V delta problems. With regard to what you're asking about did anyone differentiate between plastic rotation at the base of the wall of the structure and try and separate or, or, or look at that more closely, honestly I don't think it was done normally. I think normally it would have been acceptable practice to take that K over S value and work to that and carry on. Not to say it was done in, without due consideration or regard because we all knew that for ductile structures anyway such as frames that you could get you know 3% drift and things would still be all right and it wouldn't collapse but for the other types of structures which I've referred to as shear wall protected gravity load systems I think to put that in perspective you have to go back in time somewhat and I don't want to protract the conversation but it is really relevant to see the, how people viewed things in the early '80s and also coming from the '70s when it was the same loading standard essentially 4203 it didn't really change in 1984 with regard to the ductility requirements. So in 1976 you came from no ductility at all or the just the limited .1g load and suddenly overnight had to step up to ductility requirements which wasn't, didn't happen and there was a sort of a graduated response from the profession so they started putting in ductility requirements in the seismic elements literally subject to the seismic forces and, and movements and, and so you had these gravity load elements like columns being left alone the way they were designed previously on the basis that the stiff shear wall structures weren't deflecting appreciably and, and we did have ways not that I was designing in the '70s but in the '80s of testing those deflections by lining them all up in a 2D analysis and looking at those deflections and saying, "Okay, they're small enough not to worry about".

COMMISSIONER FENWICK:
Right so basically what you're saying, I mean the K over SM factor works quite well for a framed structure because the frame deformations are in –

MR HENRY:
Yes.

COMMISSIONER FENWICK:

Basically in sheer. When you come to a wall of course with a fixed base the wall has zero rotation at the bottom but you're saying you think in the 1970s and '80s people would have ignored the inelastic component increasing the drifts at the lower level?

MR HENRY:
Yes I think they would not deliberately ignored it but perhaps not realised that –

COMMISSIONER FENWICK:
Not thought about it.

MR HENRY:
The implications.

COMMISSIONER FENWICK:
Yes. So one can say that's perhaps an error in the code?

MR HENRY:
I think yes it's a hole in the code yes that, that wasn't, there just wasn't enough knowledge about it. Actually if - I really want to get around but I do –

COMMISSIONER FENWICK:
I think probably we ought to move on.

MR HENRY:
Yeah.

COMMISSIONER FENWICK:
I think I've got your, got your position.

MR HENRY:
Yeah.

COMMISSIONER FENWICK:

Mr Bradley.

MR BRADLEY:

Obviously I can't comment on what was common practice back in the ‘80s but if you, an engineer was considering a capacity design approach to their building then they should be considering the post elastic sway profile that you get. I’ve actually prepared a sketch. This will explain what I mean and that's BUI.MAD249, that's it.

WITNESS REFERS TO SKETCH

JUSTICE COOPER:

Well we just have to say it so we’ve got a record of what we’re looking at.

MR BRADLEY:

Oh sorry.

JUSTICE COOPER:

Just …

MR BRADLEY:

MAD.249.0587.1. And essentially what that means for a wall structure what you get is you know you use a reduced set of design loads to get an initial displacement and that on the right top right-hand side you can see that there's a displacement and a component of that could be from foundation rotation and a component of that would be from the elastic deflection of the wall and then typically basically you say well this is when my wall yields and from that you form a plastic hinge at the base of the wall which you can see in the bottom right-hand corner and once that plastic hinge forms you end up getting essentially a rigid body rotation type thing happening where you get a linear sway profile and if you were to design that wall or any of the (inaudible 09:48:35) dependent on the displacements from that wall you should be considering that sway profile in addition to the initial elastic deformations in that wall as well.

COMMISSIONER FENWICK:
Right thank you so that's the 1992 code method. Dr Hyland.

DR HYLAND:
My thoughts are that we, we move from this K upon SM factor approach you know where we were looking at 55% -

COMMISSIONER FENWICK:
I'm sorry I'm not quite hearing what you're saying.

DR HYLAND:
I'm sorry, we, we moved from the K upon SM factor to, you know, where we used a K upon SM factor of what 2.75 or something in this, in this particular code. In later versions we then moved to using the full ultimate limit state drift when we did the drift checks.

COMMISSIONER FENWICK:
I don't think so.

DR HYLAND:
Well we, we moved to you check the, the drifts using for example, you factor up the design drift by the ultimate limit state so you go say in this context you'd use a K of five K over S of five.

COMMISSIONER FENWICK:
Right so that, that's I mean I don't quite agree with you because SV factors and so on but what you're saying is you would take the elastic value and just scale it is that right?

0950

DR HYLAND:

Yes, so you'd scale it. The difference in this code is the scaling factor is a lot less than what we currently use and what was changed when we went to the latest code.

COMMISSIONER FENWICK:

Not enormously. It's .55 to .7, it's not that big a difference.

DR HYLAND:

Well, I'm not sure if we're talking in, sorry we may be talking on different purposes but when we're looking at the application of '92 you effectively, we changed the definition. Went from an S to a mju definition for ductility factors. So you'd multiply the design drift by the, effectively by the mju factor. So in this case if you had an S of one, your designing for an S of one, you would now be designing, you would then check your drifts for five times that level of design drift.

COMMISSIONER FENWICK:

Okay, no that's what 1992? As far as '84 is concerned you would do an elastic or would you say that someone should recognise that an inelastic component there?

DR HYLAND:

Well the 2.2 is indicating there's some recognition of an elastic component but so you would apply that. The issue of accounting for, okay we're not talking about the deformations, but the problem we had with this '84 code is that there seems to be an issue there of definition of elastic behaviour and how you, and perhaps assumptions of working stress assumptions that are built into that check.

COMMISSIONER FENWICK:

I think that's a different problem. I think we're, I think I've got the answer from you is you would just multiply your elastic deflection by 2.2 divided by SM. Mr Latham?

MR LATHAM:
I'm only qualified to look at what the code requires which was just the straight scaling.

COMMISSIONER FENWICK:
So your approach would be, if you satisfy the code you don't need anything else?

MR LATHAM:
Well that's all I'm qualified to look at in terms of reading the code. I don't know what a designer would have done in the '80s.

JUSTICE COOPER ADDRESSES MR LATHAM – MICROPHONE

COMMISSIONER FENWICK:
Well I'm going perhaps on the false belief that a designer should have a certain minimum knowledge of the behaviour of the structures and should recognise where certain features that the intent behind it. I mean the K over SM worked well for frames, they should I would have thought with a minimum knowledge realised that K over SM does not work for walls but I accept what O'Leary says that, and it's the code, you can ignore common sense. I think that's what he's implying.

MR LATHAM:
I mean I don't know the answer to that. All I can say is what the, what the code required. Just the straight scaling approach.

JUSTICE COOPER:

Dr O'Leary, do you want to, you were in the answers you gave saying this was the compliance issue, this is what you needed to do to comply but Commissioner Fenwick really didn't want that question answered or not, that wasn't the only question he wanted answered. He was asking what was the prudent thing to do, what would've been prudent practice in 1986 notwithstanding that's what you may have been able to comply with the code using the K over SM factor?

DR O'LEARY:

That's a much broader question.

COMMISSIONER FENWICK:

Again, my phrase was, "What would an acceptably competent engineer?" The K over SM factor works well for a frame. You multiply up the elastic deflection to allow for inelastic deformation, it gives you a reasonable answer because the frame deforms due to shear. But when you come to a wall there's a different story.

DR O'LEARY:

I think the general understanding of that in the mid '80s was not as well developed as it is now. And the standards were looked upon in the mid '80s as good standards. The concrete standard particularly was looked upon as ground breaking standard internationally. I think the general view was that if we comply with the concrete standard as it is as best we can then we have an appropriately designed structure. Now I'm not saying everyone took that, what I would call normal practice of the profession attitude but that would be my view of the view of complying with the standards at the time.

COMMISSIONER FENWICK:

Can we have WIT.LATHAM.0003.13 please? If we can focus initially on the bottom equation on that page? That's the Branson equation which is used for determining deflection of beams. It's an empirical equation. The results, you measure the results, you know, sort of an R applying the load, it's usually about 90% of a plus or minus, about 25% of the predicted value. But of course to do that equation you've got to be completely consistent. Now we have a situation here where you're applying that to a beam to calculate the stiffness. I mean there's an option given in the commentary which says you can use .5 I gross but you've elected to use a different equation where of course the stiffness is different whether you're looking at a positive or negative moment. Normally of course if we're looking at a beam, you're looking at a beam which is simply supported and you've got a positive moment over the whole lot, or you've got a continuous beam where you've got negative moments at each end and positive in the middle because it's not, it wasn't derived for dynamic, it was derived for static cases. And then you've got a problem of what I do you use? Do you use the I at the ends or an average value and the values of effective Is can vary by a factor of four or more quite easily. In fact it's very hard to know exactly how you do it. I mean there is an alternative approach for this occasion where you can join a numerical integration to work out what the stiffness is. If you've chosen to do that, and of course if you do that you would have to comply with the code section. So what I want to know is what section did you use for calculating these stiffness values and what stiffness values did you use to calculate I effective for the beam as a whole?

MR LATHAM:
For the beam? It was just the gross area of the beam.

COMMISSIONER FENWICK:
Which is?

MR LATHAM:
Which is, oh, they varied in size but 550 deep and 400 wide in the internal lines and 960 wide in the external lines.

COMMISSIONER FENWICK:

If you were to look at the standard, I've not put down the number but it's clause 3.3.6.2 you would find that the flange width on the top would be 1.75 metres. Now that will have a big effect on your cracking moment, EMA moment.

MR LATHAM:
Yes.

COMMISSIONER FENWICK:
So to be consistent when using that equation you would have to use the defined flange width in tension for your cracking moment, and that is one quarter of the span. So it'd be seven metres divided by four. So your –

MR LATHAM:
Or it could be six times the slab thickness.

COMMISSIONER FENWICK:
Yes, at each side.

MR LATHAM:
Each side's smaller, yeah.

COMMISSIONER FENWICK:
So that's bigger. The span divided by four is the smallest one of those. Okay so your stiffness is likely, you've just used a rectangular section, it's likely to be on the, your stiffness is going to be on the low side of the value given by that as you're following the standard?

MR LATHAM:
Ah, yes it will.

COMMISSIONER FENWICK:
Would you agree?

MR LATHAM:

Yep.

COMMISSIONER FENWICK:

Now if we look now at the top equation. That top equation is appropriate for the internal span of a multi-span beam with regular seven metre spans?

MR LATHAM:

Yep.

COMMISSIONER FENWICK:

It's not quite what we have though is it? Because the spans we looked at BUI.MAD249.0284.16, one of the drawings?

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JUSTICE COOPER:

Yes. MAD249.0284.16.

COMMISSIONER FENWICK TO MR LATHAM:

Q. That's it and on the left-hand side, the plan on the left-hand side, if we can bring that up. Thank you. Now you can see the spans looking at the, the ones on line 2 for instance. We can go to any line but line 2, do you see the spans are approximately 4.5 metres, seven metres, seven metres, seven metres and 4.3 metres and that will influence the stiffness of the, effectiveness stiffness of the beams won't it at the junction?

A. Yes it will. Yep. I mean I accept that the, the spans are different and I mean that, that approach was done because it was, it could be easily done by hand. Obviously this exercise was taken with the –

Q. Yes.

A. – 1986 context so.

Q. Yep.

A. You know it was, it was deemed to be a method that was easily calculated by hand to account for these different effects.

Q. Yes, yeah, it's going to be (inaudible 10:01:46). Now you've taken the beams as prismatic along the length but in fact of course the joint zones are a good deal stiffer than the beams aren't they? Would you agree?
A. Yes.
Q. It's particularly important when you're looking at the column because the column you've gone centre to centre, you've gone 3.24 metres in height but it should be 3.24 metres minus .55 and as the stiffness is proportional to the cube of the length if you're allowing for a stiff column what's the effect, if you're allowing for stiff ends?
A. Well it's going to induce more load into the column.
Q. That's right in fact for the same shear in it the deflexion's about 40% less because it's a, a cube term comes in.
A. Mmm.
Q. So when you're calculating this critical value would it not be appropriate to allow for the end zone stiffening? We're checking here a critical issue (inaudible 10:02:47), surely a competent engineer would say, I've got to do this safely. I shall allow for end zone stiffening on this and so on and that will make quite a, and also allow for the effect, end zone stiffening on the beams as well.
A. Mmm.
Q. Do you agree that should have been done?
A. It could, it could be considered. I mean again the, the approach was, you know, one that was able to be easily done by hand.
Q. Yes but that approach is going to give you an un-conservative answer isn't it?
A. Yes.
Q. And that could be an un-conservative answer by almost 50% couldn't it?
A. I don't know how much difference it makes.
Q. Well it's the cube, the length, the stiffness.
A. Mmm.
Q. And you take off .55 and do it and you'll find you lose 40% and I put another 10% on, the same effect on your beam, reduced there. So if you accept that figure then what I'd say that what you have done may

have been what would have been done but I don’t think it would have been done by, I don’t think you would have done it now if you were doing it. I don’t think it would have been done by an engineer that would have been competent and I don’t think even if would have, I don’t even think that doing, (inaudible 10:04:21) taking the full length would really be consistent with the standard if you don’t have prismatic members. Would you agree?

A. Yeah I mean the, I mean the joint’s going to have some flexibility as well which –

Q. Yes.

A. – could, could be accounted for in some way.

Q. Yes.

A. And as to the measure of that and how that was considered in the ‘80s I'm not sure.

Q. Yes.

A. So I mean there’s other factors like that that will –

Q. Of course.

A. – reduce the impact of, of the factors that you're talking about as well.

Q. I agree but we’re looking at a, we’re doing a design.

A. Mmm.

Q. We want to be on the conservative side and you might well say, well we’ll double the stiffness at the joint zone or something like this which is, you know something which is done but without that step do you accept the sort of process you're following is not going to give a safe answer?

A. It’s not going to be the most conservative, no.

Q. Or necessarily a safe answer.

A. Well I don’t know about safe.

Q. Well if there’s a limiting deflection in there and you –

A. Mmm.

Q. – choose a method which over-estimates that ability to take that deflection is that a valid approach in your serious view?

A. I wouldn't do it like that now.

JUSTICE COOPER TO MR LATHAM:

Q. Why?

A. Why, because we have more knowledge and, and tools available to look at these factors a lot easier but if you're doing a quick hand calculation then, you know, it –

COMMISSIONER FENWICK TO MR LATHAM:

Q. I can assure you I did it with hand calculations. It took a few minutes at most, and I don't believe I have any particular skill with hand calculations. You may be used to only doing it on a computer but there are other people who can do it quite quickly by hand calculations but are you saying it would have been acceptable in 1986 to have done this unconservative check?

A. I don't know the answer.

JUSTICE COOPER:

What do you say Dr O'Leary?

DR O'LEARY:

Well what we did in our practice, it was use rigid end blocks. The size of them was always a question which we debated and usually took them as the gross size of the end block. It was a vexed question quite frankly about how, how rigid the rigid end block should be. But code compliance, I, I have been trawling my way through 3101 to find out if there's any real guidance and there isn't. Stiffness of T-beams is always, is another vexed question quite frankly. So that's really all I can say. I, I'll pass on that one.

COMMISSIONER FENWICK:

I agree but if you're checking something which is a critical flexibility for stability, do you not agree you should be conservative?

DR O'LEARY:

Well I think when you're checking something that's critical you tend to go to the conservative way but rigid end blocks is pretty drastic really because they have a flexibility which, especially post-elastic.

COMMISSIONER FENWICK:
Yes but we're not talking about post-elastic, we're talking about elastic.

DR O'LEARY:
Elastic then, rigid is probably as close as you're going to get them, maybe with some penetration maybe they should be 75% of the end block or something like that. I don't know. But you'd tend to go to the conservative view.

COMMISSIONER FENWICK:
Do you want to answer this?

DR DAVIDSON:
I would like to add to that. Sorry to walk away. I went and got the, the user guide for the copy of ETABS used in the 1980s and very standard was rigid end zones and they were rigid. So it was sort of common practice at the time, it would have been a rigid end zone and the idea that we use today where we may reduce that somewhat depending on the structural type like concrete we might go say half, with steel we may make it even smaller but, so I was just double checking. So the idea of a rigid end zone and it being used in common practice is there.

COMMISSIONER FENWICK:
Thank you.

MR SMITH:
The main difficulty I had with Mr Latham's assumptions about column stiffness were that I don't believe he used the same assumptions for calculating the wall stiffness so that the stiffness that was used in the analysis to derive the deflections, you know, the standard recommendations in the standard at the

time were I gross for columns, 0.60 gross for walls and 0.50 gross for beams. So Mr Latham I understand used the .60 I gross for walls to calculate the deflections but then used a lesser stiffness on the columns when he tried to assess the elasticity effects on those columns, of those thickness.

COMMISSIONER FENWICK:
Mr Latham would you like to respond to that?
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MR LATHAM:
I mean that, that's correct. I'm unsure of where those actual stiffnesses are required in the standard though.

MR SMITH:
Sorry, they were in the ERSA panel discussion included in Dr Carr's report. It was an Earthquake Society publication prior to the, a 1980 date on it.

