of the Compusoft people will be able to find this. They've actually got a plot in their report of that recommendation plus what they have used. Now for the exterior joints – what I'm trying to explain here is none of these results that we've received on the second round at all surprised me. They're totally in support of what I thought might happen and so what I'm really saying is that we seem to think that we're miles apart. We're not very far apart at all. I think one extra analysis might be all that's necessary to fix this and my contention is this, that for the low levels of axial load that the exterior joints have on them, they are modelled about right. There's not a big difference between their first crack prediction and what the, you might say the Park and Mosalam capacities would show and those are also not too far from what my rational mechanics approach would also show. Now at higher levels of axial load however, I think there's quite a divergence between what the two different hypotheses of joint performance would predict and interestingly what they do not show there now is that it appears that the capacity is not capped. So in other words, if you get a very, very high spike in axial load the capacity just kind of keeps on going up. Now, I don't believe they've modelled it that way. I think they should correct me if I'm wrong here, but I believe they've set the level of capacity in the joints based on the static equivalent axial load. So my sense is, and I don't have it in front of me, may be that figure could be brought up so we could discuss it, but I think it's about 40 or 50% higher than it perhaps should be for the interior joints and because we are seeing in the results that the interior joints are indeed failing, and a goodly number of them, but not as many as the exterior, I think if that adjustment had been made then there would be either at least some equity between the interior or exterior frames or in fact the interior frames may well take over and fail more quickly.

JUSTICE COOPER:

Let's get the figure brought up if we know what we're talking about.

MR BRADLEY:

I believe the one that you are referring to Professor is figure C6 and C7 of page 145 of the second report. BUI.MAD249.0552.164

JUSTICE COOPER:

So C6 or C7?

MR BRADLEY:

C6 relates to the interior joints and C7 relates to the exterior joints.

JUSTICE COOPER:

Right. Now Professor Mander can you see that where you are?

PROFESSOR MANDER:

I can but I would appreciate it if interior one was highlighted.

JUSTICE COOPER:

That's the top diagram.

PROFESSOR MANDER:

Yes. Yes so you can see that's a little better thank you. Okay so the joint shear stress intensity coefficient. This I might say I think is PSI units but you just get the number and divide it by 12 and convert it to the capacity unit. The lower storeys are down on the right-hand side where the red and green lines end and converge whereas the upper storey, sorry this is the exterior frame I mean the upper storeys are on where the lines converge. The lower storeys are probably around about the .4 mark or thereabouts and you can see there's almost a factor of two difference in the strength. Now the contention is and a lot of this debate was started by the Compusoft chaps doing some due diligence searching and they found that one of the people that work in this area in California Professor Jack Murray who has kind of spearheaded or led a lot of this work was claiming that the FEMA results which is really plotted I think in the blue line were off by say a factor of two. Now again I would say that's right but if you look at this graph here this is going up quite a long way and it's not capped at all and even using the allowable stress approach, the more circle allowable stress approach which the red line is based upon, if you were to cap it based on those recommendations which is based on some quite old work now and recommendations from Professor Priestley that should be factored in axial range shell of 0.5. All right so even if you could imagine a horizontal or a vertical line at minus point - .5 going up vertically at that level we've got say a factor of two overcooking it. So in reality I'm not so sure that I would say that green is a lower bound but it certainly the first cycle of loading on subsequent loadings after initial cracking. And the only other comment I would make is that it's very unfortunate we don't have more experimental results. Like if the experimental results that are reported there are in a couple of tests of very high levels of axial load but then when you look back those experiments are quite old you know like 30 or 40 years old and I think the reinforcing conditions were quite different back then and configuration was somewhat different, but nevertheless I think this is really the detail that we're getting down to now as to why the differences remain on this figure. I must say I'm very, very much happier with what I see because it is confirming my understanding of what I would have expected to see by doing sort of hand analysis of a different type and I think as I said if you had less capacity on those interior joints then you will probably see those, a similar number fail as to on the exterior frames.

JUSTICE COOPER:
Does anybody wish to respond to what Professor Mander has just said?

COMMISSIONER FENWICK:
If I have got this correct, the joint zones were probably - certainly the internal ones were probably modelled as a higher strength than you would expect given the degradation and so that would not really alter the – you know that would indicate wouldn't it it's more likely to have been a joint failure than a relevance because the joint failure was likely to be weaker than the way it was modelled. Now is that your interpretation?

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PROFESSOR MANDER:
Yes it is, I think that column C2 ends up becoming the indicative column. So you know how it goes, it's either one, it's either the column or the joint will be

the weak link in the chain, and it seems that the column has failed both at the ground level and of course it's likely to have a similar moment at the head of the column. In some respects it doesn't matter if you get a decent sort of hinge at the ground level but if that head of the column it's going to penetrate into the joint, you can imagine that this would additionally deteriorate the joint capacity more as well and neither of those things are being modelled at the present time. So I suppose, and Professor Fenwick you had a question yesterday which I did see online about the length of the plastic hinge zone. I just would like to comment on that because when we first started our discussions we didn't, well, one of the ideas was that we wouldn't possibly need to model the columns as fibre models, rather they could be just modelled purely elastically if in the belief was true that the joints were indeed completely weaker than the columns and but at the same time still have the possibility for capturing that if necessary by having a plastic hinge in there and so how we were planning to do that, and I'm sorry I'm going to have to use this word again as a kluge. What we would've done was to have a kind of a plastic hinge at either end of the column which was really more representative of what the joint was doing. So you could calculate the shear capacity in the joint and then from that back calculate what the moment capacity in the column would mean to provide that joint shear capacity. And then model it accordingly. And the length of the 200 was really representative of what you would get with penetration into the joint below the lap splice at the floor level, so that's how that was set, and I admit if in the present way that this is being mould with fibre models, that's still about right for the floor level but on the soffitt of the beam the top hinge in other words on the column, that probably should be longer. I'm not so sure that it's going to make a big difference though. So in some ways we probably with all these swings and roundabouts, in some ways I believe we are indirectly capturing some of these effects in the column which are really happening in the joint as well. So possibly quite difficult to untangle them, and in reality they're interdependent on each other anyway.

COMMISSIONER FENWICK:

Can we just come back to the joint model for the time being? Okay there's some test results on joints that aren't shear reinforcement but I don't, are there any tests where you have bars bent up in the middle of the joint zone and receptor columns as there were in this case or is. This is a bit of an outlier isn't it? Is it really covered by the sorts of test results you've seen or are you happy to just a lap between the hooks of about what 150 millimetre?

PROFESSOR MANDER:

I, I tend to agree but I do know, based on some work that we did when I was in Buffalo and there's a paper that you could look up. It's listed in my resumé. It's ACI 1994, Accardi, Mander and Reinhorn and it's on this whole topic and that was based on testing some one-third scale specimens and then at Cornell University they did some companion tests with the White Googly and Berries I believe from memory. They did some work that looked at testing essentially what's similar to this situation and quite common of non-seismic design or gravity load construction in the eastern and central United States. Now they were one foot square, (inaudible 10:32:31) 300 millimetres at their columns. They only had four bars running through the joints, not too dissimilar to what we have. And then there was a floor slab and beams. Sometimes there were four way beams. Sometimes there were two way beams. And then the way, the common way that those connections were constructed was to have a six inch embedment with no hook on the end into the joint. Now that six inch or 150 millimetre embedment interestingly under dynamic loads, and this is as verified by shaking table tests and it's not quite so pronounced with static tests, but certainly with the shaking table tests you get full bond strength within that small length of embedment. And I think the reason for that is the columns were fairly heavily loaded. They were unreinforced in the joint shear and they hold on for the first couple of cycles and then they gradually release and then of course after a while they take no capacity. But I was thinking about this too, that even if you got complete release of the hook bar, apart from the seating issues and it hanging together there to support the gravity load on the surrounding slab, there is sufficient top steel both going through the joint and of course all the slab steel that that can hang in there to provide a very large negative moment on one side of the joint that is most probably going to be even still bigger than the combined moments coming in from the columns, particularly if the joints weakening down anyway. So I think it can hang in there and then still deteriorate, and I think looking at the forensic evidence, there was good evidence that the top bars remained intact all the time and for the interior frames there was a lot of beams lying on the ground where the hooks had not appeared to be drawn and straightened out like you might expect to see, so that of course wasn't always the case for the exterior beams. Some of those were badly damaged. And then of course you have to be a little cautious as to how you interpret that because some of that information can be, well some of the damage might be from the fall in the collapse itself.

JUSTICE COOPER:
Does anybody else wish to comment on this issue? Mr Smith?

MR SMITH:
Not on this issue.

JUSTICE COOPER:
Mr Bradley?

MR BRADLEY:
No.

JUSTICE COOPER:
Dr Hyland?

DR HYLAND:
No thanks.