COMMISSIONER FENWICK:
Let me just add to there, the commentary actually recommends for a moderately applied axial so that's not defined. A high to moderate axial load you should use I gross. The beam, it says, that's a member with no axial load .5 I gross. Therefore you are somewhere between I gross and .5 I gross. For anything which is less than the moderate axial load. That's in the commentary.

MR LATHAM:
Yes that's, that's one option. I mean that's not a requirement. It says the common assumption is .5 for the beams.

COMMISSIONER FENWICK:
Sure but if you don't –

MR LATHAM:

One for the columns in lieu of a more detailed analysis.

COMMISSIONER FENWICK:

Yes, if you don't do that you must have a more detailed analysis where you must allow for all the factors, tension stiffening, the degree of axial load and all these sorts of things which is a fairly hefty analysis when doing it by hand in 1986. It's easier now, but if you do that, I did it quickly on one of them, I found I got about .6 I gross on one of those columns but that's by the way. I mean that's, did you –

MR LATHAM:

Yeah I mean and in terms of that –

COMMISSIONER FENWICK:

Did you do that?

MR LATHAM:

In terms of that I mean again, going back to the I gross for the columns for heavily or moderately axial load of whatever the standard said, that's the kind of value that that equation 4.4 came out with for the lower two, I think it was two floors on the perimeter frames and three floors for the internal lines and then that reduced as you went up the height of the building as the axial load dropped. So, you know, that equation's consistent with what you've just asked.

COMMISSIONER FENWICK:

Well I'm not sure whether it is, but I'd like you to think about this. When you put an axial load on a column and you've got negative, you've got moments at each end, positive and negative moments at each end, so you're changing sine, you have flexural cracking limited to the two ends don't you?

MR LATHAM:

Yep.

COMMISSIONER FENWICK:
So the middle of the column is going to be all I gross.

MR LATHAM:
Correct.

COMMISSIONER FENWICK:
So when you're averaging which value you take, what do you take when you're averaging out what is the effective value you should use on this member which has got a varying moment?

MR LATHAM:
The standard recommended using the average of the maximum positive and negative.

COMMISSIONER FENWICK:
For beams, not for columns?

MR LATHAM:
Didn't specify which, what sort of member it was.

JUSTICE COOPER:
Have you finished what you wanted to say Mr Smith?

MR SMITH:
Yes I have.

JUSTICE COOPER:
Does anybody else want to contribute to this discussion?

MR HENRY:

Well I could make a brief comment about, with regard to column length and rigid end zone blocks. The sort of analysis you're talking about back in the 1980s. I think the rigid end zones were available on ETABS but if you were looking at a separate analysis on a 2D program I don't think they were incorporated into the sort of software that was around at that stage but in ETABS we wouldn't have put the secondary frames in, we would've only put the main shear wall elements or main frames, so that you wouldn't have got any output for those from the ETABS normally. Otherwise the model's just got too big and cumbersome, so that sort of calculation was done by hand. It would be done with the column length between beam and floor and it's a long time ago but I think .6 I G would've been a reasonable sort of value to use but I know in some cases using 4 I G when we definitely had a pin ended column. For example Union House I talked about those, that base isolated building we used 4 gross I G for checking the buckling of those columns. They were unrestrained by the ground. They had true pin ends.

COMMISSIONER FENWICK:

Let's just come back to this problem, where you are checking compliance deflection controlled by shear walls and your member. Now I can well believe you'd use .4 or lower values in Union House where you're wanting to make sure the deflection's not too high, but here we're doing something else, we're checking the deflection, elastic deflection can go for a certain distance. So while as you might use a low E I to calculate your drift is not to excessive, when you're calculating whether your elastic deformation go to a certain drift, it's a different type of question isn't it?

MR HENRY:

Yes, well, I'm just trying to put it in perspective of the sort of values we used. I can't think of any example where we would've used a greater stiffness in the joint for that sort of hand calculation and I can't think of any example where we would've used a rigid end block for a hand calculation. We would've used the distance between the bottom of the beam and the floors as you have indicated already for the length, and the I value, it's a long time ago. We might've tried a couple of values to get an idea of the sensitivity, perhaps be more mindful of the overall restraint on the building, how good that was, rather than just the individual element. In other words did the building have good restraint once the building became, yielding onsets to the walls.

DR DAVIDSON:
Excuse me, can I just state that I think Mr Henry is sort of confused, possibly confused people listening in that he's made the statement that he didn't use rigid end blocks in a hand calculation which I know he will have one thing in his mind, but then he said he took bottom of beam to floor lengths in the calculations for a column, which of course is in fact in itself assuming a rigid end block when he does that calculation.

MR HENRY:
Well, yeah, so not meaning to confuse but I'm differentiating between what the computer puts in an analysis as a rigid end block and us just assuming a length for the member.

DR HYLAND:
Well I just say that my recollection of practice pours is that we just did use the gross column and the, just the recommendations of the commentary with the Paulay paper, but there was that opportunity for someone to do the detailed analysis. I don't think it was done very often.

COMMISSIONER FENWICK:
So do a detailed assessment of what the stiffness would be. Yes it would be fairly complicated in those days, yeah. Just while I'm at point, if we're looking at line F. I don't know if we can have back the previous drawing, BUI.MAD249.0284.16? Line F is the bottom one parallel to the bottom of the page. That has some very big wide beams there, nearly a metre wide, 550 deep, very lightly reinforced. How did you assess the stiffness of those beams in your calculations?

MR LATHAM:

Just in a similar way to the, the internal lines. The section I value was taken for the rectangular section of the beam and then the effective section was worked out, or the effective stiffness was worked out using that equation 4.4 again.

COMMISSIONER FENWICK:

So you would've had a, it would've been an I gross for the beam?

MR LATHAM:

Ah, it, no it was less than I gross. It was around I think, by the time you averaged it it was around about the .6 sort of order.

COMMISSIONER FENWICK:

I don't quite understand that because the moment from the columns is about 200 kilonewton metres and if you put that column moment in it gives you about 100 kilonewton metres into each beam, and the cracking moment's about a 130 kilonewton metres. So I couldn't quite see how you could crack it?

MR LATHAM:

Well I, I mean yeah, I'm not sure. I'd have to review the numbers, but I remember that, I think at the mid span, you know, the section was almost at its gross capacity and at the negative moment by the columns it was a reduced value, so by the time you averaged it it was less than the I gross.

COMMISSIONER FENWICK:

Would you agree when you put the column moments in there's a point of inflection at the mid span so there's no additional moment acting at where the gravity load acts?

MR LATHAM:

That's correct. At the mid span it doesn't change under the seismic conditions.

COMMISSIONER FENWICK:

It's really the end zones which effect the stiffness of that member isn't it?

MR LATHAM:

Yep that's right, yep.

COMMISSIONER FENWICK:

Which would be I gross?

MR LATHAM:

Well that's not what I worked out but I'd have to review that.

1020

COMMISSIONER FENWICK:

And of course you were doing it by the, the standard you'd also had the tension flange as well on one side (inaudible 10:20:05).

MR LATHAM:

You could, you could consider it yeah.

COMMISSIONER FENWICK:

So then could it be then to your calculation do you think ignoring the tension value being rather again unconservative for the deformation of those columns on line F?

MR LATHAM:

They're less conservative than what you could assume yes.

COMMISSIONER FENWICK:

Less conservative, now what do you mean by less conservative. You mean they're on the dangerous side or on the safe side? Is less conservative I presume you mean is less of a safety in it, there's unconversative means you have overestimated the factual deformation capacity of the column so can you just clarify what, what you mean for me please?

MR LATHAM:

Well I mean that it's been assessed with a set of assumptions that are, I mean I don't consider them to be wrong, they're not necessarily the way that you might do it but, so in that respect then yes they're, they could be considered less conservative.

JUSTICE COOPER:

Are they conservative at all, let's start with that question?

MR LATHAM:

Are they conservative at all?

JUSTICE COOPER:

Yes.

MR LATHAM:

I, I don't know the answer. There's, there's probably not a whole lot of conservatism in there.

COMMISSIONER FENWICK:

Let me help you, you've ignored the, the stiffening bit of a block which is a metre wide with a flange on it?

MR LATHAM:

Mhm.

COMMISSIONER FENWICK:

You've ignored the stiffening down through that depth, you've got the same at the top so you've got a very solid beam column joint do you agree? You've ignored that stiffening effect. You have taken the stiffness of the beam and deflecting the flange out there I, perhaps my calculations there I don't know when I did it I got a bending moment there which was well in excess of what you could go in so how can it be that your value is on the safe side when you've ignored these deformations. Is this what you would do in practice?

MR LATHAM:
Not today no but again it was, it was –

COMMISSIONER FENWICK:
Why do you –

MR LATHAM:
- attempted with a 1986 approach with a you know simplistic view that could be calculated easily by hand.

COMMISSIONER FENWICK:
What makes you think that in 1986 this sort of problem would not have been, engineers would not have been aware of this?

MR LATHAM:
I consider it was, it was just the combination of the information I had and, and the assessments that were made.

COMMISSIONER FENWICK:
I think you're down-rating engineers in 1986. They actually, because they did things by hand I think they probably understood what they were doing and I'm quite sure they would have been aware of the, of, of the stiffening effect at the end and what this could have, don't you, I mean what basis have you got for saying that engineers 30 years ago had less knowledge of structural mechanics than what we have now?

MR LATHAM:

I'm not trying to say that at all, no.

JUSTICE COOPER:
From my point of view as a lay person on these issues you seem to be saying that you've chosen an approach which would have been simple to do by hand. Is that, is that then the guiding principle that you followed?

MR LATHAM:
Yes it is.

JUSTICE COOPER:
But wouldn't it, wouldn't an engineer have been guided by what was necessary to produce a robust design rather than simply looking at ease of calculation?

MR LATHAM:
Well I mean that's, that's an underlying requirement but you know assumptions are made throughout your analysis, they, they have to be made.

JUSTICE COOPER:
What do others think of this issue whether Mr Latham's approach would have been considered likely to result in conservative safe outcomes?

DR DAVIDSON:
Can I?

JUSTICE COOPER:
Yes.

DR DAVIDSON:
Two points really. Firstly I think he has done a non-conservative, a less safe, but the other thing is he's – I would just like to perhaps add to your statement in that he's taken a simple approach, the, the approach that he's done could

be equally simple by making some of the adjustments in the calculations that Commissioner Fenwick has done. So it's no more difficult to include the, the additional effects of the floor into the stiffness of the beam and nor is it any more difficult to take a shorter length in the column to take, allow for the rigid zones in the zone so as far as the hand calculation going, goes it is no more difficult to add those things than it is not to include them.

JUSTICE COOPER:
What do you say Dr O'Leary?

DR O'LEARY:
I'd just to make a comment about the stiff or flexible column, when the hand analysis is done you'll see that a stiffer column gives you a great fixed end moment for a given drift but the converse is of course the stiffer the column when you redistribute that moment to the beams the –

COMMISSIONER FENWICK:
Mr O'Leary, Dr O'Leary I beg your pardon, if you're going to redistribute you've got to have inelastic, inelastic deformation and you're not allowed, we're meant to say that this elastic response, you can only get redistribution if you have inelastic deformation.

DR O'LEARY:
No, no I mean in the – the way I did it in fact was in a moment redistribution I mean that, that redistribution. If you redistribute or distribute that fixed end moment in your, in the columns into the beams and columns you actually reduce the moment in the column, the stiffer the column is relative to the beam so there is a little bit of opposite effects there which helps you.

COMMISSIONER FENWICK:
I don't think anyone's suggesting you assume the column is rigid at the end. We're all assuming it's framing and the beams which deform.

DR O'LEARY:
Yeah I know that.

COMMISSIONER FENWICK:
So that, that, that deformation is allowed for.

DR O'LEARY:
No what I'm, what I'm saying is that if you, the fixed end moment for the moment distribution is higher, the more rigid the column is but when you redistribute that column to the, that moment to the beams in fact the stiffer the column the more redistribution away from the columns into the beam. So there is some compensating effect there.

COMMISSIONER FENWICK:
But we're, but we're allowing for that in the analysis?

DR O'LEARY:
Yes but I, I think the effect of the rigid end block mightn't be, it's not all one way, that's all I'm saying.

JUSTICE COOPER:
It would be helpful for me – if you told me whether you agreed or disagreed with Dr Davidson's comments?

DR O'LEARY CONFERS WITH COLLEAGUE

DR O'LEARY:
Yes please refresh me.

DR DAVIDSON:
Basically I made the point that the simplicity of the calculation was not increased in any, decreased in any way by adding the effects of rigid ends,

end blocks or increasing the I gross if you like of the beam by taking into account flange effects from the floors.

DR O'LEARY:
No that would be right and the other thing too is that hand analysis was, tended to be conservative by its very nature because –

JUSTICE COOPER:
Advisedly so?

DR O'LEARY:
Yes. Because it was less accurate. Well not less accurate, it was more basic shall we say so it would be tend to be more conservative.

JUSTICE COOPER:
Thank you. Mr Smith.

MR SMITH:
Well I would probably go one step further and say that the hand calculations that Mr Latham's done to work out column stiffness would actually be more complex than, than simply taking IG which is the standard recommendation in the standard so, and it is on the unconservative side.

DR O'LEARY:
Yeah, I, simply I agree with that and Dr Davidson's comments as well.

MR HENRY:
I agree with Dr Davidson and Mr Smith.

MR BRADLEY:
Yeah I agree with Dr Davidson and I think if you're going to do it you wouldn't be doing a simple analysis by hand you would have to do a moment curvature analysis and take quite a lot of care with it.

JUSTICE COOPER:

Thank you. Do you want to comment further?

MR LATHAM:

Oh I mean again it's just the, the approach has been an alternative to what's already been put forward and you know it's been a –

COMMISSIONER FENWICK:

Thank you Mr Latham but what you're saying it's an alternative and what we're, what I think everyone else is saying it's an alternative on the unconversative side but do you, do you not accept that?

MR LATHAM:

I accept it's less conservative than some of the other methods that have been put forward yes, um, whether it's permissible or not well then that's a different question.

JUSTICE COOPER:

I still don't really understand why you use the word conservative, you say something's less conservative, the starting point is what's conservative but at some stage you get to being a liberal.

1030

MR LATHAM:

That is a good question I mean really it is what is defined in the standards that is in effect your minimum and anything above that would increase your conservatism.

COMMISSIONER CARTER:

In simple terms you mean less safe?

MR LATHAM:

That could be one way of putting it yes.

COMMISSIONER FENWICK:
Or more dangerous?

MR LATHAM:
Yeah but if I mean if it still meets the standard then I mean...

COMMISSIONER FENWICK:
But does it meet the standard? If you are making these un-conservative assumptions how can you be sure that you are meeting the standard? It doesn't quite make sense to me?

MR LATHAM:
Well again if you follow through you know, what it outlines can be done, then this approach is you know, in line with that. There are other ways of looking at it.

COMMISSIONER FENWICK:
It is only in line with it if you are prepared to make un-conservative assumptions, isn't it? If you are neglecting end stiffness and other things like this, you are neglecting sectioned areas which stiffen the section up?

MR LATHAM:
Yep.

COMMISSIONER FENWICK:
Which gives you a bigger deflection, which gives you perhaps a false feeling this is a safe design, well if you did it more carefully, more accurately you would find out that your deflection capacity was smaller than you expected. Wouldn't you?

MR LATHAM:
You could do it you had a different set of assumptions, yeah.

COMMISSIONER FENWICK:

And you don't accept, you would use these assumptions now because you feel they are valid?

MR LATHAM:

I wouldn’t use them now, no because it is a –

COMMISSIONER FENWICK:

Why would someone use them in 1986 and feel they are valid?

MR LATHAM:

Well there was a different standard, again –

COMMISSIONER FENWICK:

Who had a different standard?

MR LATHAM:

Oh, I am saying the standard NZS3101 was a different standard. Again it is just an alternative approach to looking at it.

MR PALMER:

Sir may I assist. The purpose of Mr Latham going on the ERSA panel and preparing this work was really because of his inexperience relative to everybody else. He has approached this and I think my opening statement perhaps I didn't make this as clear as it could have been. As an engineer without the depth of experience which everyone else might have and approach this with, certainly in terms of the DBH ERSA that was undertaken which was the only other ERSA that was done at the time and so he’s putting himself in 1986 as a relatively inexperienced engineer interpreting the code as best he might at that time and it’s an exercise which gives another framework within which to look at the problems. That is where Mr Latham is coming from. I don't think he’s ever said that he would do –

JUSTICE COOPER:

So just let me make – I would like to make a note of this Mr Palmer. Mr Latham has attempted to state what a relatively inexperienced engineer would have done is that what you said?

MR PALMER:

He is not interpreting the code against a background of experience that all the other engineers in the room might have because of their length in the industry.

JUSTICE COOPER:

I know that. I have grasped that, it is quite simple but it is the next part of what you said to me that I want to understand. I thought you said he was attempting to state what a relatively inexperienced engineer would have done in 1986. Is that what you are proposing?

MR PALMER:

What an engineer of limited experience might have or how an engineer of limited experience might have approached the code using that as the compliance requirement against, in which we was working. Now if I –

JUSTICE COOPER:

Just pause while I write it down. Mr Latham has attempted to state what an engineer of limited experience might, how an engineer of limited experience might have approached the code in 1986 right?

MR PALMER:

And perhaps if Mr Latham might correct me if I am wrong in that Sir.

MR LATHAM:

That sounds like the basis that was, the approach was undertaken, yep.

JUSTICE COOPER:

Thank you.

COMMISSIONER FENWICK:

There are a number of other points that I have got down here. I think we have probably in one way or another covered several of them but you might like to comment. The first one I have noted here was selecting the critical drift and we have had a variety of answers where you can just take the value from the ERSA analysis at the bottom scale by K over SM, that is fine and others say well you should allow for the inelastic deformation you get and so on, so I don't know if anyone wants to add anymore on that particular aspect. I think we have probably covered it but there is a chance if someone wants to come back on that. Seems to be a split feeling there and perhaps we could say well was this an error in the code in 1986? Can we have a definite yes or no answer to that. The fact that you could scale it apparently, some people believe you could scale the deflection just by multiplying K over SM and forget the fact that you have plastic hinge rotation at the bottom of the walls going over within the – do we have a yes or no, was that a, from you, was that an error in the 1984 standard or not?

DR O'LEARY:

In hindsight it was an error in 1986 standard.

COMMISSIONER FENWICK:

Right, just keep on – quick thank you.

DR DAVIDSON:

Well I don't want to be too quick if I can. I think, I am really getting quite frustrated here because it seems to come from the legal profession that this is a legal document and maybe it is but it has to be interpreted by a competent engineer and I think that is coming out. Now as you pointed out earlier on that the engineers in 1986 were equally competent as they are today. They may not have had all the knowledge but Professor Paulay and Park were publishing and in '86 I think the knowledge of the behaviour of shear walls et cetera wouldn't be far short of where it is today. So I think if we are now looking at this standard and I suspect there are people in this country who look at today's standards and try and cherry pick their way around looking at clauses and how they might interpret them the way Mr Latham has tried to I suspect interpret them in a very legalistic opportunistic way and it cannot be done that way and I think we should start just accepting that and I see Commissioner Fenwick has been pushing this idea for some time that in fact you do not read any standard blindly like – the only thing you understand is English. You must have to understand engineering and the laws of nature and until that code is interpreted with that background you are just going to get silly answers no matter whatever you do. I didn't answer the question.

JUSTICE COOPER:

If I may be legalistic for a moment Dr Davidson but the question was whether this discussion is identified an error in the 1984 code and you seem to be saying if I've got the message that only a legalistic person of small understanding would find some of these difficulties that are being identified?

DR DAVIDSON:

Yes, that is correct and I – and while we are stating it may have been an error in the code it is I guess somewhat implying that the next code, not the 2012 code but the 2025 code whatever, will be perfect and it will not be. It will always require a good interpretation by knowledgeable people and people who have been trained by good people and so if we want to cherry pick our way through and find errors in this code we will be going forever and when we do it will still fall short. So I sort of feel that I am not too keen to support the idea that this was an error in the code. I think it was a, it allowed, maybe ill informed people to interpret it incorrectly that is all.

COMMISSIONER FENWICK:

Thank you for that, I mean, I will start off by saying I was interested in the code and what a competent, acceptably competent engineer would do and I think your reply fits quite in those lines. In fact one of the stories to students was you have to know when the code is wrong. That is by the way, I mean that's answering away but..

JUSTICE COOPER:
But my legal training if I may bring that up again at this point, suggests to me that it might be unfair on Dr O'Leary who did restrict himself to a one word reply to have allowed Dr Davidson such a long answer.
1040

DR O'LEARY:
I think all I really want to say is that to call it an error in a code is seeing things in a very black and white way. We know that the current loading standard for instance, and I was on the committee God help me, we still haven't got torsion right. In fact we've moved very little from 1986 and we're going to have to sort it out.

COMMISSIONER FENWICK:
From 1992?

DR O'LEARY:
Yeah there's not that much difference but we've got to sort it out. It is really, it's a festering sore.

JUSTICE COOPER:
So Mr Smith?

MR SMITH:
Yes I just think the clause we've been talking about is the scaling up the elastic deflections. It is an omission I see in the code that it could have easily said that such and such were frame structures and such and such were walls. It would've been nice to have that in the code.

MR HENRY:

I agree that definitely in retrospect wasn't clear and I think there was a wide range of users and skills and experience, you could've, could've cherry picked it so to speak with the terms being used. And I know there are those who did think deeper as well. So yes there is plenty there for things to have been not fully interpreted correctly.

MR BRADLEY:

I agree with the sentiments of Dr Davidson. There is a range of interpretations and it comes down to the skill and ability of the engineer.

DR HYLAND:

Yes I totally agree with Dr Davidson and I think the other thing is, the skill and the code development is to get provisions that are easy to apply if possible but that do envelope the difficulty of the things but perhaps on a more conservative side.

COMMISSIONER FENWICK:

Do you want to add that comment now or not?

MR LATHAM:

I'll say that the only thing I'll add is the current standard has a requirement to look at that very factor that you've been looking at with the rotation and the side sway mechanism so I guess that's recognition that there's, you know, improvements on the previous code.

COMMISSIONER FENWICK:

We haven't talked about calculating the shear forces in the joint zones at all. Whether they should be, how they should be treated or modelling the beam column joints. I don't know if this would like to start off here Mr Latham?

MR LATHAM:

In terms of how to model it?

COMMISSIONER FENWICK:

Yes. Well the design of the joint zone. What's critical? Do you need stirrups in there or ties in there or can you assume it's not required, no reinforcement, no joint zone reinforcement's required?

MR LATHAM:

Well my interpretation of the code was that there was, there needed to be more reinforcement in that joint.

COMMISSIONER FENWICK:

So in your terms the design was non-compliant with all the joint zones?

MR LATHAM:

All the joints, yes.

COMMISSIONER FENWICK:

Does anyone want to add to that or comment at all? Everyone accepts the joint zone should've been, had joint zone ties designed into them above the, that are missing, sorry, above the missing six millimetre spiral?

DR O'LEARY:

I just make a comment there if I may? The standard was confusing the way it directed you but the joint zones in my view obviously didn't comply. They didn't remotely comply quite frankly.

COMMISSIONER FENWICK:

No other comments? We've talked I think, we've got columns remaining elastic, I think we've probably talked enough about that one. The foundation rotation effects are the last question I've got down here. I think we've probably covered that one. Did anyone want to add any comments about foundation rotation and taking the rotation due to foundation deformation out?

1045

I think we can probably accept can't we that in the mid-1980s it was common practice to ignore vertical deformation of foundations. Anyone disagree with that comment? So anyone want to add anything about foundation rotations in here we haven't covered?

DR HYLAND:
I made some comment about it yesterday.

COMMISSIONER FENWICK:
Okay all right. So –

MR SMITH:
I think I would, I would like clarification. I don't know whether we got to the point did we that, I certainly felt the calculations by Mr Latham to deduct the space (inaudible 10:45:52) due to rotation was not a valid one. I don't know whether we'd reached agreement on that or not but that was my thought that, because I just didn't see that was consistent, through either you assumed there was some rotation and you carried that through or, or you don't assume any rotation at the start. So I don't know whether, have we reached agreement on that as a group?

COMMISSIONER FENWICK:
Well that's what I'm trying to find out.

JUSTICE COOPER:
Well have we, who, who can enter this discussion? Yes, Mr Bradley.

MR BRADLEY:
In my opinion you cannot take off foundation rotations from one foundation system and then take those rotations and apply them to an independent foundation system because it violates displacement compatibility between the two elements.

JUSTICE COOPER:
Does anyone disagree with that view?

MR LATHAM:
Can I just add that the basis of doing that was that the code specifically said that computer deformation shall be neglected, shall be calculated neglecting the foundation rotation. So that was the basis of that decision.

COMMISSIONER FENWICK:
And there are of course two interpretations of that as I see it. One is what you have said. The other is if you can't depend on that rotation occurring, and this is particularly true I believe for those who've studied geomechanics, that the foundation conditions may change when you put a few hundred tonnes on the foundations, the, the flexibility of that soil can change and therefore one must assume that the deformation that you have there is now going to be forced into the structure. So there maybe, I don't know whether other people agree with that other possible definition of why that was there. I agree it's an odd statement to have but anyone, I would be interested in other people's comments on this.

MR SMITH:
If we're looking at a, just a literal interpretation of that clause in the standard the way I interpreted it was that you analyse the building neglecting the foundation rotations, that's the way I interpret that clause.

COMMISSIONER FENWICK:
Right, yes.

JUSTICE COOPER:
So what's the other view of its, mean …

DR HYLAND:

Well the other view is that you, you look at the, you say, well that's the analysis of the primary frame but when you're coming to looking at the interaction with your secondary frame that, which is not on a rigid basis on the same thing it's going to be affected by the soil rotations that are occurring on the primary frame foundations. So you do need to account for the total displacements that are occurring on that shear wall that's going to be affecting the drifts on the, on the columns and that to me is just an obvious thing that you would have to think about. You couldn't say, I'm just going to ignore it because there's a, perhaps an interpretation of a clause there.

COMMISSIONER FENWICK:
It's hard to imagine someone with basic structural mechanics knowledge actually would you agree applying that, take out the rotations, the deformation due to rotations. I mean it goes against all basic structural knowledge doesn't it?

DR HYLAND:
Yeah.

COMMISSIONER FENWICK:
Does anyone disagree with that? You still don't maintain that. You're saying because it's in the standard you can do it. Is that right?

MR LATHAM:
I don't disagree with, you know, what's actually happening in the structure at all. I guess we're in this position where, you know, your structural engineering knowledge would tell you one thing and the code is saying another and so again we've gone down that other approach to see what it means.

COMMISSIONER FENWICK:
I guess that the standard was written for someone with several years of practical knowledge, design of, behaviour of structures. I think you have to interpret clauses with that knowledge. I don't think you can, sorry this is my

belief I think other people are expressing but, you know I, anyone like to comment or not?

MR BRADLEY:

I don't believe that someone with say two, three years' experience in designing buildings could interpret that clause and, in the way in which Mr Latham has applied it. I don't believe that even if you did interpret the clause to say that you could take off foundation rotations you wouldn't take off foundation rotations on one part of a structure and apply it to an independent part of the structure. I just don't, I don't think that would be possible.

COMMISSIONER FENWICK:

That's the set of questions I've got if we want to …

JUSTICE COOPER:

So Commissioner Carter anything to add from you?

COMMISSIONER CARTER:

No I think we've covered what we're looking for.

JUSTICE COOPER:

This issue that Dr Davidson has spoken of is obviously a matter of concern. The reason there may be some tension between the engineering and the legal profession is that the, compliance with the standards is the basis upon which a person wanting to construct a building can assert a right to a building permit or consent as it is these days and it seems to me for practical reasons there has to be a degree of precision about the circumstances in which a person is entitled to get a building consent. A council or a building consent authority needs to know the circumstances in which it is obliged to issue a building consent and issues of liability, when and if they arise because of the inadequate performance of a structure, are going to be largely founded on whether or not a structure complied with the relevant rules applicable at the time. So there needs to be a degree of assurance that the codes are as precise as they reasonably can be. Now I understand what is being said here, that one can find words which might be applied in a certain way but that experienced and well-informed engineers would not think that that particular approach was a reasonable one to take or one that would lead to a robust and sound structure but so long as there are possible differences of interpretation this tension that has been identified is going to continue to exist and I think it is likely that codes will never be written in the way that one might write a statute or some other legal instrument but by the same token I do think that it's going to be necessary to continue to strive for a degree of precision and recognising that ultimately it's not possible to achieve complete precision an answer may have to be found in the area of saying that certain kinds of buildings are too important or the failure of a certain class of buildings is likely to be too serious in terms of injury to persons, and even worse consequences,

1055

and that those who are designing them should be a particular class of persons with extra qualifications of some kind and also that there needs to be a worked out system of peer review and checking.

Now that little speech, not intended to do anything other than show I hope that we are aware of these issues, and I hope that that's the sort of issue that can be discussed when we have our hearings in about a month's time I think, on the way the regulatory system works. But it's, it's a question of finding the right balance and I think in the end acknowledging that what Dr Davidson says is likely to be correct, absolute precision in terms of words in a code is, will never be achievable and may not even be worth achieving if the consequence of it is going to be a system in which competent people are dissuaded from using their good judgement. I don't know if anybody would like to add to that?

COMMISSIONER FENWICK:

Well I just want to make one comment, say that Mr Latham's put in a lot of work, raised a lot of issues which I believe are of value to have discussed before the Commission, so I'd just like to say thank you for your efforts on that. I think that we've gained quite a lot from the discussion of this.

MR LATHAM:

I could say that was the objective so…

JUSTICE COOPER:

Mr Mills, hiding there in the back of the room.

MR MILLS:

They've spotted all this engineering talk going on I can tell you.

JUSTICE COOPER:

What happens next Mr Mills?

MR MILLS:

Well if this part is finished I think that there, I think for one I think Mr Elliott might have, he's shaking his head, I think you might just want to inquire when you come back whether any of the lawyers do want to ask any questions arising out of this. The answer may be no –

JUSTICE COOPER:

Well I'll do that now because otherwise we'd be adjourning early. Does anybody, any learned counsel wish to ask questions of the assembled engineers? Mr Reid?

MR REID:

No Sir.

MR PALMER:

Only a point in clarification involving Mr Henry which I think my friend will probably attend to.

MR MILLS:

Well if there's nothing more then I will take care of that. There's a letter and I'll hand up copies to the Commissioners. There's a letter that's come in from

Mr McCahon about this question of the advice that he gave to Alan Reay in 1986 about soil qualities. The thrust of it, as I understand it, is that there's some concern that in the way in which Mr Henry has recounted the conversation the two of them had which has then been put into Mr Henry's brief that there's been some level of misunderstanding between the two of them as to what exactly was being said and what was not being said. I think the simple answer to it, unless Mr Henry doesn't agree with the letter and what it's now said, is simply to make copies of that letter available to the Commissioners. It's clear to me what it's saying so the only thing is to ask Mr Henry whether he has any disagreement with what it says I think.

JUSTICE COOPER:

Well you could speak to Mr Henry about that presumably and, or has he seen it?

MR MILLS:

He has seen it and he should be in a position to answer that now so if he is it would be better to do it here I think.

MR HENRY:

Yes I can comment. The letter's fine. It reads what the understanding of my conversation was. I've probably summarised it too much in my evidence, it's fine.

JUSTICE COOPER:

But you're content with what Mr McCahon says?

MR HENRY:

Yes.

JUSTICE COOPER:

Mr Elliott?

MR ELLIOTT:

Your Honour I think there is just a brief matter in fairness to Mr Latham and Dr Reay which I should put to Mr Latham if I may?

JUSTICE COOPER:

Yes.

MR ELLIOTT:

It won't take long. It doesn't need to be in panel forum.

JUSTICE COOPER:

Is this the only issue that we're going to be dealing with?

MR ELLIOTT:

Yes Your Honour.

JUSTICE COOPER:

So we could adjourn, enable others to leave if they wish to and then we'll come back in, in five minutes to deal with that. Mr Palmer you don't have anything further do you?

MR PALMER:

No I don't.

JUSTICE COOPER:

I've been remiss. I want to thank you all for participating in this discussion. I know it's been a real commitment in terms of time and convenience. It's been a very valuable exercise from our point of view so thank you all.

PANEL RELEASED

MR MILLS:

And Sir before you go, one other thing can I just raise, I've just been reminding myself that various of these participants have prepared written briefs of course which were taken as read. I don't know whether you consider it necessary for a formal production of those. They've gone into the system, they have de facto been accepted as their evidence but that is the position that they've put in briefs which have not been formally produced.

JUSTICE COOPER:
I'm very keen for there to be a record of all briefs that we're treating in that way and I'm relying on Commission staff to do that, but otherwise briefs that have been presented will be taken into evidence and read. Everything that's been filed.

MR MILLS:
That's sufficient. We've certainly tracked what being done and we know that.

HEARING ADJOURNS: 11.02 AM

HEARING RESUMES: 11.09 AM

JUSTICE COOPER:
Mr Elliott.

CROSS-EXAMINATION: MR ELLIOTT

Q. Thank you Your Honour. Mr Latham I just have a few questions arising from some of the answers that you gave to the Royal Commissioners?
A. Yes.
Q. Now you'll have your briefs of evidence available but I'm just going to quote a couple of short sections?
A. Yep.

WITNESS REFERRED TO BRIEFS OF EVIDENCE

Q. The, in the first brief paragraph 22(a), 22(b) I'm sorry, you express the opinion I take it that the CTV building complies with the drift limits in the code?

A. Yes.

Q. In the accompanying report you say that the scope of the report, sorry the purpose was considering whether the design was consistent with design standards and codes applicable at the time of design?

A. Yes.

Q. In your second brief you give an opinion about the design of columns being consistent with the provisions of codes?

A. Yes.

Q. And in your second report you say the purpose was to consider whether the design was consistent with design standards and codes?

A. Yes.

Q. However in response to some questions from the Commissioners you used some words "an alternative approach" and also "going down another approach to see what it means"?

A. Mhm.

Q. Now by alternative approach I take it you would mean others have put one particular approach and I am now putting something other than what they've put, is that right?

A. That's correct. Yep.

Q. And in this case we had Dr Hyland and Mr Smith together with the DBH Panel, Professor Priestley, Mr Jury, Mr Thornton, Professor Pampanin and Mr O'Leary in relation to line F expressing opinions about non-compliance of columns?