COMMISSIONER FENWICK:

There's one issue but I don't know who would want to consider this but when we had a look at the precast beams on the site it was quite clear there was no roughening of the end of the precast units where they were cast into the columns. In fact, you know, I took a pocket knife and scraped the edge and it was quite powdery and there was clearly no interaction between the, inter-junction at all between the precast beams and the column concrete that was cast in situ. Now those beams were perched on a 25 millimetre ledge of concrete and it looks to me as though there would've been absolutely no shear transfer across that interface, just completely smooth, and I think as Frost and Heywood have pointed out that a bit of pressure there would've pushed the wings off because of the angle of attack and just a kind of rolling smooth surface there, and this would imply to me at any rate that all the shear was probably being tran – would've been transferred through the topping concrete and I just wonder if any of you have thought about that and whether that could be a potential failure mechanism, suddenly loading that shear onto that, that slab concrete at the top? It's the only logical path I can see apart from potential dowel action of the bars at the bottom which of course could potentially be just above a crack. It might not be the most optimum position for action, but it's just a thought whether anyone would like to comment on that?

JUSTICE COOPER:
Any volunteers? Mr Jury? Dr Hyland?

DR HYLAND:
Yeah I mean I could see if you had a, if there wasn't full bond there then, then there's no shear friction significant across that joint, but –

COMMISSIONER FENWICK:
I can assure you that, I mean it amazed me when I got down and scraped it, it just came off like a powder.

DR HYLAND:

Mmm, Mmm, no I understand that.

COMMISSIONER FENWICK:

So there would've been, I mean I really had expected to see some concrete sticking on it but there was absolutely nothing and smooth surface and I say scraped it and it was quite powdery.

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DR HYLAND:

Yes, yes I saw that and yeah I was surprised at that detail as well. But with the beams undergoing just under their gravity loads there would be some, expect there to be some compression in the bottom there closing that joint and therefore you could expect there to be a you know a sort of compression strut sort of coming into that bottom zone on the so I mean I could – I think it had – it had limited its ability but there was obviously something there that could hold it.

COMMISSIONER FENWICK:

It's perched on a 25 millimetre strip, presumably lost the wings by then, so it's quite a high compression stress on there, right on a position I would have thought where it's going to start to spall if you've got an appreciable column actions in there. I'm not saying you did but that's where you'd expect spalling to initiate.

DR HYLAND:

Yeah, I saw that. Again it gets down to what level of drift occurred along those lines at collapse. I guess a contention we've got is that you didn't actually need a lot of drift along those lines to get collapse occurring so it may not have got to that point prior to collapse initiating elsewhere and then consequentially pulling those joints apart. That's my view.

MR SMITH:

If I could just make one comment. I think from a design point of view we often detail precast to sit slightly bored into a column to purely from a practical reason of fitting the form work and things, but I would not rely on the compressive resistance of that cover concrete to hold the beam up, so normally you design the beams on the dowel action of the columns, of the bars that project into the column whether it's a smooth surface you take a lower coefficient of friction or – so that would be a design method. The other point about the slab failure is I thought from what we could see I would expect to see sort of punching failure of the slab around that zone rather than column failure if that interface had given way first maybe, but the columns appear to get all crushed with the floors on top and so I didn't really pick up but where I've seen failures in the past where a column’s projecting through a floor that had failed around it, but you’d had a shear failure obviously of that floor. We didn't see that.

COMMISSIONER FENWICK:

Unfortunately of course the slabs have all disconnected from the beams so one can't see what happened to them.

JUSTICE COOPER:

Do you wish to say anything on that subject Professor Mander?

PROFESSOR MANDER:

The only comment that I would add is that I think Professor Fenwick maybe onto something, I think there's a definite possibility. The only caution I would say is that as pointed out by myself and also before that, by Mr Frost, there would seem to be a fairly decent – sufficient evidence to suggest a complete separation of the slab and the support beams and so you're down to a really, really thin piece that's tying everything together and if that has indeed happened and then I would find it very hard to believe that such a thin slice of concrete going through the beam column connection would really provide too much in the way of shear resistance at all, other than perhaps it acts as a tie that ties everything together before it falls down. Again in my – I presented a few days ago, you may recall that one diagram I had where my hypothesis is that if the columns are indeed, I mean the joints are weaker than the connections then essentially what's happening is that they themselves turn into a fuse and in some respect protect the columns and I think that's one of the reasons why we see so many columns, particularly in the upper parts where they survived the fall, in reasonably good shape, particularly the interior ones from what I can tell, and yet there's no evidence of joint concrete and I agree with what Commissioner Fenwick says about the powder in the joints, like it was astonishing to me that there's just no evidence at all of anything left so it seems to me that everything really just turned into powdery rubble within the joint and that's why it's lost for good.

COMMISSIONER FENWICK:

Can we – I mean the south wall. No one, this doesn't seem to feature in anyone's collapse scenarios. Can we rule out quite definitely that there was no problem with the south wall and the shear transfer into that south wall? It's one of the aspects and you all received the minutes, saying please look at this. Everyone's quite convinced that south wall was – and the load transference it was adequate. Would you like to comment?

DR HYLAND:

I've just got – I mean there is a comment I think it's been covered in the design area about whether the design of it was done correctly, at the time in terms of interpretation of the standard but in terms of the performance of it I don't believe that you know that it caused the collapse in terms of it didn't collapse itself, until something else –

COMMISSIONER FENWICK:

But let's forget about compliance with the standard. The forces we think went into it, could they actually be transferred into that wall. I mean I'm quite intrigued that we have the stack-up slabs at that point as though they just dropped and can we rule out that there was no fault there that might have initiated that type of collapse that we saw?

DR HYLAND:

Yeah I believe, I believe so.

COMMISSIONER FENWICK:

You believe there is? Just for –

DR HYLAND:

Yes, if you're looking at the shear transfer in compatible with the level of damage and the drift associated with that damage in that south wall, I believe there was adequate shear capacity there to get the transfer of load in there.

COMMISSIONER FENWICK:

What about the development of the beams into the wall which was based on four high strength 24 millimetre bars going in I think it was 700 millimetres in length. Was that an adequate anchorage of those bars into that wall?

DR HYLAND:

Well there were places in the collapse debris where it had appeared that the slab had pulled away from the wall elements, you know, you could – along construction joint lines but reflecting on that I think that must have occurred during the deconstruction phase. There was certainly evidence of those H24s where they had been developed into the wall, you know, fracturing.

COMMISSIONER FENWICK:

I'm sorry, I'm struggling to hear you and I'm missing a bit, now the slabs were stacked up one after the other. Now a question I'm putting to you is that the beams which were – the slabs were attached to the beams, were the beams adequately tied into that slab transfer. I mean when the you're transferring the inertial forces through you have to transfer them into the beams and then back into the wall.

DR HYLAND:

Yes.

COMMISSIONER FENWICK:

Now could there have been a connection failure at that point, so the bars just went into the wall, there was no specific lap reinforcement to pick up that force and carry it through the wall, could those – is it possible that those bars actually pulled out of one or more beams which then would have lead to a progressive collapse?

DR HYLAND:

No, I don't – I don't believe so, or that's not what my analysis shows. If you're applying a drift of say .4 percent where you're getting the coupling beams, level 2, level 3, level 4 yielding in shear, which seems consistent with what was seen, on site, at that level the level of shear transfer would be adequate just through the reinforcing steel that was over the length of the shear wall itself so there was adequate capacity there even without the H24s being fully developed in my view.

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COMMISSIONER FENWICK:

So in your view the eight, was it eight or 12 I can't remember now, six millimetre, 12 millimetre bars coming out would have been adequate to transfer the shear?

DR HYLAND:

Well that's what the calculation shows, yes, at that level of drift.

COMMISSIONER FENWICK:

At what level of drift?

DR HYLAND:

.4 percent.

COMMISSIONER FENWICK:

Oh they were talking more about several percent drift I thought in those – in the south wall?

DR HYLAND:

Yes but if you – in the NTHA if you look at the damage in the wall that we see and sort of work back from the damage and say, well what is the level of drift that would be consistent with that damage, it would indicate it was perhaps in the order of.4 percent at the time of collapse and at that level of drift the shear transfer into the wall would have been at a level that would have been quite adequately covered by that level of reinforcement.

COMMISSIONER FENWICK:

Okay and that .4 percent drift would have been adequate to account for the yielding of the bar to the base of the wall would it?

DR HYLAND:

Yes, yes.

COMMISSIONER FENWICK:

That surprised me. Okay thank you.

MR SMITH:

I think just to clarifying something there, the drifts, the 3 percent or whatever of drifts was the maximum throughout the record whereas the failure was indicated to appear prior to that, so you really only need to look at the drifts prior to the failure having occurred because we're looking at really a non – the situation following that is not stable in the analysis basically, as soon as we get one column failing.

COMMISSIONER FENWICK:

So the analysis did not stop when you predicted failure.

MR SMITH:

No it didn't stop as I showed that green line on that plot falling away indicating a column failure, the analysis continued beyond that point but the results beyond that point are not reliable I would say.

COMMISSIONER FENWICK:
So we can be sure that the failure occurred before those large drifts were induced in the wall?

MR SMITH:
Well we can look back at the drifts prior to that point in the analysis, that's all I'm saying, rather than looking at the maximum throughout the record yeah. The point about the slabs being stacked up on the inside. The points I noted in the collapse debris that I inspected was again that there were smooth construction joints in that wall at the underside of the floor and on top of the floor so the wall was poured up to the underside, the slab poured through and then the wall continued again but all of those joints, no roughening, and so we saw that interface where that slab had pulled out. Again the reinforcing bars from the slab were not anchored around the slab, the wall reinforcing, there's no real anchorage out of plane so I think the collapse could have occurred by the slab pulling out of plane and then dropping once it had lost its seating.

DR HYLAND:
That's another observation sir, is if we look at the failure surfaces of the slabs to the beams on line 1 and also actually at line 4, we see a sort of vertical face with the reinforcing steel necked and fractured where it’s come out which indicates a positive or negative bending moment depending on your definition so downwards, a downwards movement. On the interior beams we didn't see evidence of a downward or a negative bending type of failure. We saw as though the slabs had pulled away in the reverse direction which to me would indicate perhaps that the beams were dropping as the slabs pulled away from the line 2 beams.

JUSTICE COOPER:

Mr Jury, do you wish to contribute to that issue?

MR JURY:

Not on that issue Sir.

JUSTICE COOPER:

Professor Mander, anything to add on that issue?

PROFESSOR MANDER:

No Sir.

JUSTICE COOPER:

Dr Bradley.

DR BRADLEY:

No Sir.

JUSTICE COOPER:

Mr Smith was there something you wished to raise?

MR SMITH:

Yes I just wanted to come back to discuss the modelling of the soils again. There was some criticism about the depth that we went into to modelling the soils in the second analysis, or in the first analysis for that matter. I only became – first became aware of that in Mr Rennie's questioning yesterday of Mr Sinclair and I hadn't heard that issue raised in the expert panel so I was surprised to hear that at that stage. I did discuss it with Dr Davidson yesterday and he had some thoughts on it which I thought he might like to give us at this point.

DR DAVIDSON:

Thank you Mr Smith. I heard Dr Bradley's discussion half an hour or so ago and certainly I know he has a greater background than I do, a certainly more recent background. When we started out this modelling process we were sort of using what we would consider state of art for the investigation back I guess close to a year ago when we started with the DBH and the major effect it's our belief that the gapping or the ability for the model to lift off the soil so having no tension capacity in the soil is probably would cover a large effect, and when I say large I mean 80 percent of the effect of the soil. There's no doubt that modelling soil is more common in other types of structures like dam structures, and maybe some very heavy structures, such as nuclear power stations and so on which have very big rafts and they deem to – tend to lower if anything the demands on the structure. In other words my experience and I have not done a lot of it, but I have done this, is that usually modelling the soil with limited strengths and that is some non-linearities that was suggested tends to take away demands on the structure and I would say in that regard not only in the analysis of the CTV building but another analyses that Compusoft engineering have done on structures in the Christchurch region, with our one might say simplistic soil modelling as maybe that's been stated, we do tend to see that the demands on the structures from our analyses tend to exceed what seems to be observed on site. Now there may be other reasons like the non-structural parts of the structure, partitions inside the structure, maybe in this case the masonry, maybe absorbing more energy than our analyses include, but it could also be the soil modelling so there's no doubt that you know with unlimited time and money we could do always better and it's worth talking about the soil but I think in the main I think the soil modelling we've got is covering the major aspects of the behaviour and I think if we did something more sophisticated we would probably see a little less damage in the structure from our modelling.

COMMISSIONER FENWICK:

One feature in the – when you look at the results and I'm sorry, it's some drafts back from this one, I hadn't had a chance to look at this one yet, but the inter-storey drifts seem to be very regular right up, almost uniform right up the building which implies either that there were plastic hinges forming at the base of the walls or that they were rocking. Was there any evidence of disconnection of the soil and rocking. How much did that actually account for the deformations you got?

MR BRADLEY:
Yes, there definitely was rocking both in the north core and in the south wall, that occurs relatively early in the response and it did influence the drifts.

COMMISSIONER FENWICK:
So if this had been designed assuming rigid foundations you would have got very different results?

MR BRADLEY:
Correct.

JUSTICE COOPER:
Dr Bradley.

DR BRADLEY:
Yeah, just a comment on those points from Dr Davidson. I think two points in particular. The first would be that when you do have this rocking behaviour that causes uplift on one side of the wall by definition that means you have reduced contact on the opposite side of the wall which will therefore increase the pressure acting downwards and therefore greater likelihood of non-linear soil behaviour. The other point would be that Dr Davidson noted that he often found there's a reduction in demand. I think that comment is probably made in the context of possibly a reduction in force demands but possibly an increase in displacement demands. Given this structure is probably more sensitive to those displacement demands particularly in the gravity frames which aren't designed for those displacements, I would argue that's more important than the force demands into those elements.
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PROFESSOR CARR:

Yes, as I was cited earlier by Dr Bradley, I might make some comments. That work was based on research work done with (inaudible 10:58:35) element and boundary element methods in the early 1990s. The students found that the soil properties didn't affect the response for the structures and I can't remember whether the period of the structure was shorter than the period of the ground or vice versa. I'll have to go back and look at the data to see on that now, but we also did a lot of research into the 1990s on wall structures which were allowed to rock and that's the uplift situation and that's very important for wall structures on shallow foundations. The fact is how do you stop the rocking. There's no real means of doing so and our observations were, yes the deflections do increase. Maybe not the inter-storey drift per se because the structure's rotating as a rigid body perhaps a wee bit more problematic here because the different columns have different, separate footings and it's like a form of base isolation. It tends to reduce the force response in the structure. As I say, it does increase your deflections. You can't win both. There's a lot of work going on to get better soil footing models which allow for plasticity as well as uplift and that but most of it still is done by moderately simplistic approaches. Inelastic springs maybe. We've had quite a bit of research in the last four of five years using such models which seem to be working out reasonably well. I just wanted to make sure, yes, you can model it but it depends again on what the programs permit you to do.

JUSTICE COOPER:

Was there anything else you wanted to raise Mr Smith [no], Dr Hyland [no], Mr Jury?

MR JURY:

Just a comment really. I heard Professor Mander refer to errors in the first analyses. I'm not aware of any errors. I'm aware of refinements. I'm aware of differences of opinion in modelling but I'm not aware of any errors. I wonder whether Compusoft could comment on whether they are aware of any errors in their earlier work.

MR BRADLEY:
Not that I'm aware of. As I've explained there was some simplifications particularly with regard to the column hinge used but we believe that it adequately captured the building performance and we've seen that basin results of the refined analyses closely matching that of the original.

JUSTICE COOPER:
Professor Mander do you want to comment?

PROFESSOR MANDER:
Well I concede that saying errors is putting it strongly but I think it had the desired effect that if I had not said it that strongly maybe we wouldn't have done all this re-running. So I think it was worth putting it like that. Really I suppose a more correct word would be to say that these were significant shortcomings, the fact that I felt the concrete strength needed to be at least varied and that the beam column joints really did need to be in some way incorporated because a hand analysis showed very quickly that the joints are the weak link in the chain. So I think those to me were the two principal ones and then of course is the interaction I did feel that it was a gross overstatement in the initial runs to come up with the conclusion that the exterior frames were largely affected by the spandrel panels where knowing that the spandrel panels at the ends themselves are relatively flimsy. I felt that that was, if you don't want to call it an error, it's a miss call in that I really don't believe that could have happened, but those are my views there and I do feel that this is still not finalised but it's definitely a big step in my view in the direction because I believe it is actually pointing towards where things are and part of this discussion is how to get it right and I see that time is of course not on our side and it never really is even if you have an infinite amount of time you have to call it quits somewhere so if we did more iterations I think we would make improvements and at the end you have to call it quits when the

marginal improvements, you know, the law of diminishing returns kicks in and you're not getting much improvement for additional iterations and I think we've done one major iteration and that's a major improvement and really I think it would be nice, it's desirable but perhaps not essential, to do one more iteration at least on one earthquake, perhaps at CCCC because that's the one that's been run all the way end on end to look principally at some of these other matters that have been brought up and my own personal one that I feel that is important is again the joints to do some variation on that, run it through with a lower strength, and I do feel that Professor Fenwick has some really excellent points that he has raised that perhaps all of us have been remiss in not looking at a little bit more forcefully and it brings my memory back to work that we did in the Canterbury University labs in the early 2000s where we did the large scale so-called Mathew's test and where the floor slab steel was more or less identical to what we have here and we only did a few cycles of loading and the HRC mesh which I think was 665 mesh from memory in a 75mm topping literally tore apart after a few cycles and the drifts I don't recall what they were but they were very very modest, and so I think it is very true what Professor Fenwick just mentioned that the slab connections over the south wall are very vulnerable and interestingly no matter how much you calculate this, the calculations don't always show what some of these very large-scale experiments that have been done would show if we had the luxury of doing something like that. So really all we can do is fall back on things that have already been done and see if we can implement some of those ideas. So those are my thoughts and I think, you know, maybe the time is now to draw the line and say well if we were to do it again we might see some differences in the joints but at the moment the columns are working as a surrogate for that. It's either one of the two without a doubt in my mind that it failed and led to the collapse and then of course the connections to the walls in both the north and south core are perhaps equally as important and in many ways they're actually more challenging to model because of the way the whole program and software are set up to do this.