A. Yes.

Q. But you have put an alternative approach to that have you arriving at the position of saying they were compliant?

A. It's an alternative set of assumptions yes.

Q. So just to be clear on your evidence to the Commission you're not saying, "In my opinion the CTV building was compliant". What you are saying, "If one uses this alternative approach which I outline one could arrive at a point where they could assert the building was compliant"?

A. Partially. I haven't looked at every aspect of the building, I've only looked at the columns essentially and -

Q. That – sorry.

A. Yep so I mean I haven't yeah, I haven't, I haven't looked at every aspect of the building so I'm not in a position to say that the building was compliant. I've looked at the design of the columns and the beams with respect to earthquake loadings and from my set of assumptions and analysis then we, the, the design of those are consistent with, with the required standard.

Q. All right well I just put that distinction to you again so we're just talking here about the columns and the beam column connections?

A. Yes.

Q. So you're not expressing an opinion about whether they were compliant, what you are doing is you're saying if one adopts this alternative approach then one could arrive at a point where they could assert that they are compliant, you are doing the latter?

A. Yes.

Q. Mr Palmer made some comments to His Honour which you adopted, they related to your evidence attaching to what an inexperienced engineer would have done in 1986, do you recall that?

JUSTICE COOPER:

Limited experience.

CROSS-EXAMINATION CONTINUES: MR ELLIOTT

Q. Limited I'm sorry. I apologise Your Honour. So the purpose of that was to model your evidence was it on what Mr Harding would have done?

A. No, my, my evidence in an analysis is independent of what Mr Harding did. The purpose of my evidence was more we were, we had one model from, that was done out of the DBH work and we wanted to look at the sensitivity of some of those assumptions and you know get a better idea of you know changing some of those assumptions what that actually means in terms of the design of the building and with respect to the required standards.

Q. I see, so your evidence as you've said is based on what an engineer with limited experience would have done in 1986?

JUSTICE COOPER:

Sorry, that's right yes, well I had understood him to be saying "might have done".

MR ELLIOTT:

Might have done.

JUSTICE COOPER:

Not would have done.

MR ELLIOTT:

All right might have done.

JUSTICE COOPER:

Q. Is that right Mr Latham?

A. Yes that seems to be right.

CROSS-EXAMINATION CONTINUES: MR ELLIOTT

Q. Well you've in fact calculated V delta haven't you?

A. Yes I have.

Q. You've seen Mr Harding's calculations?

A. Yes.

Q. He didn't calculate V delta did he?

A. He's, he's partially calculated it. It's you know, it's unclear exactly what he's done if you look at the calculations alone.

Q. So it's correct to say that Mr Harding appears to have done even less than what an engineer with limited experience might have done?

A. Again, I, I don't know. My work isn't related to what Mr Harding did or did not do.

Q. And I just want to ask you one or two questions about a page of Mr Harding's calculations. BUI.MAD249.0273.44.

WITNESS REFERRED TO CALCULATIONS

Q. This is G41A of his calculations? You've seen this before?

A. Yes I have seen this page.

Q. And can you confirm this is the one and only page of the calculations relating to the calculation of transverse reinforcement of columns?

A. I couldn't confirm that. I'd have to review the calculations.

Q. And is it correct to say that in the first section of that document what Mr Harding does is to calculate the transverse reinforcement relating to the, applying the non-seismic provisions of NZS3101?

A. Yes.

Q. And he arrives at R6 at 250 spirals, is that right? Is that your reading of it?

A. Yes. Yes.

Q. And he then says, "Compute seismic loading case and potential P hinge regions", so do you agree that that demonstrates that he has identified parts of the columns, parts of columns as potential plastic hinge regions.

A. Yes it appears he has identified they could be.

Q. And he then goes on to calculate transverse reinforcement of the columns using the seismic provisions of NZS3101. Is that what he does?

A. Yes that's what it appears.

Q. And he arrives at R10 at 100 or R6 at 40 and then tests his spacing? Is that right?

A. Yes that's what it appears.

Q. And then his final words, "These do not apply as columns are non-seismic"?

A. Yes that's what he says.

Q. (inaudible 11:17:43) So on the face of his workings there his reason for choosing to design columns as non-seismic was because the columns are non-seismic that's what the documents indicates doesn't it?
A. It does.

CROSS-EXAMINATION: MR REID AND MR PALMER – NIL

QUESTIONS FROM COMMISSIONER FENWICK - NIL

QUESTIONS FROM COMMISSIONER CARTER - NIL

QUESTIONS FROM JUSTICE COOPER - NIL

WITNESS EXCUSED

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MR REID CALLS

JOHN STUART O'LOUGHLIN (SWORN)

Q. Your full name is John Stuart O'Loughlin?
A. Yes it is.
Q. And you area structural engineer living in Christchurch?
A. That is correct.
Q. Mr O'Loughlin could you please read your evidence from paragraph 2.

WITNESS READS BRIEF OF EVIDENCE FROM PARAGRAPH 2

A. From paragraph 2?
Q. Yes please.
A. I graduated with a Bachelor of Science from Canterbury University in 1968. I then commenced my degree in engineering and graduated with a Bachelor of Engineering Honours in 1970, also from Canterbury University. I then worked for the predecessors of what is now known as Holmes Consulting Group. I commenced my own practice in 1974, which was known as O'Loughlin Taylor Spence Limited. Since 1974 I have been engaged on a significant range of commercial and industrial buildings throughout Christchurch and the South Island. I have 42 years experience in the profession. Along with my business partner we sold the business of O'Loughlin Taylor Spence Limited to Babbage Consultants Limited in January 2012, and the name changed to OTS Limited. I am currently employed by OTS Limited as a consultant.

I was a chartered professional engineer and an international professional engineer until December 2010. I am currently a member of the Institution of Professional Engineers of New Zealand (MIPENZ).

Throughout my career I have dealt closely with Council reviewing engineers on a regular basis in Christchurch and in other locations in New Zealand. I am also regularly required to review the work of other engineers within OTS Limited and my current role involves a large element of supervision and oversight. I am currently being frequently asked to peer review the work of other consultants from outside our office.

I have been asked by the Christchurch City Council to provide the following evidence concerning the CTV building.

(a) The expectations of the engineering community in Christchurch in the 1980s as to where the nature of the structural review undertaken by the Christchurch City Council reviewing engineers in the context of considering applications for building permits.

(b) In relation to the non-compliances identified in the Hyland-Smith CTV building collapse investigation report, the Hyland report, how difficult it would have been for a competent Council engineer carrying out a review of the documents submitted with the building permit application, to have identified these alleged non-compliances.

(c) Any other relevant matters rising out of a review of the Christchurch City Council file for the CTV building permit and in particular the Council's approach to the processing of the 1986 building permit application.

I have read the code of conduct for expert witnesses and agree to comply with it. I confirm that all of the matters to be addressed in my evidence are within my areas of expertise.

Q. Just stop you there Mr O'Loughlin.

MR REID:

Sir, the next piece is just a reference to the material that has been reviewed.

JUSTICE COOPER:

Yes.

MR REID:

Do you want him to go through that?

JUSTICE COOPER:

No that can be taken as read, thank you.

THE COURT ADDRESSES MR REID – PARAGRAPHS 7, 8, 9 TO BE TAKEN AS READ

EXAMINATION CONTINUES: MR REID

Q. Just skip through please Mr O'Loughlin to paragraph 10.

WITNESS CONTINUES READING BRIEF OF EVIDENCE FROM PARAGRAPH 10

A. And recommence at?

Q. Paragraph 10.

A. Right. Background to engineering practise in the 1980s.
The early 1980s was the dawn of the computer era in engineering practise in New Zealand. Personal computers were available but with very limited storage capacity or capability. Larger mainframe computers were available on a time use basis, but most calculations for small and medium sized projects were completed manually using scientific calculators. I am not surprised to see the extent of hand calculations for the CTV building. The only reference I can find to computerised computations is at S11 to S23. You want me to read out that reference number?

JUSTICE COOPER:

No you can omit the numbers thank you.

EXAMINATION CONTINUES: MR REID

A. ...which appears to be the ETABs output. Software for structural analysis programs was available but programs for performing more sophisticated analysis came later. For example, the pushover analysis which was the result of research at Canterbury University during the early to mid 1990s and published about 1997.
In my experience, at the time the cost of a building permit for the construction of a building of a similar size to the CTV building would be approximately $2500. This fee would cover the cost of checking all

building permit issues including planning, car parking and traffic, fire, architectural and structural details. In my opinion the structural checking would have been likely to have taken approximately one full day in total taking into account the total fee and the other items that needed to be considered. By comparison, a full peer review for a building like the CTV building in today's terms would take five to six days by an experienced engineer and costs around $7000.

It was my experience however that at the time the CTV building permit application was processed, cost did not control the effort or time put in by processing officers to check a permit application. Their overriding requirement was to be satisfied that the building complied with the relevant bylaws. Difficult or complex projects was seldom put out for peer review to other consultants at the time the CTV building permit was processed.

The Council review process.

It is important to understand the role that the Council reviewing engineers had when considering plans submitted for permit building applications. I would draw a clear distinction between the design role of an engineer and a review role. Designers are tasked with ensuring that the building complies with the standards of the day and with coming up with a design that works within the instructions given by the client. It was not the reviewing engineer's task to debate these matters. Rather, the role of involved checking at a general level that the designer had considered and dealt with the compliance issues appropriately.

I have read the letter of 17 August 1986 to Alan Reay Consulting Limited from Graham Tapper on behalf of the Christchurch City Council. It was not uncommon to receive letters from Council staff after plans had been submitted on a building permit application.

JUSTICE COOPER:

"Similar letters," you left out the word, "similar."?

EXAMINATION CONTINUES: MR REID

A. It was not uncommon to receive similar letters from the Council staff after plans had been submitted on a building permit application. I regularly receive these and often dealt directly with Council engineers over the types of matters raised in the letter. In my view the purpose of such letters was to check that a design engineer had appropriately dealt with any queried issues.

I have read the two briefs of evidence filed with the Royal Commission by Peter Nichols. His recollection of the role

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played by a Council reviewing engineer broadly accords with mine. I note that Mr Nichols refers to a number of basic mathematic checks that he would typically carry out on a set of plans. If it was typical for Council reviewing engineers to carry out these kinds of checks I do not recall any reviewing engineer at that time referring me to his calculations when querying aspects of any plans I submitted. I therefore suspect that Mr Nichols' practice may have gone further than the practice of other reviewing engineers at other councils. Alternatively, the checks Mr Nichols is referring to would most likely have been at a very general level.

With the building consent process now in operation under the Building Act 2004 it is much more common for projects to be issued to a large or specialised consulting firm for peer review. Under the building consent process the applicant pays for all such costs.

From the 1970s up to the mid 1980s when the CTV permit was processed, I dealt with building permit applications and associated issues with the Dunedin City Council, Christchurch City Council and Wellington City Council on mid-sized to large projects as well as most smaller city and borough councils throughout the South Island. On a regular basis during my practice in Christchurch I dealt with both Bryan Bluck and Graeme Tapper at the Christchurch City Council during the years that they were employed by the Council.

I have always found the Christchurch City Council's staff to be astute and competent in handling the permit processing. The Christchurch City Council employed competent engineers who were respected and were active members of the local engineering community. The structural checking section did not have the staffing resources that some consulting engineering companies possessed, but they compared favourably with Dunedin and Wellington cities and more than favourably with the smaller councils and boroughs many of which did not employ engineers at all but relied on building inspectors to process building permits.

As intimated earlier, computers and software analysis programs were in their infancy in 1986 and I don't believe the Christchurch City Council made much use of these tools for structural checking at that time. I don't recall seeing computers in their offices and most checking was by visual study of the permit documents and the manual checking of calculations with calculators. The same was true of Dunedin and Wellington City Councils.

Q. I'll just stop you there Mr O'Loughlin.

HEARING ADJOURNS: 11.32 AM

HEARING RESUMES: 11.49 AM

EXAMINATION CONTINUES: MR REID

Q. Mr O'Loughlin you were at paragraph 21. Just carry on reading please.

A. I have read the Hyland report and the comments by Dr O'Leary on this report as to whether the CTV building complied with the applicable standards and the bylaws of the time. I now deal with each of these issues raised at pages 27 to 30 of the Executive Summary in the Hyland report in turn with respect to building permit processing. My comments take into account the expected time a reviewing engineer would have to process his or her section of a building permit which would be approximately one day in total. I have also borne in mind that the period when the building permit was processed was about the height of the property boom prior to the 1987 share market collapse and the Council would have been processing more than the average number of building permit applications.

From an initial inspection of the structural drawings it is clear that the lateral loads to the building were intended to be taken in the east-west direction by the east-west shear walls on lines 1 and 5 and in the north-south direction by the shear walls on lines C, CD, D and DE.

The detailing of the frames on lines 2 and 3 was consistent with gravity frames only and the block wall on line A, levels 1 to 3, was separated from the columns and beams and so was intended to act as a boundary firewall only.

Building Inter-Storey Drift Limits and Drift Capacity of Columns (and Column Confinement).

The application of the loading standard, NZS 4203, and the concrete standard, NZS 3101, to the design of concrete columns is a difficult area and there is room for varying approaches. I have read the evidence of Dr Arthur O'Leary, William Holmes and Ashley Smith's third statement. The varying views of these experts illustrates this point.

At a practical level Council reviewing engineers did not in my experience enter into the detail of these alternative options. The standards provided for a variety of approaches and it was up to the design engineer to provide a workable solution given the demands of the client. A Council reviewing engineer would not normally debate the design option chosen by the designer.

When it is obvious that the columns do not form part of the lateral load resisting system of a building the approach would have been that the columns did not need to be designed for ductility. Columns for these types of buildings were simply treated as props. In my view it would have been completely impractical for a reviewing engineer to carry out the kind of review necessary in order to make the fine judgments about the application of NZS 4203 and NZS 3101 to the design of concrete columns.

I note Dr O'Leary's view that the frame on line F did not comply with NZS 3101, in particular clause 3.5.14.3. I do not disagree with his view. However, when I reviewed the stamped and permitted plans for the CTV building, it occurred to me that the beam column joints set out on S19 could have been interpreted as being pin joints. I also note that this was apparently the intention of the designer, Mr Harding. The reason I make this observation is that the beam bottom steel shown on detail 7 of S19 is not continuous, rather, it is truncated at a bend. Certainly from the perspective of the Council reviewing engineer, this may well have been how the plans were interpreted.

In my view, the precise determination of whether the columns were required to be detailed for seismic purposes in terms of the standard required complex analysis. A number of the present day expert commentators giving evidence in these proceedings have used computer based mathematic modelling that would certainly not have been readily available to the Council reviewing engineers at the time the CTV building was permitted.

Minimum Shear Reinforcing of Columns.

The requirements of the reinforcing of the columns in the CTV building are dependent on whether the columns needed to be designed as seismic elements or not, as explained by Dr O'Leary at paragraphs 56 to 58 of his first statement. Dr O'Leary's view that the columns on grid F were required to be reinforced for shear relates back to his interpretation of NZS 3101 clause 3.5.15.3. As I have said at paragraphs 25, 26 and 28 above, in my view a Council reviewing engineer would not normally enter into a debate about the design options chosen for the building on these fine matters of interpretation. I therefore consider that the reviewing engineer could well have assessed the reinforcing of the columns on the basis that none of them needed to be designed to meet the additional seismic requirements of NZS 3101, and could have assumed that clause 6.4.7 of NZS 3101 applied. Indeed, the designer's calculations show the spiral reinforcing to be calculated as clause 6.4.7, which is without shear in the member.

I confirm that the spiral hoop reinforcement complies with clause 6.4.7 of NZS 3101 which covers limits for transverse reinforcement on columns and piers.

Spandrel Panel Separation.

This issue was raised in the Hyland report at pages 110 and 111. The report concludes that there was insufficient separation between the specified spandrel panels and the columns on line 1 and grids A and F.

I note Dr O'Leary's view at paragraphs 59 to 62 of his first statement that the plans specified a sufficient 10 millimetre clearance between each panel and the columns on either side.

Looking at the plans, the gap between grids 2 and 3 on plan S15 is specified as 7.5 metres. Each column is 400 millimetres leaving a clear distance between the columns of 7.1 metre. The spandrel panels are specified as 7.08 long on plan S25. Therefore I agree that the plans specified a 10 millimetre gap at the end of each spandrel panel.

I believe that a Council reviewing engineer without calculating the likely deflection could have reasonably considered that this an adequate clearance.

Beam Column Joints.

Whether the beam column joints were required to be detailed for seismic purposes is dependent on issues of interpretation and application of NZS 3101. Dr O'Leary deals with these issues at paragraphs 63 to 69 of his first statement of evidence.

For the reasons I set out in paragraph 25, 26 and 28 above in my discussion concerning the columns, in my view it would have been impractical for a Council reviewing engineer to carry out the kind of review necessary in order to make fine judgments about the application of NZS 4203 and NZS 3101 to the design of the column beam joints.

Plan Asymmetry and Vertical Irregularity.

In my view a Council reviewing engineer could reasonably have formed the view that the building was not particularly asymmetric. The building as specified had shear walls on both the north and south sides, and I believe that overall a reviewing engineer could have considered the building to be reasonably symmetrical about the centre of gravity.