JUSTICE COOPER:

Thanks very much. Now I'm somewhat taking my courage in my hands. May I ask whether learned counsel present have any questions they would like to pose to any of these gentlemen.

QUESTIONS FROM COUNSEL REID, CLAY, RENNIE AND ELLIOTT – NIL

JUSTICE COOPER:
Where does that lead us Mr Elliott?

MR ELLIOTT:
Well that would conclude the session Your Honour. It might just be worth doing one check on the matters of areas of disagreement that have been identified in the joint report and I think it appears to me that they are mainly arising from Dr Hyland and it also appears that Dr Hyland has articulated his reasons in his statement but it might be worth just checking to that extent disagreement has been articulated.

JUSTICE COOPER:
Dr Hyland, your views you've set out in summary form haven't you?

DR HYLAND:
Yes.

JUSTICE COOPER:
And Professor Carr as far as you're concerned as the person who organised this, have the areas of disagreement been properly aired?
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PROFESSOR CARR:
I think so Your Honour, yes.

JUSTICE COOPER:
Well then can I just ask Mr Elliott, having reached that point is there anything further we need to do?

MR ELLIOTT:

I do need to ask the experts to sign the final version so if they can remain.

JUSTICE COOPER:

That can be organised, yes.

MR ELLIOTT:

But no Your Honour, not beyond, of course we have the ERSA session to move onto next and some of the same experts are staying for that. So perhaps if we could take the morning adjournment next.

JUSTICE COOPER:

Well if we take the morning adjournment now. Professor Mander you're not involved in the ERSA are you?

MR ELLIOTT:

He is Your Honour. He was.

PROFESSOR MANDER:

I think I'm on that but I really don't have a lot to participate with, unless you really would like me to participate. Like I've been a, let's shall we say a silent partner on that and I'm very relaxed about issues on that. I don't think I've dissented from anything and I don't have a lot to contribute, so unless you would like me I can either stay or go, either way, whatever is your pleasure?

JUSTICE COOPER:

Mr Rennie, what do you think?

MR RENNIE:

Well Sir I am essentially in the hands of the professor in terms of his judgement as to how he can contribute and our primary participation in the ERSA is through work that Mr Latham has done, with no disrespect to the professor. The only other matter I'd mention Sir which is not quite on topic is that I have received authority from Professor Mander to sign the experts' report on his behalf and you might just with him to tell you that directly before the video link is broken Sir?

JUSTICE COOPER:
Yes, yes did you hear that Professor Mander?

PROFESSOR MANDER:
Yes Sir, I did, I'm very happy for Mr Rennie to sign on my behalf.

JUSTICE COOPER:
All right, I think we will release you Professor Mander until we meet again. Thank you.

HEARING ADJOURNS: 11.10 AM

HEARING RESUMES: 11.28 AM

MR ELLIOTT:
Commissioners, the next session deals with ERSA which stands for Elastic Response Spectra Analysis, I'm told, and that is a computer modelling process was used by Dr Hyland and Mr Smith as a means by which to assess whether the CTV building was compliant with code. I understand, although Professor Carr could correct me at the right time, that it's a form of successor to ETABS which was the modelling used at the time by Alan Reay Consultants Limited.

JUSTICE COOPER:
Mr Elliott, you're not all that easy to hear at the moment. You need to get closer to the sweet spot.

MR ELLIOTT:
How is that Your Honour?

JUSTICE COOPER:
That's much better, but can you keep it up?

MR ELLIOTT:
Should I repeat what I've said?

JUSTICE COOPER:
Yes.

MR ELLIOTT:
I've just outlined that the Elastic Response Spectra Analysis was used by Dr Hyland and Mr Smith as a means by which to consider whether the CTV building was compliant with the applicable code and as I understand it the ERSA is a sort of successor to ETABS which was the computer programme used by Alan Reay Consultants Limited back at the time of design. So the manner in which Dr Hyland and Mr Smith used it was to assess the extent of inter-storey drift and to assess column capacity as against that drift. And again, as with the non-linear time history analysis there were inputs involved and so therefore some scope for debate and discussion about the selection of the inputs. For that reason the Royal Commission again directed that experts confer and the Commission made an order, a copy of which is at tab 4 of the Commissioner's bundle. That order was dated 18 June 2012 and I'll just refer to one section of that BUI.MAD249.0533.2.
If paragraph 4 could be enlarged please. So the purposes the experts were directed to confer, they were directed to endeavour to reach agreement on the input data to be used to conduct an ERSA of the response of the building to determine whether the design was consistent with the provisions of the applicable codes. Secondly, where agreement could not be reached on the inputs, to identify those inputs which could not be agreed and the reasons for

the disagreement and as a further purpose to produce ERSA results which provide the most reliable model for the purposes set out in the previous clause 4.1 and which could then be analysed and interpreted. That was the purpose. The experts conferred and they produced a joint report for the Commission, a copy of which appears at tab 5. Your Honour will see that that has not been signed by everyone and the copy that the Commissioners have but I have now obtained signatures by all except Mr McVerry but I think again his involvement was minimal.

JUSTICE COOPER:

We have one that's signed by some but not by others. But anyway, it's all signed except for Dr McVerry.

MR ELLIOTT:

Yes Your Honour, my copy is, yes.

JUSTICE COOPER:

Thank you.

MR ELLIOTT:

And as was the case with the previous session, Professor Carr has also produced an independent report effectively with a record of discussions and that appears at tab 6 of the Commissioners' bundle and once again I wasn't intending to ask Professor Carr to read that out but it's there as a record for the Commission but I will be inviting Professor Carr shortly to read out his joint report and then in the same way to the extent there was disagreement that can be canvassed between the experts. Your Honour, the experts have now been joined by Mr Doug Latham who appears to Mr Smith's right. Just by way of brief introduction, Mr Latham has a bachelor of engineering degree with honours. He's a graduate member of IPENZ. He's employed with Alan Reay Consultants Limited as a structural engineer and during his time at the firm he has conducted or been involved with various linear elastic structural analyses of multi-storey buildings. Your Honour he will need to be sworn or affirmed and asked to confirm if he's read the code of conduct.

DOUGLAS LATHAM (AFFIRMED)

MR ELLIOTT:
Mr Latham, if you could confirm that you've read the code of conduct for expert witnesses and agree to comply with it.

MR LATHAM:
Yes, I've read it and I agree to comply.

MR ELLIOTT:
I therefore invite Professor Carr to present his report Your Honour.

JUSTICE COOPER:
Thank you Professor Carr.

PROFESSOR CARR:
This is a joint report of the Elastic Response Spectra Analysis panel. Introduction. This is the joint report pursuant to clause 9 of the order of the Royal Commission dated the 18th of June 2012 in relation to Elastic Response Spectra Analysis (ERSA). The purpose of the experts conferring were to endeavour to reach agreement on the input data to be used to conduct an ERSA or analysis of the response of the CTV building to determine whether the design of the building was consistent with the provisions of NZS11 3101:1982 and NZS 4203:1984.
2.2 Where agreement cannot be reached on the inputs, to identify the inputs which cannot be agreed and the reasons for the disagreement. Then to produce ERSA results which provide the most reliable model for the purposes set out in clause 1 to which can then be analysed and interpreted.

Topics. This report addresses the following topics.

3.1 Areas of agreement.

3.2 Areas of disagreement, including the reasons for the disagreement.

3.3 The results of any further Elastic Response Spectra analyses.

Areas of agreement.

Use of the Elastic Response Spectra Analysis (ERSA). From the lack of response from the panel, I had about two emails in total I think that occurred during the duration, it appears that all members of the panel were comfortable with an Elastic Response Spectra Analysis being used to verify the degree of code compliance of the CTV building. I did have comment from Mr Latham. He queried whether an Elastic Response Spectra Analysis was required by the design standard of the time but this panel was asked to consider the data for an Elastic Response Spectra Analysis and not whether it was, or was not, needed.

Clause 3.4.7.1 of NSZ4203:1984, was obviously not written to deal with buildings that were dominated by a torsional response. Sub-clause (b) suggests that a two dimensional Elastic Response Spectra Analysis could be used instead of an equivalent static analysis. This two dimensional Elastic Response Spectra Analysis would not pick up the torsion responses at all. Sub-clause (b) then did recommend that a three dimensional Elastic Response Spectra Analysis could be used and I have noted that the Alan Reay Consultants Limited designer did in fact use the three dimensional Elastic Response Spectra Analysis.

Point 6, I believe that the current standard NZS1170-5:2004 does not necessarily deal with torsionally dominated structures any better than the 1984 standard. The current standard requires the number of modes considered to be enough to represent 90% of the translational inertia or mass of the structure. The torsional inertia doesn’t have units of mass. It has units of mass times length squared so it’s a bit like comparing apples with oranges, you’ve left the oranges out. Further consideration needs to be given to buildings whose fundamental response is in a torsional mode and I can make a wee comment, I’ve always told students that if their mode one is torsion, go back and think again. It’s not a good structure.

Areas of disagreement.

Weights used in the analyses.

There was a degree of disagreement between Mr Latham and Dr Hyland on the weights used in the Compusoft elastic response spectra analysis that referred to in the Hyland Smith report and those used in the original design by Allan Reay Consultants Limited, ARCL. Mr Latham regarded that the weights of certain elements assumed on the Compusoft analyses were greater than those used in the original design. He considered that the overall building mass was therefore overestimated by approximately 5%. Mr Smith countered that the calculated weights differ by less than one percent from those used in the original analyses. It is important to note the original calculation of the building mass was likely to have been carried out before the building was fully designed.

Foundation stiffness.

Mr Latham requested that the original Geotech Consulting Limited, Geotech, soil stiffness values supplied at the time that the building was designed to be used in the analyses rather than the Tonkin & Taylor Limited values used in the Compusoft elastic response spectra analyses. The Geotech soil stiffness values are smaller than those supplied by Tonkin & Taylor and which were used in the Hyland Smith report. A lower foundation stiffness would give a longer natural period of free vibration for the structure. This would imply smaller design actions and possibly smaller inter-storey drifts. It appears from the Allan Reay Consultant Limited calculations that the original analyses assumed a rigid foundation, i.e. the foundation is infinitely stiffer than that used for the Hyland Smith report.

Point 9. the analysis programme ETABS used at the time of the original design would need to use a dummy storey below the building to represent a foundation, or foundation flexibility and there was no evidence in the design calculations that this was done. This however cannot be confirmed unless the original ETABS input or output is available. Dr Hyland found similar variations in responses when he conducted analyses, including and excluding the group 2 gravity only frames within the computational model. Dr Hyland commented that the stiffer structure leads to higher frequency response and the more flexible structure leads to a lower frequency response, resulting in similar demands. In the context of a subsequent capacity design process the differences do not appear to be significant. The original design model appears to have excluded the gravity only parts of the structure, and this was also the basis of the Compusoft elastic response spectra analysis used for code compliance checks in the Hyland Smith report.

A.

Scaling of results.

The standard requires that the elastic response spectra analysis be scaled if the actions from the elastic response spectra analysis were less than a specified fraction of those obtained from the equivalent static analysis. Mr Latham queried whether the scaling used by Dr Hyland was in accordance with the 1984 loading standard. Mr Smith responded by providing a revised table with corrections to two of the base shears and acknowledged the scaling was not exact but within a few percent which he considered reasonably good agreement.

11. a static analysis is much simpler than the elastic response spectra analysis and as a result is a little more conservative in the design actions used, i.e. the equivalent static design actions are somewhat greater than those obtained from the elastic response spectra analysis. An example where they were not was at the south wall. Dr Hyland commented that his understanding was that the elastic response spectra analysis drifts should be used if they are greater than the static analysis drifts. This is a little problematic in that theoretically the elastic response spectra drifts calculated the difference between maximum floor displacements and not necessarily the maximum drifts between floors unless a point drift option is used in the ETABS analysis. This option was not available in the ETABS programme used at the time of the design. However, Dry Hyland's checks indicated that the direct use of the elastic response spectra drifts are very similar to the point drifts. Dr Hyland thought that it was also better to use the elastic response spectra analysis drifts and perhaps make an allowance for additional effects

rather than ignore them on a technicality. The scaling was done in accordance with the requirements of the standards NZS4203 1984. Mr Latham – oh, sorry, –

Results of any further elastic response spectra analysis.

Mr Latham requested a further elastic response spectra analysis to be undertaken using the Geotech soil stiffnesses and the corrected masses. With little response from the panel to the discussions, and given that the different foundation stiffnesses would have compensating effects on the analysis, Dr Hyland, Mr Smith and myself consider that a further elastic response spectra analysis was not warranted. The Compusoft elastic response spectra analysis input files were transmitted to Allan Reay Consultants so that Allan Reay Consultants could review the inputs and carry out further elastic response spectra analyses with modified inputs if they wished. And that's the end of the report.

JUSTICE COOPER:

Just in relation to that last paragraph Professor Carr, the second sentence?

PROFESSOR CARR:

Yes.

JUSTICE COOPER:

Where different foundation stiffnesses would have compensating effects on the analyses. "Compensating effects," could you just explain that for me?

PROFESSOR CARR:

Well you're going to have a longer natural period. You'll have slightly decreased actions in the structure but it depends where on the spectra you happen to be but I suspect that the actions would decrease a little bit, but you'll have now a softer structure which will probably have slightly bigger deflections. So it was felt to be a compensating type, it would not make a great difference to the drifts.

COMMISSIONER FENWICK:

But normally you'd expect those drifts to increase wouldn't you?

PROFESSOR CARR:

If it's foundation it's a rigid body rotation. It depends how you measure inter-storey drifts in those cases because part of it is actually a rigid body rotation rather than deformation in the structure.

COMMISSIONER FENWICK:

In this case though you have foundation pads. I mean that's true for the wall itself, but we –

PROFESSOR CARR:

Yes, for the foundation pads you'll get –

COMMISSIONER FENWICK:

We have the beams and so on?

PROFESSOR CARR:

You'll have pretty much the same drifts probably in that case. One lot would increase it, the other lot will decrease it, so it was felt to be a compensating effect and therefore not of great concern.

JUSTICE COOPER:

Mr Palmer?

MR PALMER:

(inaudible 11:46:01)

JUSTICE COOPER:

Dr Bradley, Mr Palmer has kindly indicated that you might be able to contribute to the discussion which has taken place between Commissioner Fenwick and Professor Carr?

DR BRADLEY:

Yes, I think both views are respected in that regard. One comment I would make is that it is at least my understanding that the values of soil properties that have been used in the ERSA analysis are those provided by Mr Sinclair and they were for the dynamic properties of the soil. I understand however in the 1986 design of the structure the values were provided by Mr McCahon and they were for the static properties of the soil. From previous evidence and values provided by Mr McCahon and Mr Sinclair it's relatively obvious that those values from Mr Sinclair are on the order of two or more times larger, and therefore there seems to be some difference in terms of how this ERSA model has been set up as of 2011 compared with probably that which would have been set up in 1986.

COMMISSIONER FENWICK:

Can you go on to tell us what effects do you think it would have in using the long-term softer values on the analysis results? What do you think?

DR BRADLEY:

In terms of the design demands on the structure itself, at a period of approximately one second the CTV building sits on the descending branch of the design spectrum and therefore as the effective period increases, which would be the case if using softer values of the soil stiffness are consistent with those which would have been supplied to the designer by Mr McCann were used, that would result in an elongation of the period of the structure and therefore a reduction in the design spectral acceleration ie the design forces would have been reduced.

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COMMISSIONER FENWICK:

And the effect on the displacements?

DR BRADLEY:

By definition the displacements would increase.

COMMISSIONER FENWICK:

And which feature do you think would be critical, the displacements or the forces?

DR BRADLEY:

I think that would be dependent in particular on do you reach for example does that increase in displacements cause you to go beyond a point at which you would then have to design that gravity system for seismic loading?

COMMISSIONER FENWICK:

In that terms the displacement would be critical?

DR BRADLEY:

In that regard yes.

MR PALMER:

I've been looking for an opportunity but haven't found one but now's as good as any particularly relevant to these questions. Just so that you're aware the questions that have been asked may be answered by some work that Mr Latham has done, I don't know whether you've seen it and I apologise sincerely for the lateness but we got caught by the twin problems of having the panel sign off without another ERSA being run and this ERSA panel being brought forward to today that we originally thought that it was going to be next week and that the intention was to file some evidence by the end of this week relevant to the very issues that you're talking about now. I understand that it's on the shared website but Mr Latham has prepared a second statement of evidence and noting that the last paragraph of Dr Carr's report that inputs were provided to Alan Reay Consultants Limited. Mr Latham has busily re-run the ERSA using the different inputs as Dr Bradley said from Mr McCann's work in 1986 and he has prepared a report and attached that to a second statement of evidence and I again apologise for the lateness of this but we did the best we could in the circumstances. I don't know how to or what to suggest as to how to go forward on this. Mr Latham would be in a position to read it today but I imagine that nobody else has seen it either but it is certainly evidence which is germane to what we're talking about now and which will perhaps be relevant to the Code of Compliance section later in the hearing schedule.