The only qualification to this view relates to the block wall on line A, which extends over the bottom three levels. This wall is on the west side of the building, sorry this wall is on the west side of the building and is not mirrored by a similar wall on the east side. If the wall was intended to operate as a stiff design element, this would have introduced a degree of asymmetry into the design. However, the plans show the wall as being separated from the columns so it is clear it was not intended to operate as a shear wall.

The Hyland report indicates on page 112 that there may have been an issue with the vertical irregularity of the building. The issue seems to be related to the construction issues with the wall on line A. As discussed it is clear that the wall was intended by the designer to be separated from the rest of the structure. In my view, a Council reviewing engineer could have reasonably concluded that vertical irregularity was not an issue.

Wall on Line A.

As stated this was shown to be separated. If indeed it was not then that would have been a construction issue that occurred after the building permit processing.

Diaphragm Connection.

This matter is raised at page 113 of the Hyland Smith report. Dr O'Leary discusses the issue at paragraphs 77 to 82 of his first statement of evidence. Both Hyland Smith and Dr O'Leary conclude the diaphragm connections to

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the northern shear core wall were insufficient. Before dealing with how the Council reviewing engineer could have approached this issue, I would like to make some general observation about how diaphragm connections were understood during the 1980s.

In my view the question of adequate connection between the floor diaphragms and the lateral load resisting system was not a significant check item during the early 1980s, probably because at this time shear walls were generally designed to be of adequate total length in relation to total floor area. By check item I mean an item that would traditionally have attracted the attention of a reviewing engineer.

During the 1980s designers increasingly wished to provide open, airy and sunlit offices. Long boundary shear walls were often inconsistent with this requirement. The trend was to reduce the overall length of shear walls in relation to the floor plan areas. Consequently, the need to have an adequate connection between the smaller shear walls and the floor diaphragms was gaining more recognition than previously.

In addition, earthquake engineers became increasingly focussed on this issue following the October 1989 San Francisco earthquake. There were a number of well recognised instances where floor plates had separated from shear walls in buildings subjected to the earthquake. Reconnaissance teams from New Zealand visited San Francisco and came back with reports of what had occurred.

Therefore in my view by 1990 there would have likely have been increased awareness of the significance of the connection of diaphragms to shear walls in earthquake engineering, by comparison with the start of the decade.

I note that Mr Tapper did indeed identify an issue with connection between the shear wall system and the floor slabs in his letter of 27 August 1986. It appears to me that the structural plans to which Mr Tapper is referring in his letter of 27th of August were different to the permitted drawings stamped 30th of September 1986. Mr Tapper identifies a number of matters to be attended to in his letter. He indicates for example that, "S16 shear core floor slab and stair landing details are missing." The stamped copy of S16 clearly shows these details.

Equally, Mr Tapper states, "Sheet 15 incomplete notes. Reference line 1, HiBond mesh reinforced casing does not provide restraint to HiBond for FRR which stands for fire resistant rating purposes. Also floor connection to shear wall system and general connection between floor slab and walls." Mr Tapper may have been identifying an omission in the plans he was reviewing at the time related to the floor slab diaphragm connections. It may be that the stamped copy had some additional information on it that showed that the design engineer had considered the issue which had been raised.

Mr Tapper also requested a copy of the calculations to support the design in his 27th of August 1986 letter. The calculations I have reviewed show the mechanism to transfer loads from the slab diaphragm to the shear walls on pages S56 and S57. I note that the calculations mark the load "ok" at the bottom of S56. The calculations showing the mechanism to transfer shear to the shear walls are set out on page S57. The calculations indicated a reinforcing requirement of "D12 at 400 centres slab ties at core wall." This level of reinforcing on its own may not have seemed unusual, however the length of wall over which the force was to be transferred was only 1.4 metres which means there would have only been room for four D12 bars. I would not have considered that reinforcing to be adequate.

In order for the Council reviewing engineer to have picked up this lack of connection, a critical review of the calculations would have been required. In my view the issue should have been picked up by the reviewing engineer.

I also note that there is an error on S57 where the shear stress is calculated using a figure of 30,000 newtons whereas the maximum shear to the side walls shown at the top of the page, first line, is 300 kilonewtons, or 300,000 newtons. The figure appears to have been mistranslated. This has the effect of significantly increasing the load to be transferred and a significant underestimate of the required reinforcing. I note that this error has not been discussed by other expert witnesses. A line by line analysis of the calculations would have been required to identify this error and in my review it would therefore not have been readily apparent to a Council reviewing engineer.

JUSTICE COOPER:

Q. In your view. You said, "In my review," but the text is, "In my view."

A. Oh, yes.

EXAMINATION CONTINUES: MR REID

A. A line by line analysis of the calculations would have been required to identify the error and in my view it would therefore not have been readily apparent to a Council reviewing engineer.

Robustness.

The question of robustness is dealt with in chapter 9 of Hyland Smith report and Dr O'Leary discusses it at paragraph 83 of his first statement. My understanding is that neither NZS 4203 or NZS 3101 define the concept of robustness. In practice, in my view a building was robust if it complied with the standards of the day.

Documentation.

The issues concerning documentation are raised at page 114 of the Hyland report. In my view, the plans and specifications for the building were of a reasonable standard in common with the practice of the day.

Concluding comments.

I have tried in this evidence to convey to the Commission the general scene of building design and permit processing in the mid 1980s rather than discussing contentious technical issues such as whether gravity frames should be designed for ductility. I have left these issues to the many highly qualified experts to agree or disagree on.

In my view, the scope of a Council reviewing engineer's role in the 1980s was quite limited. A detailed peer review of plans and calculations provided on a permit application was impractical. The approach was as described by Mr Nichols in his first and second briefs of evidence. The Council reviewing engineers were checking that matters had been dealt with by the designer in general terms. They would not enter into debates about design options chosen. In relation to the drift capacity of columns and column reinforcement, in my view it would have been impractical for a Council engineer to carry out the kind of review necessary to make fine judgements on the application of clause 3.5.14.3 of NZS 3101.

In terms of the issues concerning the diaphragm connection, it is not clear to me whether the matters raised by Mr Tapper had been dealt with by amended plans being provided. A review of the calculations provided to support the design would have revealed weaknesses with the design approach.

The other matters identified as non-compliances in the Hyland Smith report which are spandrel panel separation, plan asymmetry and vertical irregularity, the wall on line A, robustness and documentation, are not matters that I consider would've raised the attention of a Council reviewing engineer.

Q. Mr O'Loughlin you've got a second brief of evidence in front of you?

A. Yes I have.

Q. If you just read that document please from paragraph 2 onwards.

WITNESS READS SECOND BRIEF OF EVIDENCE FROM PARAGRAPH 2

A. Since preparing my 18th of June 2012 statement I have had the opportunity to read the evidence of Geoffrey Nigel Banks dated 31st of May 2012. I wish to comment on paragraph 28 to 31 of this evidence. At paragraph 28 Mr Banks indicates that when he reviewed the calculations for the CTV building in 1990 he noted that there were no calculations relating to the connections between the floor diaphragms in the "two walls in question".

In my evidence I refer at paragraphs 40 to 49 to the calculations shown on S56 and S57.

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Q. I'll just stop you there, so the reference that's missing is BUI.MAD249.0272.65 and 66. Just carry on Mr O'Loughlin.

A. The calculations I was referring to relate to the connection between the diaphragms and the two walls on the western side of the north shear core wall and the wall on line 5. The walls to which Mr Banks is referring to when he notes the absence of calculations relate I surmise to the two eastern walls of the north shear core. Mr Banks was quite correct that there are no calculations relating to the transfer of load for an earthquake in a north-south direction to these walls.

I should note that the calculations on S57 refer to a wall length over which the load is being transferred of 1.4 metres on line 5 (also refer to diagram on S56). I note in fact that the length of the wall to which the load was transferred in the final permitted drawings is shown as a total length of 2.35 metres. One possible explanation as to how this may have been achieved is that there was a reduction in the dimensions of the air conditioning duct in the western corner line 5B/C.

It may be that this change of length of wall as referred to in the calculations and as shown on the final drawings was a response to the query raised in Mr Tapper's letter of 27th of August 1986. It is not possible to be certain whether this is the case given that, as I understand it, copies of the plans as submitted to the Council and prior to amendment are not now able to be located.

On the basis of my review of the calculations and plans, I believe that the amendment to extend the length of the wall over which the east-west load was being transferred would have led to my concerns with regard to the east-west loading being addressed and would likely have brought the building into or close to compliance for the east-west load transfer.

I note however and on this point I agree with Mr Banks that there were no calculations relating to the load, to the transfer of load for a north-south earthquake on the two eastern walls. The calculations on S56 and S57 give sufficient comfort of load transfer for a north-south earthquake for the two western walls of the north shear core but not those on the eastern side.

Q. Yes thank you Mr O'Loughlin, now there's one further matter. In the evidence of Antony Joseph Scott which is at WIT.SCOTT.0001.1 there's a matter that you wish to comment on. It's at page 8, if that could please be brought up.

WITNESS REFERRED TO WIT.SCOTT.0001.1

Q. Now the issue that Mr Scott is discussing here relates to a problem that he says he had identified with the construction of the, or the design of Durham Towers, that's correct?

A. Yes that's correct.

Q. And Durham Towers was a building that Halliday O'Loughlin and Taylor as the firm then was designed. Is that correct?

A. That's correct.

Q. And do you have some personal knowledge about the circumstances of the design that you want to relay?

A. Yes, he says that the lifts in this particular building stopped at the ground floor and didn't go to the basement. In the basement there was a laundry and I was, my recollection was that a lift went down to the basement and it makes a lot of sense because you wouldn't have people carting linen from the basement to the first floor to then transfer sheets and towels up to the 10th floor. So I both checked with the maintenance engineer last weekend, the maintenance engineer for Copthorne Hotel and Millennium Hotels and he confirmed to me that yes there was a basement laundry and yes one of the lifts did go down into the basement which is contrary to this evidence.

Q. And would that have addressed the concern that he's referring to about an inadequacy in the design of that building?

A. And he also mentioned that the walls didn't extend down to the basement and there was general concern that the weight of the lift walls and the weight of some water towers on the top of the building sitting on the lift walls would have overstressed the floor slabs and the walls simply sat on the floor slabs. I dug out from our archives the original consent drawings for that hotel and can confirm that there are block walls under the lifts which extend into the basement. There's three lifts in this building: one is a lift which services the laundry and the other two are passenger lifts which do only go down to the ground floor but there are block walls underneath those and the walls sit on beams. They just don't sit on the floor slab. There is a horizontal construction joint between the underside of the beams and the top of the, the top of the block walls of about 20 millimetres to separate the stiffness of the basement block walls and the floor slab at ground floor which is acting as a diaphragm to take loads out to the perimeter of the building.

Q. Thank you, Mr O'Loughlin. So that, with those clarifications that would have addressed the concerns that Mr Scott was referring to is that correct?

A. Yes and if the Commission wishes I can supply those drawings and I can supply the contact number for Mr Jolie who is the maintenance engineer at the Copthorne.

JUSTICE COOPER:

Yes well perhaps you could talk to Mr Mills about that later.

CROSS-EXAMINATION: MR ALLAN – NIL

CROSS-EXAMINATION: MR PALMER

Q. Mr O'Loughlin you've as I read your evidence have never worked as a Council checking engineer have you?

A. No I haven't. I've been, all my evidence is given from the public side of the counter.

Q. Yes. So on, on matters directly relating to the checking that was done or might have been done in those days I imagine you would defer to those witnesses that have direct experience of those issues such as Mr Nichols?

A. Mr Nicholls, Mr Henry, both worked on the, for the Council as checking engineers.

Q. In paragraph 12 of your evidence you mention that your assessment of how long it would take would be approximately one day but you, you recall that comment?

A. Yes.

Q. And you would however accept that it might have taken several days for certain projects, very large projects as Mr Nichols has stated in his evidence wouldn't you?

A. Yes. And you take into account broken time as well.

Q. Yes. Given the way the cost structure worked at the time is it the case that effectively on the larger jobs there was something of a Council subsidy for the additional work required for larger or more novel propositions to be reviewed?

A. Yes that would be a reasonable assumption but in my, because in my view they didn't just stop on a cost basis, they tried to see it through to coming to a reasonable conclusion.

Q. Yes I think you've noted in your evidence that the overriding requirement was to be satisfied that the building complied with the relevant bylaws?

A. Yes.

Q. That's your paragraph 13?

A. Yes.

Q. Would you accept then that the Council checking engineers would simply allocate such time as was necessary to achieve that overriding requirement?

A. In general, in general yes.

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Q. And one imagines by inference that something that's traditional, relatively straightforward, would be allocated less time than something that was out of the ordinary or perhaps very large.

A. Yes and your comment about subsidy, there were swings and roundabouts. I did buildings which were an exact repeat. They were service stations.
Q. Yes.
A. They were all exactly the same. So obviously once they've done one it was simple to do another one, process a permit for a second one, third one and a fourth one. So that's where the subsidy was taken up. Then you get a more pioneering design and you spend more time on it.
Q. In 1986 you were in practice in Christchurch weren't you?
A. Yes.
Q. Did you know Mr Bluck personally?
A. Yes. Well not that I visited his house but that I would talk to him as Bryan at engineering meetings and –
Q. In a professional sense.
A. In a professional, yes.
Q. And you knew him to be in charge of the Council's engineering review code compliance team didn't you?
A. Yes he was.
Q. Would you accept that all of those that worked in his team would follow his practices?
A. Sorry that truck went by and I couldn't …
Q. I'll rephrase it anyway. Would you accept that all those that worked in his team followed practices that he directed?
A. In a general sense, yes.
Q. And Mr Nichols was one of those that worked under Mr Bluck.
A. Yes. At a different time to when Mr Tapper was there I believe.
Q. Is it reasonable to assume that if Mr Nichols was following practices laid down by Mr Bluck so too would Mr Tapper have been?

MR REID:

Well Sir it's my submission it's completely separate. This witness is not giving evidence about what occurred in relation to the CTV building between

Mr Bluck and Mr Tapper and it's just entirely speculative for him to be asked to comment on it.

JUSTICE COOPER:
Yes. Mr Palmer?

MR PALMER:
I won't pursue that question.

CROSS-EXAMINATION CONTINUES: MR PALMER
Q. Well in paragraph 16 of your evidence you say that you suspected Mr Nichols' practice may have gone further than the practice of other reviewing engineers at other Councils.
A. Yes.
Q. Were you here yesterday when Mr Jacobs gave evidence?
A. Yes I was.
Q. Well Dr Jacobs I should say.
A. Yes.
Q. And did you hear him say that the Auckland City Council had a very careful seismic checking procedure that they ran through their code compliance team?
A. Yes I'm not sure the date that he was referring to. They certainly would, as all other councils would have now that the 2004 Building Act has been passed. I am not familiar with the Auckland City Council practices and I excluded those from my, in my, which councils I have dealt with.
Q. So Auckland's excluded from the other locations you refer to in paragraph –
A. I mention Dunedin, Wellington and local councils.
Q. In paragraph 35 of your evidence you say it would have been impracticable for a Council reviewing engineer to carry out the kind of review necessary in order to make fine judgements about the applications of the two relevant codes, 4203 and 3101, to the design of the column beam joints.

A. Yes.

Q. How do you reconcile that with your evidence that the over-riding requirement of the Council team was to be satisfied that the building complied with relevant bylaws?

A. To decide whether the building was to be designed, the gravity frames were to be designed for, for seismic purposes you needed to do quite a complicated calculation, analysis which had been done by ETABS analysis and the calculations which had been provided to the Council and – so that was my comment that the Council simply did not have that capacity within their office to do that sort of calculation.

Q. And in terms of –

A. And that's in terms of fine judgements rather than general judgements.

Q. What about compliance with simple code standards such as the spacing of spiral reinforcing?

A. Yes they would, they would, if they were worried about that they would ask questions of the designer, had he considered this, please provide evidence that this complies in terms of the codes and bylaws. They, in my view, seldom said, look we've done these calculations, here they are, here's a copy of them, you compare our calculations with yours and then see who's right. It didn't happen in that way. The process was that if they saw a deficiency they asked the designer to provide proof of evidence that that design complied and that's in the bylaws and the introduction to the bylaws. I think it's something like clause 5 where you talk about proof of evidence.

Q. Well are you familiar with clause 9.4.8 of NZS 3101?

A. I have read it, yes.

Q. Do I need to show it to you, or if I read the relevant part which says that in terms of confinement, "In no case shall the stirrup-tie spacing in the joint core exceed 10 times the diameter of the column bar or 200 millimetres whichever is less."