JUSTICE COOPER:

Yes, well that's the way – we've heard about this and we've been wondering when you would be seeking leave to introduce it, but it's not in accordance with the way we want our proceedings to run or statements to be made, which others haven't had an opportunity to read and –

MR PALMER:

Yes Sir.

JUSTICE COOPER:

- your comment on.

MR PALMER:

You will appreciate that this session was slightly earlier than intended and Mr Latham –

JUSTICE COOPER:

I don't, I don't actually but I mean that's – I didn't – I thought we always intended to deal with the two matters on which we'd asked the experts to confer together but I don't know that I'm right on that. Do we need – we don't need to –

MR PALMER:

We don't need to go there so.

JUSTICE COOPER:

So we'll deal with that in due course, probably during the design phase evidence. Is that –

MR PALMER:

Well in the Code Compliance section, Mr Latham's due to come back for that. Is that as you see it Mr Mills?

MR MILLS:

Well that's certainly what I proposed and I don't want to make a big issue about this changed schedule but I think this has always been scheduled here but certainly in terms of where to go from here, my understanding is that these ERSA issues beyond what has been reported today are being raised on behalf of Alan Reay Consultants in respect of Code Compliance issues. That seems to me to be the logical place to accommodate this and then allows everyone who's affected to have time to properly read these new briefs and consider what they might be saying and how they might be responded to.

JUSTICE COOPER:

Well the only thing that would give me pause would be if there's a suggestion that what we're intending to do today would misfire because of this other evidence.

MR MILLS:

It's certainly not my understanding. I think what we're hearing today is the response to the direction given by the Commission about what was to be done by the expert panel, facilitated by Professor Carr and the additional issues which have now been picked up which that panel chose not to pursue, about getting another ERSA done with different data, because that does relate as I understand it directly to Code Compliance issues it seems to me on my knowledge of it any rate that that can be deferred and that's the proper place to deal with it.

JUSTICE COOPER :
Well that'll be – that's as we all see it too.

MR PALMER:
I'm happy with that Sir but just to point out that it's not quite as simple as that like all things there are complexities and one is that Mr Latham I imagine will draw upon the work that he's done and will present later during the hearing in his answers on the ERSA panel today but I imagine that he'll be able to give enough of an explanation in his verbal responses to inform the Commission about what he's discussing.

JUSTICE COOPER:
So I think this takes us to, sensibly to the areas of disagreement that are identified, the first of which is weights used in the analysis and Mr Latham's proposition is that the weights of certain elements assumed in the Compusoft analyses were greater than those used in the original design resulting in an over-estimation of the overall building mass by approximately 5 percent and Mr Smith considers that the difference is less than 1 percent so there's an obvious disagreement. Can I – shall we hear from you on that Mr Latham?

MR LATHAM:
Yes, we're comparing different things there. There's one, the original mass used in the original design which would have been calculated before the building design was finished because it's in effect one of the first steps you must work out before you can continue with your design. There's another mass that Mr Smith has used for their analysis and those two masses are within one percent according to Mr Smith.

JUSTICE COOPER:
You don't disagree with that?

MR LATHAM:

I don't disagree with that number no. I have calculated what I think the building mass should be and I get a different answer from Mr Smith by 5 percent. My answer is also different from the original maths used in the original design so there's three different masses there hence the different numbers.

JUSTICE COOPER:
Yes, well what is the significance of this Mr Latham?

MR LATHAM:
So the mass of a building is, or the design forces of the building are directly apportional to the mass so reducing the mass by 5 percent reduces the design loads by 5 percent, it reduces the design deflections by 5 percent, so therefore I think that should be considered. The basis of my mass calculation is using product data that was available at the time. That was also used in the original calculations, there's a factor called a superimposed dead load. That allows for the weights of partitions, ceilings, mechanical services. The number that was used in the original design was 0.5 kpa, the number, and that's the number I have used in my calculation, the number used in the Compusoft ERSA was inflated from that figure and I do not know the reason why.

JUSTICE COOPER:
What was the figure, do you recall?

MR LATHAM:
I believe it was 0.55 kpa.

JUSTICE COOPER:
So these are for what elements is this?

MR LATHAM:
That was for the superimposed dead load. The other difference was the weight of the floor, so there was in the main part of the structure it had a 200

thick Hibond floor. Now I have a copy of the product literature that was applicable at the time which I sent around to the panel and that recommended a weight of 4.0 KPA for a 200 thick floor. That is not the number that was used in the Compusoft ERSA analysis.

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JUSTICE COOPER:

What was their number?

MR LATHAM:

I'm not sure on the exact number Sir. I believe it was around 4.2 to 4.3 KPA. The third one is the weight of –

JUSTICE COOPER:

Just on that floor weight, your 4 KPA, was that the same as was used by Mr Harding?

MR LATHAM:

Yes, yes it was.

JUSTICE COOPER:

Yes, well the next element.

MR LATHAM:

The next one was the density of concrete. In my mass calculation I used a density of 23.5 kilonewtons per cubic metre which was the same density assumed by Mr Harding in the original design and it appears that the number in the Compusoft ERSA was 24 kilonewtons per cubic metre.

MR ELLIOTT ADDRESSES THE COMMISSION

Excuse me Your Honour. I asked Compusoft to stay. Mr Bradley has just said he could respond to these points if necessary so I would invite him to return to the witness box please.

MR BRADLEY RETURNS TO EXPERT PANEL AND AFFIRMS

JUSTICE COOPER:

So we've got density of concrete. Is there another element?

MR LATHAM:

Those are the primary differences and I guess it's just how you add them all up. Now I've done that. That is provided in the report that was submitted yesterday which has been talked about earlier so that will be available for other people to review as to how I've determined my answer.

JUSTICE COOPER:

You spoke of there being three figures: those of Mr Harding, your own and Compusoft's. In terms of the individual elements about which you've just advised us, there doesn't seem to be any difference between you and Mr Harding –

MR LATHAM:

That's correct.

JUSTICE COOPER:

That I take it the difference then comes in, in how you do the addition?

MR LATHAM:

That's correct and so when Mr Harding would have calculated his mass the building was not designed so he couldn't necessarily be a hundred percent sure on all the geometry of the structure and so he probably would have made some simplifying assumptions and come up with a number that was greater than what could be justified.

JUSTICE COOPER:

These analyses are undertaken aren't they for the primary purpose of demonstrating compliance with the standards?

MR LATHAM:

Yes.

JUSTICE COOPER:

So speaking as the only non-engineer on the panel why is that required to be done at a time when the structure is not fully designed, leading to what you now report as significant inaccuracy?

MR LATHAM:

It's part of the design process, Sir. I mean to start off with there's a lot of things that you don't necessarily know for certain. Generally you'll make some assumptions about those things. You'll then carry out your design and then you need to go back and check.

JUSTICE COOPER:

You need to recalculate it?

MR LATHAM:

Well you need to check your assumptions and what that means in terms of your design. Now in this case if Mr Harding's mass was greater than what could be justified, you could make the decision that no I don't need to recalculate it or redo all the analysis because it would result in conservative loads but if you're looking at what could be justified then he had the potential to go back and recalculate the mass and in effect make any changes accordingly.

JUSTICE COOPER:

Well then going back to paragraph 7 of this joint statement Mr Smith countered that the calculated weights differ by less than 1% than those used in the original analyses. Mr Smith that calculation was presumably based on the structure as you understood it to be built?

MR SMITH:
That's correct, well just explaining –

JUSTICE COOPER:
Well the finished design I mean.

MR SMITH:
As designed, yes, just explaining the process that we took. Mr Bradley and myself independently calculated the weights of typical floors and found ourselves to come to an independent agreement about that number and then when comparing it with the original calculations, again very good agreement within one percent I think I said. So we considered that we had independently come to agreement about the seismic weights that would normally be used for design. There may be variations, as Mr Latham has pointed out, between these three items. We haven't had an opportunity to review his full summary of where all the weights come from but we're happy to do that before the code compliance session. But I think the point I'd make is I believe we're talking about the difference between a process that might be used for design where I wouldn't be at all surprised if you were five percent higher than what was actually there because that is a common process of adding up what you believe the weights of various elements and in my experience there is invariably elements that you hadn't considered at that time. So five percent over estimation is not uncommon from my experience in the design sense, but I guess Mr Latham is saying at this point it may be critical that we refine that and he's got varying opinion but we haven't had a chance to review that in detail.

JUSTICE COOPER:
Mr Bradley is there anything you wish to add to that?

MR BRADLEY:
Yeah, I think one of the differences is, you know, this density in concrete that was the difference between the two. When I assessed my weights I used

what is a generally accepted density of 24 kilonewtons per cubic metre. Now in reality that number will vary and it will vary across the country depending on what aggregates you have in your concrete. My brief was to create analysis model that would be appropriate for design at that time using the stiffness parameters that were appropriate for 1984. Now when I did that I obviously just went through and used 24 as being a standard density to use for when you're assessing something like this or if you're going about design. I would note that quite often you do your best to allow for as much weight in the appropriate weight distribution in a structure but there are a number of things that can add to that like for instance there could be ponding. You specify a minimum thickness of slabs, say 200mm. In reality it varies from 205 to 210 or 200 or 195 so it's common practice to actually make allowances for a bit of ponding and stuff like that and generally engineers would err on perhaps the conservative side and without knowing the exact thicknesses of the floor it's difficult to say what the exact weight of that building would be, but the approach we took was to take one as if you were designing that building now or modelling that building now what would you go about doing and that's what we did. I agree with Mr Smith that it's not uncommon to get variances, one engineer added them up and another one a few percent is neither here nor there. One thing I would say though that Mr Latham's assertion that a 5 percent change in mass would lead to a 5 percent drop in demands isn't quite correct because it depends on where you are on the force spectra or the load spectra. If you drop the mass you increase the relative stiffness of the building, the period drops, therefore the demands actually also increase so you chase your tail a little bit, it's not a direct correlation, there would be a drop but it's – a 5 percent drop would not lead to a 5 percent decrease in demand.

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JUSTICE COOPER:

Do you disagree with that Mr Latham?

MR LATHAM:

Yeah, again it depends on where you are on, or your building period. The spectrums in effect got two flat portions and one range where that spectrum is on a diagonal so if you're on that diagonal range then yes what Mr Bradley's saying is correct, if you're on the flat range then what I consider it is a direct – it is a direct drop but it is dependent on where you are, what your period of your building is.

COMMISSIONER FENWICK:

And that's – get this straight, your period range was of the order of one second and the flat range goes from .45 seconds, 0.45 and from 1.2 up so you're on the sloping range exactly as –

MR LATHAM:

Yeah, so if I just respond to that. With softer soils that period, or the period of the structure is in effect going to lengthen, so in effect using the recommendations by Mr McCahon, the original Geotech engineer on the building, his recommendations result in a period that is longer than 1.2 seconds so therefore you are getting that direct drop.

COMMISSIONER FENWICK:

Can I just comment on the concrete strengths. My understanding was that 23.5 was the standard value for Christchurch aggregates, provided you didn't have a lot of reinforcement in the section, and 24 is pretty well the standard value where they have a higher density aggregate in Auckland, again provided you didn't – you had a reasonable reinforcement content. If you have a high reinforcement content it's too low so you've always got to look at that. If you want to get down to the 1 or 2 percent that we seem to be discussing now and I'm not quite sure why but 1 or 2 percent we haven't an answer, but if you want to get to that level then you have to start allowing for the reinforcement content as well.

MR BRADLEY:

I agree.

MR LATHAM:

So one comment I can make, there was some density tests done in the Hyland Smith report, I can't recall the exact figures but they were – my understanding is the general average was around the 23.0 mark and so that would then give you some allowance for reinforcing on top of that.

MR BRADLEY:

I guess it comes down to the approach of whether you're looking at it from a design perspective where you don't know the density and whether you're looking at it for sort of a forensic and in hindsight where you can actually ascertain what that density is. In design you just generally take general guideline, and engineers tend to, if there was a variance from 23 and a half or 24 most engineers would probably take the 24.

JUSTICE COOPER:

As I understand what's being said, Mr Harding took the 23.5 kilonewton per cubic metre figure and that may have been his standard approach.

UNKNOWN:

Is that correct that he did?

JUSTICE COOPER:

In Christchurch?

UNKNOWN:

Can we confirm that he did actually use the 23 and a half?

JUSTICE COOPER:

Is that the case? I think that's what was said before.

MR LATHAM:

Yeah, I mean if I can – we can look at the calculations if you wish.

JUSTICE COOPER:
Well you've told us haven't you that that's what he used?

MR LATHAM:
Yes, that is correct he did use 23.5, that's evident in the calculations that he prepared.

JUSTICE COOPER:
I was just reporting what you had said a few minutes ago. Dr Bradley?

DR BRADLEY:
I think there's an important distinction that needs to be made here in regard to whether the question is were the inputs required correctly, and then the second question is given the inputs into the model and the forces that the model, the ERSA model predicts does the detailing of the structure then comply given those forces. So I think if it is acknowledged for example 23.5 kilonewtons per cubic metre is an appropriate value then I think given that Mr Harding used that value it would seem appropriate that that is then used in this ERSA analysis to obtain demand forces similar to what he would have obtained rather than taking the view that 24 is a relatively ballpark figure and maybe something which we would use today. My understanding is the idea is to replicate as close as possible that which Mr Harding would have used.

DR HYLAND:
Is that really correct?

JUSTICE COOPER:
Well it depends for what purpose this is being done, if we're seeing it as a Code Compliance issue that does sound to me like it might be the correct approach but we haven't got a firm conclusion to that effect.

DR HYLAND:

I mean I guess the counter to some of this is what would have been reasonable design practice at the time in terms of considering estimating super-imposed dead loads and allowances for ponding and unknowns during the design process and one may take a very minimalistic approach which may have been taken here to perhaps not account for ponding and not into, to consider perhaps very low bottom end consideration superimposed dead load but is that what would be normally done. I think what we ended up with is something in the analysis which is close to what Mr Harding used. I guess there's some debate about whether that was perhaps sharpening the pencil as much as you could and whether a check by another engineer using perhaps normal or perhaps slightly higher superimposed loads would perhaps also be quite consistent with practice at the time.

JUSTICE COOPER:

Can I just enquire whether this is an issue that's going to be addressed by Mr O'Leary or Mr O'Loughlin, Mr Reid?

MR REID:

Well certainly part of Mr O'Leary's evidence is for the purposes of compliance it's important to try and replicate what would have been done at the time, so he addresses that specifically.

JUSTICE COOPER:

Does he descend into the detail of the ERSA analysis?

MR REID:

No, no and he's – I mean he'll have the opportunity no doubt to review it before the compliance section. I'm not sure what he'll make of it.

JUSTICE COOPER:

Mr Bradley, this will be no criticism of you if you didn't have these figures but are you in a position to respond to what's being said about (inaudible 12:15:55) Mr Latham. I haven't asked you Mr Latham whether you're a Dr Latham?

MR LATHAM:
No, just a Mister Sir.

JUSTICE COOPER:
Just a Mister. You're in quite good company. These figures that have been quoted, the differences in the inputs?

MR LATHAM:
I obviously don't – it's a while ago since I've done it but as a designer I wouldn't have complaints. A few percent difference in building mass, it's neither here nor there from a design perspective if you were doing a design at the time, if you reviewed a designer I don't think you'd complain that your mass was 2 percent or 1 percent different or in that order of magnitude.

JUSTICE COOPER:
All right, well can we take that any further? Is there anything –

PROFESSOR CARR:
I'd just make a brief comment is that a 5 percent difference in mass gives a two and a half percent difference in natural period. It works its way down a wee bit but there are differences. Engineers don't all think exactly the same.

JUSTICE COOPER:
Yes is that – does that open the floor to you Dr Bradley?

DR BRADLEY:

Yes Sir, I think maybe as a way of for the Commission to move forward would be to note that this difference in density of concrete is going to result in a relatively uniform increase or decrease of the mass throughout the structure and therefore one could take the current values and simply multiply them by the difference in design spectra acceleration that you would have using these two assumptions so it wouldn't require two completely different sets of analyses to be (overtalking 12:17:46) so I think the Commission could move forward with that in hindsight when reviewing the result.

JUSTICE COOPER:

Thanks very much. But you – I thought you might be wanting to respond to what Professor Carr has just said about the significance of this?

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MR LATHAM:

I agree with Professor Carr. It's a square root relationship but it would be closer to 2.3% rather than 2.5 if you want to be pedantic Sir. I was using mental arithmetic.

JUSTICE COOPER:

So it's slightly less significant than Professor Carr thought?

MR LATHAM:

That's correct.

JUSTICE COOPER:

2.3 %?

MR LATHAM:

Approximately.

JUSTICE COOPER:

To one decimal place. That's a few seconds?

MR LATHAM:
Yes.

JUSTICE COOPER:
Well is there anything else anybody wishes to raise about that area of disagreement?

MR LATHAM:
The only thing I would say is if the mass that was used in the Compusoft ERSA, if they took those three items, the concrete, the Hibond floor and the SDL and used the values that I have used and were used in the original design I'm sure that they would get very close agreement to what the number that I have come up with.

JUSTICE COOPER:
Right well it's just a matter of arithmetic isn't it at that point?

COMMISSIONER CARTER:
What allowance one makes with a definite thickness of the slab. I mean it's certainly one thing we do know, it won't be precisely 200 millimetres.