A. Yes.

Q. Do you recall that?

A. Yes.

Q. Could you just bring up please BUI.MAD249.027344. If you have a look in the right column you can see the reference there to, in terms of the column – R6 at 250 spiral.
A. Yes.
Q. It's reasonably plain here that this calculation provided by Mr Harding to the Council showed a spiral spacing at 250 which was in excess of the minimum set out in clause 2.4.8 doesn't it?
A. Yes it does.
Q. Would you not expect a Council checking engineer with the requirement to ensure compliance with the code to have noted this inconsistency with the code?
A. Well at the beam column joints, I did make the statement that the documents were of a reasonable standard but to look as a reviewing engineer at that beam column joint you had to look at the column joint. There was four drawings which you had to visualise and assemble in your mind as to what was the particular arrangement of reinforcing in the beam column joint.
Q. And if, if those plans also showed the spiral reinforcing to be at 250 millimetres –
A. Quite clearly.
Q. – would you not then expect the checking engineer to have picked that up, that issue up?
A. That particular clause that you refer to, 9.4, is dealing with beam column joints. The spirals at 250 centres, as I've said in clause 6.4 if you're looking at design of columns and piers without shear you would have looked at that and said, yes, well that's all right.
Q. However, if you'd looked at the same plans and seen as they show that the spirals go through the beam column joints at 250 rather than 200 would you accept from me that the Council engineer should have identified that inconsistency?
A. Yes he, to get all the information on the pitch of the spirals he had to go to four different sections of 3101. When he looked –

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Q. Yes but in terms of the beam column joint the minimum spacing as you have accepted under 9.4.8 was 200 millimetres wasn't it?
A. Yes, yes it's clear there.
Q. So if the plans clearly showed reinforce – spiral reinforcing through the beam column joints at 250 would you not then expect the Council checking engineer to have identified that inconsistency?
A. I think it's not clear for him to identify that and so it –
Q. What do you mean by that?
A. Well he looks at the columns and says yes, these are columns that not part of the lateral load resisting system. You then go to the drawing of the columns, see the pitch at 250, you look at columns in pairs without shear, 6.4, you say yes that complies. It gives them some sort of comfort. You then look at, try and look at the beam column joint. You don't have a clear detail of that, you have to assemble it by looking at the reinforcing at the ends of the precast beams. You look at that column drawing I was referring to. You'd look at a drawing which shows a plan but doesn't show all the reinforcing and you look at the slab plan to see where the top reinforcing in the beam column joint is.
Q. Perhaps we need to bring this up but it is plan S14. What I am going to suggest when S14 comes up, I will give you a number at the moment, is that the plans clearly show the spiral reinforcing going through the beam column joint at 250 spacing. It's BUI.MAD249.028 – it has come up, if you could enlarge the left column, the left-hand diagram please.

JUSTICE COOPER:

Let's just get the number in the record please. BUI.MAD249.0284.15.

CROSS-EXAMINATION CONTINUES: MR PALMER

Q. Now if you look please here at the detailing shown of the R6 spiral and you will note, I think you will accept from me that it shows here that the R6 spiral at 250 pitch goes right through the beam column joint, doesn't it?

A. Yes it goes from the underside of a beam to the underside of the next beam, it is not clear how the joining of those spirals occurs.
Q. No but nevertheless it is showing the spirals at –
A. Yes.
Q. - 250, now that is not in accordance with –
A. With the beam column joint.
Q. – 3101 is it?
A. But that is not a detail of a beam column joint. That is a detail of a column. I am just looking at it from the way, you are not a designer as a reviewing engineer. As a reviewing engineer you are given specifications, given drawings, you are given the information and it is not how you would do it. You look at what is presented and so you flick between drawings and you look at one item at a time so – and that drawing which is on the screen at the moment I would look at that and say, ah, columns. What is the spiral reinforcing in the column. I wouldn't look at that drawing and think about beam column joints.
Q. But the calculations were also requested, weren't they?
A. Yes, they were.
Q. And they referred to 250 spacing as well didn't they?
A. Yep.
Q. So wouldn't the checking engineer want to verify that the spacing through the joints was at 250 and looked for such a calculation or a plan?
A. He – it would be something he would want to look at, yes. He would have difficulty in visualising that arrangement.
Q. And you would accept wouldn't you that without identifying that, the reinforcing through the joint was at 200 millimetres or less in accordance with 9.4.8, the checking engineer wasn't undertaking his primary requirement to ensure compliance with the code. Would you accept that?
A. In fairness of, if he missed seeing the detail which was clouded in terms of the beam column joint then he has missed something which maybe he should have picked up.

Q. Could you now bring up BUI.MAD249.0141.8 and could you enlarge the bottom of the page please. I am simply showing this to you Mr O'Loughlin because you were in practice in Christchurch in 1986?
A. Yes.
Q. If you look at the columns and the signatures and the column about four from the left, headed, "STR."?
A. Yes.
Q. You see that signature there. Do you recognise that at all?
A. No. As a signature do I recognise it?
Q. Yes?
A. No, I don't.
Q. And –
A. But I could construe it would be GT.
Q. It would be Mr Tapper's signature?
A. Yeah. I haven't the recollection of what his signature looked like but it is obviously a column STR stands for structural and of the other people, knowing that Graham Tapper was reviewing the structural design that is most likely his signature.
Q. And if you look now beneath the columns and you see the word, "Received 26/8," it looks like, again like the tail of that signature that you have just identified as being Mr Tapper's?
A. Ah, yes –

JUSTICE COOPER:
Look Mr Palmer the witness is hardly a handwriting expert is he.

MR PALMER:
I was simply going to ask one more question which is whether he identified that writing as Mr Tapper's or not.

JUSTICE COOPER:
Well he said a few minutes ago he didn't recognise the signature.

MR PALMER:

He said he recognised the signature I was going to ask about the writing "Received."

JUSTICE COOPER:

I thought he said he didn't recognise the signature but perhaps I am not listening properly.

MR REID:

That is my note Sir, he said that he didn't recognise the signature but that he could construe that it said GT.

JUSTICE COOPER:

Well yes but he's a witness, he should be giving evidence about matters of factual opinion not on handwriting.

MR PALMER:

Well Sir he was present in 1986 and he may have been the recipient of Mr Tapper's writing and I was simply going to ask if that was –

JUSTICE COOPER:

Well I am not going to argue with you Mr Palmer. Answer the question Mr O'Loughlin.

CROSS-EXAMINATION CONTINUES: MR PALMER

Q. Do you recognise the writing, "Received 26/8."?
A. I don't recognise it no.

CROSS-EXAMINATION: MR MILLS

Q. Just a couple of points first Mr O'Loughlin. I take it your evidence is that there are areas of the CTV building that in your view were non-compliant at the time of permitting?
A. Yes.

Q. But you say that it is unrealistic to have expected the Council reviewing engineers to have picked that up, is that where you get to?

A. Well it wasn't his job to fix them up. It was his job to point them out.

Q. But –

A. But they did get fixed up by the design engineer.

Q. But there are, I think you would agree, issues with this building that other people have identified as potentially non-compliant, which you don't think the reviewing engineers could have been expected to identify, that is your position is it?

A. There is one or two, for example the claimed asymmetry of the building is not as bad as I believe has been commented on.

Q. What about the question of the spirals and the columns that you were just being asked about? I thought that you were saying that that wasn't something, that the Council reviewing engineers could be expected to pick up?

A. The fine judgement of the spirals wasn't something that, he did probably check as a column but not as a column working in a beam column joint.

Q. So in other words the question of whether or not those columns were compliant with the code is an issue that you say the reviewing engineers couldn't be expected to pick up. That is your evidence isn't it?

A. It is in terms of the fine judgement that you make about how compliant they are.

Q. So you say they should have picked up that it was compliant – or non-compliant but not how badly non-compliant. Is that where you get to?

A. No, the fine judgement on whether there was drift capacity of the columns and therefore whether it needed to be detailed for ductility or not.

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Q. So in other words the question of whether those columns were code compliant, you're saying that was not a matter that one could realistically expect the Council reviewing engineers to have picked up?

A. All, to get, come to a perfect solution.

Q. Could we just have a yes no and then you can amplify but it would really help us to move along if where you can give a yes no answer first that would help then you can add whatever you –
A. And then qualify it. Yeah.
Q. So you're saying that that's an issue that you couldn't realistically expect the Council reviewing engineers to pick up?
A. Which issue?
Q. The question of whether there was adequate confinement in the columns and whether they were code compliant or not as a result?
A. I would expect that he should have been able to pick it up.
Q. So they should have been able to identify that that wasn't compliant with the code? If that was the case?
A. Yes.
Q. All right. Now just make sure then that there's no misunderstanding here. Are you saying any area where the CTV building was not code compliant, any area, they should have all been picked up by the Council reviewing engineers?
A. No not all areas.
Q. All right so you are saying then that there are areas where the CTV building was not code compliant but where it was not realistic to expect the Council reviewing engineers to identify that?
A. That's right.
Q. Right. Now that, that statement that you've just made, are you in a position to say whether that limitation on the expectations that one would have as a structural engineer of what the Council reviewing engineers would be capable of doing whether that would have been a widely shared, shared view amongst Christchurch structural engineers in the 1980s?
A. On some buildings yes.
Q. How about on the C, a building like the CTV building?
A. It was a pioneering building. I wouldn't, it would have stretched the capacity of the Council staff to fully understand how that building was working.

Q. Right.
A. And I suspect that the Council engineer, Mr Tapper, reviewing it didn't have other colleagues working in the office with him who he could discuss it with. If that became a, a issue in a, for say a peer review in a consulting office there would be two perhaps three engineers would –
Q. Yes.
A. – discuss it amongst themselves and he didn't have that advantage.
Q. All right. Now you've had according to your evidence a lot of experience of mentoring and reviewing the work of others, other structural engineers presumably younger structural engineers, less experienced structural engineers?
A. Yes.
Q. Is that correct?
A. That's correct.
Q. If you in your own career had an inexperienced structural engineer working for you – inexperienced in doing buildings of the kind that the CTV building was – would you consider that it was appropriate to put them on to the job of designing the CTV building with essentially no supervision but on the basis that you could rely on the Council reviewing engineers to pick up any deficiencies in that design?
A. No it wouldn't be appropriate.
Q. No. Right let me then ask you about one of the principal areas I think where you have said that you think that the reviewing engineers should have picked up the problem with the building and I take it that's the connection between the floor diaphragms and that north shear wall?
A. Yes.
Q. And you say that should have been identified?
A. I think I was referring to the drag bars largely because he did ask the question of the design engineer about the general connection of the floor slab in that area to the wall on line 5.
Q. This is the question Mr Tapper raised in that letter I take it?
A. Yes, yes.
Q. That was referred to.

A. Yes.
Q. I'm not quite clear what it is you're saying there. Are you saying that you think that that Mr Tapper did pick up –
A. He –
Q. – the problem?
A. He, he picked up that there was limited connection between line 4 and the wall on line 5.
Q. Yes.
A. In terms of the slab and the diaphragm connection to those walls but he didn't ever look and he missed that for those, for that diaphragm to walk – to work and for in a north-south earthquake for the slab diaphragm to be connected to those north-south walls that you needed some sort of better connection than what was there at the time.
Q. Well of course we don't know what he picked up do we? All we know is what he said in that letter?
A. That's exactly right.
Q. Yes.
A. We don't know because in my experience there was a lot of phone connection between a review engineer and the designer.
Q. Yes.
A. And they would ring you up and say, "Hey what do you, have you looked at this?" and we haven't got that evidence.
Q. Yes so when you say what Mr Tapper looked at or didn't look at, you're speculating aren't you all you know is what's in that letter?
A. That's right.
Q. Mhm. You've looked I take it at the permit drawings for the CTV building?
A. Yes I have.
Q. Do you agree that those permit drawings still show a non-compliant connection between the north shear core and the floor diaphragm?
A. The permit drawings, yes because the drag bars aren't shown –
Q. Yes.
A. – on them.

Q. So whatever concern Mr Tapper might have had about that connection you'd agree it clearly wasn't met in the permit drawings because they remained non-compliant?

A. In that respect yes.

Q. Mhm. Now as I understand your evidence it is that what the Council reviewing engineers are doing on their initial inspection should pick up sort of basic structural problems with the building. Is that really the essence of the, of what you're saying?

A. Yes if, if you look at the bylaw it actually talks about examination of, in relation to consent fees the particular clause talks about examination of drawings.

Q. Yes, so you'd expect it to pick up basic structural issues about the building when they look at it?

A. Yes.

Q. And I think you said in your evidence that, for you any rate, that initial review would pick up that it was what one of the witnesses Mr Henry has referred to as a shear wall protected gravity load system?

A. That's right.

Q. Yes. Now for you any rate, if you were to do, and I know you're not the Council structural, the Council reviewing engineer, but if you were to do an initial look at those drawings would there be any concerns that would have been triggered by you looking at it?

A. Yes there would have been.

Q. What were those?

A. Well I tried initially to put about two or three days into looking at the drawings and the documents as is, as the Council would have been presented with.

Q. Yes.

A. And one of my concerns is the mechanism between line 4 and line 5. We had a very stiff wall on line 5 that half the shear, 300 kilonewtons as the calculations show–

Q. Yes.

A. Arrive at line 4.

Q. Yes.
A. And I looked at you have to get that shear out to line 5 you've got a lift shaft so you can't transfer it through the lift shaft?
Q. Yes.
A. Because it's a hole.
Q. Yes.
A. You've got a stairwell with landings on the north side of the stairwell so you can't transfer loads through there and you have between lines C and CD a roughly 3.5 by four metre section of slab.
Q. Yes.
A. With a duct in one corner.
Q. Yes.
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A. So what I came up with is that to transfer those loads across there's, there's four items. There's the bending out-of-plane of the four north-south walls because at line 4 the slab impinges against the ends of those walls and bends them out-of-plane and that's quite a soft, it's not a rigid connection, and then you have that four metres by 3.5 metre section of slab with a hole in one corner.
Q. Mmm.
A. And that has got to transfer load through that in terms of shear.
Q. Yes.
A. And it, it suddenly made me realise that it doesn't matter how stiff that wall is on line 5. It could be of infinite stiffness.
Q. Yes.
A. It wouldn't affect the torsional rotation that much because the stiffness is in that connection between line 4 and line 5 and that that may not be as stiff as the south wall on line 1 which has been described in the calculations as a coupled shear wall, two, four, two two metre walls with a connecting coupling beam between them.
Q. Yes.

A. But if you looked at the elevation of that wall you would assume that it's a cantilever wall five metres long.
Q. Yes.
A. And it's 400 thick compared to the wall on line 5.
Q. Yes.
A. So I came up in my little analysis with this wall on line 1 could be as stiff as the wall, as the connection, the mechanism on line 4 to 5.
Q. Yes. All right.
A. Which changes – and so that, it's an area between four and five of very subtle complexities.
Q. Mmm.
A. And I think it's, that point I've just made has been missed by a lot of people.
Q. All right. And that's an issue which I think you said, what did you say, you'd spent one to three days on, on this.
A. Yes.
Q. To form that view.
A. I've spent more time since but –
Q. Yes, no I understand that.
A. But I wanted to, to me that was important because your drift demand on the columns depends to a large extent on rotation.
Q. Yes.
A. And if the centre of rigidity is a long way from the centre of gravity -
Q. Yes.
A. – you've got a coupling moment which is going to twist the building and provide extra drift demand on the columns.
Q. Yes I've certainly come to learn that during the course of this hearing.
A. If you don't have that, if the, the effect of what I've said of, hey, line 1 is probably stiffer than people imagined, and certainly the evidence of the details when they went down to Bexley and looked at the panels from line 1 indicate that it did work as a single cantilever shear wall –
Q. Yes we've heard that.
A. – with stiff coupling beams.

Q. Yes.
A. E1, E2, E3 I think it is.
Q. Mmm, yes.
A. In the Hyland Smith report.
Q. Can I ask you just a couple of questions in relation to that. The first is whether you think that a competent structural designer ought to have appreciated that problem in the course of designing the building? So that's my first question.
A. I think so. Someone like me grew up doing hand analysis with a slide rule.
Q. Yes.
A. Someone like Mr Tapper, similar age, he, you looked at things from first principle.
Q. Yes.
A. And, and you followed the load paths from where the mass was excited by the earthquake and looked at what was going to resist it.
Q. Yes. So I think your answer is that you think that a competent structural engineer should have picked up this issue that you've just been describing?
A. Not everyone has picked it up.
Q. No.
A. Because it's got subtle complexities.
Q. Yes I understand that but on your own view of what is a competent structural engineer do you have an opinion on whether this ought to have been picked up?
A. I think he should have picked up that the, that the torsion isn't as great as what it appears because you've, you have this very stiff element out on the north side. This is unusual about this building that the, the shear core was pushed outside the building.
Q. Yes, no, I think we all understand that.
A. And so you immediately hone on, we've got this whacking great stiff wall out here.
Q. Yes.

A. On line 5 but what's the influence of that compared to the balancing wall on line 1.

Q. Yes. Now let me just ask you then –

A. And so without the, the full implication of that you would start to see that maybe it's not as torsionally sort of design building –

Q. Yes.

A. – as it would first appear.

Q. All right. Now my first question which you've answered is about what a competent structural engineer should have picked up. Let me ask you then in relation to the Council reviewing engineers, just so I'm clear on what you're saying here, are you saying that you think that a Council reviewing engineer also should have identified the general issue that you're describing if not with the same level of sophistication?

A. In, in the general issues, yes.

Q. All right. Thank you. Now this may be the same issue that you've just referred to in what you've been saying but we've heard evidence from both Mr Nichols, and you know who he is –

A. Yes.

Q. – he was an ex-Council reviewing engineer, and Mr John Henry who also at one stage in his career was a Council reviewing engineer, that very quickly the imbalance between the north and south shear walls, they identified as a concern. Is that the same issue you're describing or a different one?

A. No I'm, I'm saying that there is an imbalance on paper as you just look at it.

Q. Yes.

A. A very stiff wall on line 5 and 11½ metres long, 300 thick, compared to a five metre long wall 400 thick on line 1.

Q. But you think it's less of an imbalance on close analysis?

A. But it's less, it's less and, and this is the problem with modelling that if, if you had something on line 5 which is of infinite stiffness, and for infinite stiffness imagine one of the great pyramids of Egypt, it just wouldn't move.

Q. Yes.
A. And then you put a rubber connecting beam, horizontal beam between that point of infinite stiffness and the north side of the floor plate, line 4, and you modelled that mechanism.
Q. Yes.
A. Would you model the stiffness of the wall on line 5, of infinite stiffness or would you model the flexibility of that connecting beam?
Q. And what's your answer to that?
A. You model the connecting beam.
Q. Yes right thank you. Now, so we've got agreement then that this issue that you've been describing, Council reviewing engineer should pick it up, once the Council reviewing engineer identifies the issues you've just been describing should that trigger any other concerns about the building's design in your view?
A. There would, it would be enough for him to ask questions of the designer about this imbalance as you say.
Q. Yes.
A. He's a good designer, a good reviewing engineer would pick up the, initially it looks like a bad imbalance.
Q. Yes.
A. It's probably not as bad as you think when you consider the mechanism but it's still there, what have you done to address this?
Q. Yes. So that's the process really -
A. Yes.
Q. A kind of iterative process which you say the Council reviewing engineer should have engaged in with the designer once the big picture problem had been identified. Then they go to the designer and say, I've got questions for you that you need to answer. Is that the process that you're describing?
A. Yes.

HEARING ADJOURNS: 12.59 PM

HEARING RESUMES: 2.15 PM

CROSS-EXAMINATION CONTINUES: MR MILLS

Q. Now you said this morning, at least before the adjournment, that you referred to some areas of non-compliance which you thought the Council reviewing engineer should have picked up. I think we've dealt principally with the diaphragm connection. Are there others that you have in mind where you think that one could have expected the Council reviewing engineer to have picked it up?

A. I thought that drag bars in the north-south direction.

Q. Yes I was aware of that, but you've referred to "some" and I just wondered if you had any others where you thought also in addition to that?

A. No, of those issues brought up by the Smith Hyland report –

Q. That's the principal one is it?

A. – that's the principal one and the one of shear connection onto line 5, he did ask the question about that so.

Q. Yes, and the issue of the line F columns, where you've accepted I think that there's non-compliance. That's not an area that you think should have been identified as a potential issue?

A. If he was looking at, line F is very dependent on the amount of torsion in the building and the distance between the centre of mass or gravity and the centre of shear resistance, and if he was looking as I looked at it, that those were closer together than a lot of the witnesses have put in their statements then he may not be concerned so much about line F or line 1 which are the lines most extreme from the centre of rigidity.

Q. So you don't think that's an issue that at the level of understanding that I think you thought a Council reviewing engineer would probably have had after doing an overview of the building, you don't think that that would've caused concern about potential issues on that line F column line?

A. He may have looked at the, on line F the spandrels and whether they interacted with the columns or not. I think he would probably have satisfied himself that there was sufficient clearance there and that they weren't that rigid anyway.

Q. But not, you wouldn't think there'd be any concern then about the lack of any ductility in those line F columns?

A. Again he didn't have the opportunity to look at a detail of the beam column joints on line F or line 1 to critically appraise it. He would've had to assemble that in his mind.

Q. Although you said, didn't you, in your evidence that you thought that a Council reviewing engineer could have concluded that the beam column joints were pin joints?

A. Yes.

Q. Doesn't that suggest a level of scrutiny?

A. That, that came from a detailed, part of the detail of a beam column joint that the steel wasn't continuous at the bottom of the beam.

Q. Yes, and as I understand it you're saying in your evidence that the Council reviewing engineer could have identified that?

A. Yes.

Q. And you don't apply that level of analysis equally to the confinement in the columns on line F?

A. I think you wouldn't look at all the columns, you would look at one set of columns and see how they were detailed.

Q. You don't think line F would've been seen as one that was particularly vulnerable?

A. I don't think he would've seen that as very special compared with any other line.

Q. Now do you agree with me that in fact the, that column joint is not a pin joint?

A. Sorry, just repeat?

Q. Do you agree with me that that joint that you've described as a pin joint is not in fact a pin joint?

A. Yeah, it's held in by the beams on each side of it.

Q. Yes.

A. By the top steel.

Q. Yes, so if the –

A. I suspect the bottom steel doesn't really go far enough in to develop tension capacity, but I accept that the top steel would hold the beams on each side of the column together sufficiently that it in fact didn't work as a pin joint.

Q. Yes, so when you say that a Council reviewing engineer could've concluded that it was a pin joint, that conclusion would've been wrong?

A. If he concluded that, yes it would be wrong.

Q. Now the, you agreed I think with Dr O'Leary that there was non-compliance with the reinforcement for shear, correct? In the columns?

A. In the column, in the beam column joints.

Q. Yes.

A. The 200, it wasn't at 200.

Q. Yes and I think you said in paragraph 29 of your evidence that the Council reviewing engineer could have concluded that that was okay?

A. If he had made the assumption that it was a pin joint.

Q. Yes, so once again that would've been wrong to make that conclusion?

A. Yes.

Q. Yes. Do you agree with me that the areas of non-compliance which have been identified, only some of which you say the Council reviewing engineer could've been expected to pick up, appear to have been significant weaknesses in relation to what happened on the 22nd of February?

A. If you, in hindsight it would be easy to say that. At the time he was reviewing the drawings it would be much more difficult to do.

Q. But in hindsight I take it you do say yes to that question?

A. In hind – the benefit of hindsight, yes it's a great thing.

Q. I just want to ask you just a few questions now about your evidence around the 1980s as being a period of a property boom?

A. Yes.

Q. And if you need to it's paragraph 21 of your evidence.

A. I remember it.

Q. But they're general questions. I just want to get clear what it is that you're saying in that part of your evidence. Are you saying that because of that period being a property boom that realistically one couldn't expect as much time and care with the reviewing process within the Council as one otherwise would have?

A. That's part of the reason. That they would've been busier than normal and therefore they may, and I don't say they did 'cos I wasn't there on that side of the counter, but they may have shortened the time they gave to each job. But the second thing in a property boom is that you don't get these clients, like I did a design job of a multi-storey building for Lloyds. They said this year you set out the brief, the next year we design it, the year after we'll get it, in the next two years we'll get it built. So it was a planned four year process. In a property boom you get developers putting an option on a site. They then get a quick design done before their option expires. They then go to the bank, get a price, go to the bank, get a loan for building it, bridging finance, then the pressure's on to get it done so part of that pressure in a prop – in a boom is brought about by the developers which working and time is very much the essence for them and they push people like myself to get the drawings done, get the consent through, we've just landed so many million in our bank account and we're paying interest on it sort of thing.

Q. Yes so when –

A. So that's the other side to it. The property boom does bring about this demand from the public that you shorten the processing time for consents.

Q. Yes, so referring to your own experience in practice in Christchurch during that period, do I take it that you would have from time to time been putting pressure on the Council to speed up getting a permit through?

A. I have yes.

Q. Would that include putting in staged permit applications?

A. Yes, yep, the foundations so that you got it underway and designed the superstructure as a stage 2 application.

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Q. Yes and when you say you were engaged in putting pressure on the Council to get the permit through, what form would that pressure take?
A. If they rang up or sent me a long list of things which were complicated you would try and expedite them as quickly as possible.
Q. Mmm.
A. If they appeared that they were set in their idea of what they expected of your design then you might say that it's unreasonable. It was just a general look. In the bylaws you're supposed to issue a consent within 20 days.
Q. Yes.
A. 20 days is up, where's the consent.
Q. Yes and so you'd be pushing at that point.
A. Yes.
Q. Saying where's the consent.
A. Yeah.
Q. And would this include going into meet with the senior building staff officers to try and get things moving?
A. Not generally. You'd do it by phone call.
Q. Right and who would you call when you would call?
A. The person who's writing the letters to say, we want more information on this, that the other and the next thing.
Q. Right and would you on occasion go directly to Mr Bluck?
A. I didn't usually go over the head of the person I was dealing with.
Q. I take it you wouldn't do it usually but I take it you would do it sometimes.
A. I possibly have done it –
Q. Yes.
A. – on one or two occasions.
Q. Now just to come back to this type of building that you're describing and the building environment in the 80s, developer driven, speculative developments, high cost of money, it was very high interest rates back then wasn't it.

A. Yeah, 20%.

Q. Do you agree with me that in fact far from giving less time at the reviewing stage within the Council to these buildings during this period inherently these were the types of buildings which required closer scrutiny because they were riskier?

A. Not always, no. I mean someone might have wanted to put up a factory which is a very basic building, tilt panels, it would be a very simple building but they just wanted it done because they thought they had someone who could rent it.

Q. Do you agree with me that quite a lot of these developer buildings were not going to be held long-term by the developer?

A. That's right. They wanted to sell, on-sell them.

Q. Build them and move them on.

A. Yes.

Q. Does that not suggest that there might be less quality go into the buildings than if it was an institutional owner such as AMP Trust or in modern days Kiwi Property Trust that would be a long-term holder?

A. That's one aspect of it but the other side is that they wanted to attract a tenant before they could sell it, on-sell it.

Q. Yes.

A. And the best rent would be paid by the best tenants. So there was advantage in designing a high quality building, well thought out, to get the best tenant, to get the highest rental so that you sold it at the maximum price.

Q. Tenants wouldn't normally look below the surface of the quality of the structural engineering work would they?

A. Some would.

Q. Some would, yes. All right. Now if as I think you're saying the Council itself through its reviewing engineers and the building department cannot be expected to identify all of the structural weaknesses that a building might have, how does the Council satisfy itself that a permit should be issued? Now in your experience of dealing with the

Christchurch City Council in the 1980s, was it common or characteristic for design certificates to be required for buildings?

A. Yes, generally.

Q. It was.

A. And we used to supply a design certificate.

Q. And when you say generally, in the cases where one wasn’t required by the Council, do they fall into any kind of category so that you can say, well the ones where they weren't asking for them are this type of building, or was it more random than that?

A. No you either supplied calculations or you supplied a design certificate.

Q. And was that the choice of the permit applicant or of the reviewing engineer?

A. The, usually the applicant.

Q. I see. Now –

A. And the Council didn't have to accept the design certificate if there was an engineer they didn't know –

Q. Yes.

A. – and they weren't sure of his work they may reject the acceptance of a design certificate and ask for calculations.

Q. I see. So the way in which the Christchurch City Council was dealing with permit applications in the 80s was, do I take it, significantly affected by who it was that was the design engineer on the project?

A. I think they came to a general realisation of the strength and weaknesses of various design officers.

Q. Yes.

A. And so they sort of modified their attitude towards design certificates on that basis.

Q. And so more willing to accept the assurances of somebody that they regarded as competent than someone they didn't know of?

A. Not only someone that’s competent but the strength in the design office to, and sort of checking systems that would more likely have sort of produced a result which would be compliant with the, with the bylaws.

Q. All right and for that category that you've just described more likely to accept assurances from that design engineer that the building was one that was just fine, or however you would put it?
A. Yes.
Q. The environment you've been describing about the 1980s and the expectations of Council reviewing engineers, is it any different today in your experience in the way in which these things are being done within the Christchurch City Council?
A. It's very different since 2004 Building Act, that is a building like the CTV building would be accepted by the Council only when all the documents are complete. You wouldn't get it across the counter unless you had the foundation report, all the drawings complete and designed, specification complete and a whole lot of other issues and then for a building like CTV it would, if it was a more pioneering building like that where the shear core is outside the plate of the main floor slab then they would put it out for peer review to one of perhaps three large design firms, design consultancies.
Q. Now just on this question of pioneering, and I think it's the second time you've used that term, is it the fact that the north core is outside the main frame of the building, is that what causes you describe it as a pioneering design?
A. Yes up to that time most buildings and the, the Landsborough House which has been cited as forerunner of this building –
Q. Yes.
A. – the shear core was within the building foot plate.
Q. If you were designing a building like that do I take it that you would take a great deal of extra care because of it being sited outside the main frame of the building?
A. Yes because it was a newer concept and probably architect driven to give better spaces inside the building you would tend to say, well this is a bit different to what we've done before so what is different and let's recognise those differences.

Q. Yes. Finally I just wanted to ask you a few more questions about the supervision and mentoring role that you've played within your current firm. I take it you've been engaged in that for many years have you?
A. I've been –
Q. Supervising and mentoring staff in the -
A. Yes as an older engineer you definitely do that and you move away from the coalface of design.
Q. And is that a structure that has been deliberately put in place by the firm that you're in, to use the older, more experienced engineers to mentor and supervise?
A. Well we're not a big firm. We're only six members. Three, we've always had three senior engineers. We've employed from time to time a junior engineer who one of the three partners, as we were, would have watched quite carefully what he was doing.

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Q. Yes, how would that, be a little bit more precise about that, how would that care and observation of the more junior member of staff how would that be done?
A. Well we've nearly always had the one office, an open plan office, and we would look at what he was doing and he would be able to converse with us. We've had a very open policy of people talking to us and so he would ask us questions and he wouldn't feel there's any barriers to his asking questions.
Q. Yes and so I take it it wouldn't just be a question of that junior member of staff approaching you for advice but you would be making sure you knew what they were doing at any given time.
A. Yes.

JUSTICE COOPER ADDRESSES MR MILLS

CROSS-EXAMINATION CONTINUES: MR MILLS

Q. 1980s.

A. That would have been the case yes. We were three senior engineers at that stage, or experienced engineers. We took on a graduate. We would watch him and that sort of came from when I started engineering and Lyall Holmes used to sit down with me and go through line by line all my calculations.
Q. This is when you were at Holmes Wood.
A. Yes.

CROSS-EXAMINATION: MR KIRKLAND – NIL

CROSS-EXAMINATION: MR ELLIOTT

Q. Mr O'Loughlin my questions relate to just one paragraph of your brief, paragraph 49, and could I ask for that paragraph to be displayed please.
A. Sorry, yes. The shear stress….
Q. It's the paragraph beginning "I also note".
A. Yeah, this is where the 30,000 newtons.
Q. Correct. Now I note that Mr Harding is present and I'm just going to ask him to follow the questioning because you've made some comments about his calculations. I'm going to ask you a few questions about that and then we can ask Mr Harding to comment when he gives evidence shortly.

WITNESS REFERRED TO PARAGRAPH 49

Q. So you're just noting what you say is an error, which I'll bring up in a moment, with the calculation of shear stress using a figure of 30,000 newtons and you refer to what appears to be a mistranslation and you say that has the effect of increasing the load to be transferred in a significant underestimate of the required reinforcing and you say that that wouldn't have been readily apparent to a Council Reviewing Officer.

WITNESS REFERRED TO BUI.MAD249.0272.65

Q. This is the first of the two pages that you're referring to, this is S56, is that right?
A. Yes.

Q. With the heading Slab Diaphragm and Connection to Shear Walls and then if we look at the second page so you can identify it.

WITNESS REFERRED TO BUI.MAD249.0272.66

Q. And that's the second of the two pages that you're referring to, is that right?

A. Yeah that's what I'm referring to.

Q. You've been through the calculations.

A. I've read through them all, yes.

Q. And can you confirm that they are the only two pages that are there relating to slab diaphragm connection to shear walls?

A. Yes that's all I've seen.

Q. Now is it right that if you look at the bottom of the first page, the left-hand page there, we can enlarge this if you'd like to but you probably notice already, what Mr Harding seems to do first is ask how much force must the connections be capable of sustaining and the figure he arrives at is 501 kilonewtons, is that right?

A. That's right.

Q. And then we go to the next page, so we just need that second page now and at the top of the next page he apportions the 501 kilonewtons between the two side walls but uses a factor of 0.6 rather than 0.5 introducing some conservative –

A. Yeah I see that, that's how he arrived at the 300.

Q. So he arrives at 300 kilonewtons and that is for the load to be borne by the connections at line 1 and also by the connection at line 4/5.

A. Yes, 5 yeah and four.

Q. And we're talking here about the east-west direction.

A. That's right.

Q. So where he says for line 1 he uses that figure of 300 in that first line.

A. Yes.

Q. And so your point is that for line 1 he appears to have correctly transposed 300 kilonewtons in his calculation.

A. Yes what he's looked at, that 300 is what's attributed to line 1 and then he's said the building's 30 metres long therefore it's 10 kilonewtons a metre that has to be transferred over all of 30 metres.
Q. And then do you see we have line 4, referred to further down the page.
A. Yep.
Q. And the point you're making is he's put in 30,000 newtons.
A. Yes.
Q. And is it correct that the figure should be 300,000 newtons?
A. That's correct.
Q. So that's the error that you're referring to.
A. Yes that's right.
Q. So what he's done is to quote a force of 10 percent of what he actually intended to quote, is that right?
A. Sorry?
Q. He's quoted a force –
A. Yes.
Q. – a load, which that connection should have been capable of bearing, of 10 percent of what he had actually intended to apply.
A. That's right yes.
Q. So is the effect of that that on the face of it he's only designing the north core diaphragm connection to be capable of sustaining 10 percent of the forces which, by his calculations it should have been capable of sustaining?
A. Not really because he's then worked out the shear stress in the concrete on the basis of 30,000 newtons and he comes up with 0.17 newtons per millimetre squared and then he looks at what concrete can take and that's worked out a few lines above where it says BC=0.17 the square root of 25 and he comes up with a figure of 0.85. So when he worked out the shear stress from his calculation for line 4 and he came up with 0.17 newtons per millimetre squared that was actually about four times less or a quarter what the concrete could take.
Q. So his starting point was incorrect but he may have been compensated for that in terms of –

A. It was compensated but not totally.

Q. Just let me finish the question. He's compensated for that in terms of the capacity of the element he's then designed, is that what you're saying?