MR LATHAM:
Yes, yes that's correct.

COMMISSIONER CARTER:
So in fact using, using a number that comes out of a chart in a handbook might be a convenient way to establish a number but it has no exactness related to it.

MR LATHAM:
No, no it does not in that sense.

COMMISSIONER CARTER:

So therefore, you know, one, whether one should be conservative in that regard or not immediately comes to mind.

MR LATHAM:

Yep, so I mean in that respect I mean effects such as ponding, which would increase the allowance or increase the weight. In this particular case the floor was propped and it was pre-cambered so it was built with a, or should've been built with a hog, and so those effects would be minimal. I also understand there was some floor measurements taken of the actual thickness. I can't recall those exact values but they were consistent with a 200 thick floor. I believe two of the numbers were under 200 and one number was above 200. I don't know where those measurements were taken from or what –

COMMISSIONER CARTER:

Effectively we don't exactly know what the thickness of the frame is?

MR LATHAM:

No you don't know, all you can do is say what a designer would've reasonably been expected to do. There's literature from the manufacturer of the floor system which provides a recommendation and I guess then the question is can one rely on that recommendation or not?

COMMISSIONER CARTER:

And I think we also need to just note that the mass of this building was dominated by the weight of the slabs.

MR LATHAM:

Yes.

COMMISSIONER CARTER:

The extra weight in other vertical elements and beam thickening et cetera would be quite minimal so the question of the accuracy of that slab calculation, it's either there for the point of view of a designer of the structure, or for the triggering of a particular requirement in the code which is something I think that you intend to develop later. There's two particular points of emphasis for thinking about this matter, and I think they're probably thought of differently, whether you're a designer or whether you're actually calculating how to utilise the code. Is that, do you agree with that sum – that summary?

MR LATHAM:
Yes.

DR HYLAND:
Can I comment?

JUSTICE COOPER:
Yes?

DR HYLAND:
Just because I think Mr Latham there said that there wouldn't have been any ponding because the high bond deck was propped. Now if we were to look at the drawings of the slab I think the props were at quarter points. I think that's correct isn't it? About 1.9 metres, something like that, or was it something like that? Perhaps if we had a look at the drawings but.

JUSTICE COOPER:
What drawing do you want us to look at?

DR HYLAND:
Let's see, it'd be the floor slab drawing? S15, that's 0284.16. Okay so the slab is propped at 1.775 centres. That's 1.975 at the end spans perhaps. It's 200 millimetres thick. This isn't a precast floor system where if you propped at that distance you'd expect there to be no ponding but this is actually just a tray, like a, we've got a portion of it over there and it will deflect, it will deflect.

1.775 metres is a reasonable span for this sort of deck and particularly with that level of concrete on it. So there would be a ponding effect that would need to be allowed for when you're doing it and that is normal with these composite metal deck slabs. You do have to allow for ponding in addition to what the manufacturer said is the ideal weight for the cross-sectional area so there would be some ponding that should be allowed in my view, and weight –

JUSTICE COOPER:

That increases the thickness?

DR HYLAND:

That'll increase the thickness. I think if I recall my interview with Bill Jones about pouring the slab he noted the flexibility of the system as he was putting it down and it is quite common when you're putting concrete down on these sort of slabs that you do get, they do feel quite flexible and require quite some skill to get a consistent thickness of concrete through the slab, and experienced designers understand that.

JUSTICE COOPER:

Thank you, I'll have my plans back now thanks. Is that accepted Mr Latham?

MR LATHAM:

Yeah, I mean there's a number of things and it depends on the construction methodology of the floor and whether it was screed to level or screed to thickness would have effects. I don't know the answer to that. Again I'd come back to there was a number provided by the product manufacturer in their literature and you know, was a designer reasonably expected to be able to rely on that number?

JUSTICE COOPER:

Which is what we understand of the views that have been exchanged on that subject. We'll move on to foundation stiffness and Mr Latham you were proposing that the original stiffness values be used which are smaller than those provided by Tonkin & Taylor, is that right?

MR LATHAM:

Yes that's correct. The numbers recommended by the original Geotech engineer.

JUSTICE COOPER:

Now I suppose for the purposes of a, well again this is this issue isn't it that I infer what your purpose is in carrying out this analysis? You're coming from the point of view of code compliance again are you?

MR LATHAM:

Yes, yes I am.

JUSTICE COOPER:

Dr Hyland?

DR HYLAND:

Foundation stiffness?

JUSTICE COOPER:

Yes.

DR HYLAND:

I think it's probably best for Mr Smith to answer that because he did some analysis on that.

MR SMITH:

I'm just getting the reference for a page to put on the screen.

JUSTICE COOPER:

Dr Bradley did you wish to say something in the meantime?

DR BRADLEY:

My comment Sir was, it's certainly, as I understand it, that the purpose of an ERSA analysis is for code compliance rather than for an understanding of how the structure actually behaves. How the Commission wishes to see it is of course up to them.

JUSTICE COOPER:

Well in fact is that an issue that we need to resolve or what view do you take of that?

MR SMITH:

I'm using it for the purposes of code compliance, or I'm referring to it for that purpose. I'll just point out one of this, this on the screen if we can. WIT.CARR.0002B.15. Can we zoom up on the, in the left-hand column it's got accidental eccentricity. If we can zoom up on the portion below that please, or including that and below, so it's just the bottom section? What we're looking at here is when we did this response spectra analysis we wanted we're looking at the purpose for code compliance but we're also testing the sensitivity of various, various components. So if we look at the top line as the most relevant for code compliance because that is the model that we had with the concrete walls only, so and we understood the original design basis modelled the structure with just the concrete shear walls. The other ones below that were including the potential effects of masonry and the masonry in the concrete frame but we, we went back to the top line because we understood that was the design basis. So that's probably the first thing to agree. But so we're calling it model 1A and if we're looking across that line we're looking at, actually I apologise, I need to go, include the next row up that starts, "Model foundations," so we've just missed one row out there. "Model foundations," at the left-hand side. So we're looking at the top row of the highlighted numbers and we're looking at the column headers for that. The code compliance ones that we looked at were based on an upper bound stiffness given by Tonkin & Taylor so that's what I've referred to as 1.36K at the top column, top row. However we also tested the response assuming that the foundations could be rigid, which was a point of contention in the original standard. But by comparing the two, so we look at the two first columns entitled "concentric" that's with the mass concentric. Comparing the, those two values we see there's quite a big change in the base shear which is the kilonewtons, the overall force at the base of the structure due to this effective lengthening or shortening period. So if we obviously go to the rigid foundation we're getting a lot more base shear than with a flexible foundation. So and then, so my purpose for this is that we, we did some studies to investigate the variance on this sort of behaviour and Mr Latham has taken it to another extreme of softer soil again. So we just look, have to look at his numbers for that to see that, but we did sort of form some view on how that effect would change things.

JUSTICE COOPER:

Well outside the context of the discussion which we're going to have in several weeks time about code compliance, it's unclear to me what we can achieve with this today. What do you think? Does anybody disagree with that, and you will have had an opportunity to see Mr Latham's approach on this issue given some flesh?

MR SMITH:

I'm happy with that.

JUSTICE COOPER:

Now in paragraph 9, Professor Carr, that's just reporting, yes. So scaling of results. The question raised there is whether scaling in fact had been used by Dr Hyland that was in accordance with the 1984 loading standard and the response given in the procedures that have been followed. Mr Smith providing a revised table, is that that one we've just been looking at in fact?

MR SMITH:

Yes it is, yes it is.

JUSTICE COOPER:

So here we are again. So the scaling is not exact but within a few percent which you considered reasonably good agreement. Do you want to comment on that Mr Latham?

MR LATHAM:

I don't really need to comment on that. I mean initially the numbers didn't really add up from my calculations. That correction was made. I accept that they're a few percent out because they still don't, you know, add up exactly but they are, they are a few percent out.

JUSTICE COOPER:

But the difference is not significant?

MR LATHAM:

I, I can't answer that with any certainty but they assure me anyway that they're done in accordance with the standard. I haven't, I haven't rechecked them all myself.

JUSTICE COOPER:

Well what, I don't think we can discuss that issue further either can we? So and then paragraph 11 is more reporting isn't it or comment?

PROFESSOR CARR:

Well sort of tidying up loose ends and the confirmation of the scale to the 1984 standard.

JUSTICE COOPER:

So that's an agreed position that's set out there. And then there's a disagreement about a further ERSA being needed. I suppose that's a matter for the Commission to resolve isn't it?

PROFESSOR CARR:
Well I think the additional analysis is possibly the one that Mr Latham is going to give us in his statement.

MR LATHAM:
I was going to say that the report submitted yesterday outlines that additional analysis using the inputs that I've put forward so that's been presented to the Commission.

JUSTICE COOPER:
So there can be a discussion about that in due course.

HEARING ADJOURNS: 12.37 PM

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