A. He's compensated for it in the fact that at 0.17 newtons per millimetre squared the concrete could have taken a lot more, as he's shown up above.

Q. Now you make the comment that this may not have been necessarily evident to a Council Reviewing Officer. I just invite you to assume that the Council Reviewing Officer had some particular interest in the connection to the north core. You've agreed that we have only two pages here relating to that area, that's right isn't it?

A. That's right.

Q. And this particular calculation relating to line 4 is only three or four lines long isn't it?

A. That's right.

Q. So really it wouldn't have been difficult at all for a Council Reviewing Officer to pick that mistake up?

A. If he went through the calculations line by line he would have picked it up but just looking at the page he may not have and I don't know who else picked that error up and I didn't see a mention of that in the Hyland Smith report for instance and then there was a review in 1990 which may or may not have picked that up.

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Q. And just one or two questions about that review of 1990 on the same point. Can I just show you a section from NZS4203:1984, ENG.STA.0018.53. Just highlight 3.4.6.3 please. Just read that to yourself please?

A. Yeah I see that.

Q. So in the CTV building the floors were acting as diaphragms distributing seismic forces to the shear walls, is that right?

A. Yes.

Q. And those similar forces would also have needed to have been distributed through the connections between the diaphragms and the walls?
A. That's right.
Q. And so is the effect of that particular clause that in relation to diaphragm connections 3.4.9 which is the parts and portions provisions of the code, set out the forces required?
A. Yes.
Q. Now Mr Harding, it appears, he can confirm this, used forces derived from the equivalent static method. Is that right, can you say? Or you don't know?
A. Well I haven't been through all the output of his ETABS analysis and I am not particularly conversant with ETABS.
Q. Well if you don't know –
A. So he could have – it could have been a combination of the both, it could have been a static analysis or it could have been –

JUSTICE COOPER:
Q. You were asked if you knew, just say whether you knew?
A. I don't know that he, how he derived that.

CROSS-EXAMINATION CONTINUES: MR ELLIOTT
Q. Thank you, we can ask him. But you have read Mr Banks' brief, haven't you?
A. No I haven't actually.
Q. I thought you said in your second brief that you had read it?
A. Oh...
Q. That is all right?
A. I am – this is on the drag bars?
Q. Yes?
A. Yes, yes, from Structex.
Q. Well I will just show you one page from Mr Banks' calculations of the same area. It is BUI.MAD249.0130.22?

A. Yes.

Q. It is different to mine Your Honour. I have the same number but it is different. That is it now, thank you, so it is - for some reason it is a different number Your Honour BUI.MAD249.0130.17 and Mr Banks I think used the parts and portions provision of the code?

A. Yes.

Q. And the figure he arrives at is 1241 kilonewtons, do you see that? Up the top there?

A. Yes.

Q. So – and then what he does is he apportions it to line, wall 4, and arrives at a figure at 724 kilonewtons, do you see that?

A. Yes, that is right.

Q. So it appears that Mr Harding reached a figure of 300 kilonewtons in relation to the force that the connections at that north end must be capable of sustaining while on the other hand Mr Banks has reached a figure of 724 kilonewtons -

A. Yes.

Q. – for the same area. So does that indicate to us that apart from just dropping a zero Mr Harding may also have failed to use the right section and the right forces to calculate the appropriate capacity of the diaphragm connections at the north core?

A. Well I have seen other evidence where the figure is higher than 724 in some of the other technical evidence given, so it does, to answer your question if that 724 kilonewtons is correct then the answer is, obviously is that it hasn't, Mr Harding is under estimated or under calculated what that load is. But from the point of view of the Council reviewing engineer he tends to look at the calculations and he wouldn't probably have come up with a number like that or any other number.

Q. It would have been possible for a Council reviewing officer to either identify from the calculations or check with Mr Harding, "Have you used the forces set out in the parts and portions section or have you used – "

A. Yeah –

Q. – or have you used the equivalent static method?

A. That is correct, if he was worried about that area which I think he was then he could have asked that question.

Q. If Mr Harding has used forces derived from the equivalent static method and not the parts and portions, doesn't it follow that it is possible that he has in fact under calculated the applicable forces for the diaphragm connections at every level of the building, north and south?

A. Well he has used the same figure for the north and the south in that particular page 59 of his calculate – of Mr Harding's calculations, so that would follow.

Q. Or to put it another way, the forces which he has used to determine to the capacity of the connections at north and south wall at every level may be less than the minimum required by the code, that is right is it?

A. If, because the parts and portions are part of the requirement for compliance then that would be a correct statement.

RE-EXAMINATION: MR REID - NIL

QUESTIONS FROM COMMISSIONER FENWICK:

Q. You were talking about the shear actions between, the walls between lines 4 and 5 in that top area between the wall on line C and the wall on line CD?

A. Yes.

Q. And you were pointing out that this has some flexibility in that zone and therefore it has an influence on the stiffness, the structure and the torsional restraint, is that correct?

A. Yes.

Q. Did you look at the stresses in the floor near that zone, not in the actual wall but just outside the zone contained in the wall, did you make any estimate of what these –

A. No I looked at the stresses on that page S59 and what stress value was in the 150 slab –

Q. Yes, so you have got the shear stress of 1.78 should be two megaPascals walls but –

A. Yes south of line 4.
Q. Yes but you didn't look at the flexural actions there?
A. No I didn't I thought well that is a complete line of 30 metres.
Q. Yes.
A. So it shouldn't be, in terms of shear too high.
Q. Yes.
A. But in terms of the mechanism between line C and CD which is the amenities area where the toilets were.
Q. Yes.
A. For that to work as a diaphragm you develop tension and compression forces each side of it.
Q. Yes?
A. And that is where you would have to look at the transfer to that floor diaphragm.
Q. Yes or looks not very easy to get the forces through doesn't it in that case, yes. The other thing I just wondered about was, would some checking engineer or someone, when they are looking at columns six, 20 millimetre bars and a 12 – a 6 millimetre spiral going around it, 350 link, would they be expected to say, oh, that won't stay in place, it's
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going to be too flexible. Would that be something which would raise alarms that when they put it in the column they couldn't actually (inaudible 14:55:15) because it would be too floppy? Would that be something which would strike you as a potential problem?
A. In terms of it being, the, compared with individual links that you've got this very thin spiral (inaudible 14:55:34) –
Q. Very small fine steel, yes.
A. – and that the column could have a sideways bend in it.
Q. That's right.
A. It wouldn't be as rigid as if it had individual links tied because when you put individual links in you tend to tie each link to the vertical reinforcing.
Q. Yes.

A. When you put a spiral in you don't get these four corners or six bars that you tie it to.

Q. Yes.

A. So there's not going to be as many ties from the spiral to the vertical reinforcing as there would be with straight links.

Q. Probably if you doing the links -

A. And so you're right it would be more flexible.

Q. I was going to say, probably doing the link you'd probably use a 10 millimetres wouldn't you rather than the six.

A. The links would be 10 millimetres, yes.

Q. 10 millimetre.

A. 10 or 12.

Q. That would stiffen it again wouldn't it in (inaudible 14:56:26).

A. (inaudible 14:56:26).

Q. Look thank you very much.

QUESTIONS FROM COMMISSIONER CARTER:

Q. Just one question with respect to the discipline in seeing that drawings were properly checked. In the 80s I presume it was the practice in your office to have a panel on the drawings which showed whether the drawing was checked, who had prepared it and whether they'd, had it been checked. Was that normal practice?

A. Yes usually the draftsman's name was on the drawing and then there was a checked by and there was a signature there and that was the normal practice in the 80s.

Q. Would the Council take much notice of that? Would they require that to be filled in before they would accept a drawing?

A. There's been occasions where when you provide engineering advice to an architect and he does the drawings that they ask that you sign each drawing and that's in the bylaw, but if the consultant provides the drawings and it's on the consultant's drawing, his name's on the, on the structural drawings then it wasn't always picked up and asked for.

QUESTIONS FROM JUSTICE COOPER:

Q. When you talked about the checking process in your office you said the draftsman prepared the plans and then it was checked by the engineer but was there any other checking process so that the work of the engineer was itself checked?

A. Yeah within, the structural design was usually done by more than one engineer for a project of this size. For a house or a small factory it may be one engineer did it by himself but for a building of the complexity of CTV and that bulk of building there would be two engineers involved who would bounce things off each other.

Q. And then would both sign the plans or would one sign them as the primary author and another as a checking person?

A. The person who was assigned to the design as the lead designer would sign the drawings.

Q. Right but before the drawings were submitted to the Council for a building consent there would have been some internal process in which the plans were checked?

A. They were checked within the office.

Q. I just want to explore briefly with you a question that was raised by Mr Mills and I'll just take as my starting point the proposition which I think your evidence is based on that there are some things that a Council reviewing officer can be expected to check and satisfy him or herself about and there are others of which that cannot be said. Now what's the basis upon which you categorise matters in either, or place matters in either category. Is it simply a question of complication or difficulty?

A. Well different checking engineers had their own pet hobbies if you like. Bryan Bluck was very strong about old brick buildings. He was interested in brick buildings and how they performed or didn't perform and so if he was checking something of masonry construction he would probably take a big interest in how the out-of-plane forces on brick walls were having their loads resisted. So, and that is just human nature that you, you get, a reviewing engineer has some things he looks at specifically and some things which he may not look at so specifically.

Q. But it's the duty of the Council isn't it to satisfy itself in 1986 terms that a proposal complied with its building bylaw?

A. Yes.

Q. That's the duty of the Council isn't it?

A. Yes.

Q. So it can't simply be a matter of the luck of the draw as to what reviewing officer you get. Isn't it the case that the Council is obliged to do what is necessary to satisfy itself that the building for which a permit is sought complies with the bylaw?

A. That would be their overall task. Within that bracket there would be some things they would look at more closely than others.

Q. Well in a circumstances where there are difficult aspects of the design Mr Mills asked you how does the Council, how would the Council satisfy itself on this compliance issue and the answer you gave was that design certificates were generally required from engineers from whom such a certificate would be regarded as worthwhile.

A. Yes.

Q. And in other cases calculations were sought. Is that right?

A. That's correct.

Q. And presumably the calculations would have been sought so that they could be checked. Is that right?

A. That's right.

Q. So it wouldn't just be a matter of the Council satisfying itself that the calculations had been done. The purpose would be to make sure that they had been reliably done. Is that right?

A. That's right but the checking engineer isn't working as a designer, he's working as a scrutineer to look that the proper processes have been followed through with the calculations as well as looking at details on the drawing.

Q. I accept unreservedly that the checking engineer is not a designer but his role is to ensure isn't it that the plans comply with the building bylaw?
A. That’s correct.
Q. And a building permit should not be issued unless that is the case. Is that right?
A. That’s right.
Q. Okay. Thank you.

QUESTIONS ARISING: MR PALMER

Q. I've got one sir which relates to the supervision issue. In 1986 in your office how many, what was the structure of the office? How many engineers did you have?
A. Five, I think five engineers, one a graduate and maybe four draftsmen and a secretary, around about, I can't be precise but that order.
Q. And of those, apart from the graduate were the other engineers like you, a similar level of experience?
A. Yes three were and one less so.
Q. And your firm is a three-principal firm. Is that right?
A. Three-principal firm.
Q. So the fourth engineer that you're referring to would be somebody who wasn’t a principal but nevertheless still with –
A. With say four years’ post-graduate experience.
Q. Four years post-graduate. Post-graduate or post-registration?
A. No, post-graduate.

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Q. And how many years post registration?
A. One.
Q. So you didn’t have any long term engineers perhaps with the same number of post registration years as Mr Harding in your office did you?
A. No.

RE-EXAMINATION: MR REID

Q. Mr O'Loughlin His Honour was questioning you before about the matters that, or the distinction between the matters which a reviewing engineer might be able to pick up as against those that he might not be able to pick up. Does the role of computer-aided design and the ETABS program play any role in that assessment?

A. I doubt if anyone in the Council had experience in the use of ETABS at that time and, therefore, they would have relied very much on the designer having completed a competent analysis.

QUESTIONS ARISING: MR MILLS - NIL

WITNESS EXCUSED

MR KIRKLAND CALLS
DAVID HARDING (SWORN)

MR KIRKLAND:
Before Mr Harding starts, Sir, at the end of re-examination, if any, he wishes to make personal statement to the Court or the Commission, if that forbearance would be granted to him I'd be grateful.

JUSTICE COOPER TO MR KIRKLAND:
Q. Right. Is there a statement of evidence?
A. There's not, Sir, there's only two paragraphs in Mr Harding's first brief on the Code and we've all been a bit rushed today and I just wondered if he could read those two paragraphs again just to get back into the flow of matters.
Q. Oh, that's right he read them on the basis that he'd be cross-examined later, didn't he.
A. Exactly, Sir.
Q. Well just let me get that in front of me if you wouldn't mind.
A. They're at pages 21 and 22, Sir.

EXAMINATION: MR KIRKLAND
Q. Mr Harding do you have your first brief in front of you?
A. Yes.
Q. If you could just read commencing at paragraph 38 please.
A. I agree with many of the recommendations in the reports.
Following observations of a number of buildings following the earthquake it appears that many buildings have experienced lateral sway which is somewhat larger than expected from the design calculations and computer analysis.
This may be partly due to assumptions made during the modelling of the building, such as the assumption that shear walls are rigidly fixed at the base; no allowance being made for foundation flexibility; the flexibility of the soil below the foundations and liquefiable soils; the degree of

cracking in the concrete; the state of the concrete and reinforcement following earlier earthquakes; and the reinforcement content of the walls and columns.

There was no provision for vertical acceleration on buildings in NZS4203:1984. Even the current Building Code, NZS1170.5:2004 Structural Design Actions, New Zealand, only requires that the vertical acceleration be assessed at 0.7 times the horizontal acceleration.

As previously noted the paper written by Bradley and Cubrinovsky titled 'Near Source Strong Ground Motions Observed in the 22 February 2011 Christchurch Earthquake' reports on the observed vertical accelerations. The vertical accelerations were three times the horizontal accelerations at the Pages Road pumping station. That report goes further to report, relative to both the 22 February Christchurch earthquake, and the 4 September 2010 Darfield earthquakes:

"it can be clearly seen that vertical to horizontal ratios above one are frequently observed for distances up to 40 kms in both these events (as well as other historical earthquakes worldwide) and hence the code prescription of 0.7 is, without question, significantly unconservative".

It would be my hope that, due to the excessive lateral movement which takes place in an earthquake that the present code be amended to also require that all columns be detailed for ductility, irrespective of the calculated lateral sway of the structure.

Q. Mr Harding, as I recall your earlier evidence was that the earlier drawings, the architectural drawings, were given to you by Dr Reay and the design was a shear wall stabilised gravity frame. Is that what faced you?

A. Yes.

Q. And it's my further understanding that the beam columns were to be part of a gravity frame and, therefore, there was no requirement to design for ductility. Is that your understanding?

A. That was my understanding.

WITNESS REFERRED TO WIT.JACOBS.0001.17 AND 18

Q. Mr Harding you have read the brief of Dr Jacobs?

A. Yes.
Q. And at paragraph 53 he summarised and makes a number of observations, in particular in non-compliance with the Code. You've read those sub-paragraphs A to F inclusive.
A. Yes.
Q. And you've given some thought to each of those paragraphs.
A. Yes.
Q. Paragraph A – your response to that Mr Harding please.
A. Well I think that in the Code it says you shall give consideration to it and you shall try to keep a building symmetrical but I think that there's no actual criteria as to how symmetrical the building has to be. It's an object to aim for and, as I understand it, there's no part of the Code that we don't comply with in that regard. I think the Code requires that the building shall be designed using a dynamic analysis rather than equivalent static method if the building is torsionally irregular and we did do that. So, as far as I'm aware, we complied with the requirements of the Code in that regard.
Q. Paragraph B.
A. It's not actually in front of me at the moment. The columns were not designed for ductile behaviour. That is true they weren't designed for ductile behaviour. They weren't intended to be part of the ductile lateral load resisting system. I guess the clauses in the Code which specify that they should have been, relating to it being a secondary frame and being beyond elastic, that may well certainly be the case, that it was beyond elastic, but I haven't done that calculation. There's a note in the Code, I think, regarding, which he refers to, saying that any element which is a danger to life – I sort of think that's a bit of an all encompassing one and I think it's probably a matter of judgement because you could argue that just about any component in a building is of risk to life if it fails. So there must be a point at which the engineer has to make a judgement and I think normally that ductility is applied to

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the elements of the building which are providing the lateral load resisting force rather than those which provide vertical load resistance.

Q. I'll come back to that, Mr Harding, I think it's paragraph 15, but just before I get to that and we deal with or address subparagraph C and I'll read paragraph 33 of Dr Jacobs:

"My interpretation of this clause is that if a member is checked for its ability to accept deflections derived from V delta and the member is still within the elastic range then there is no need for ductile behaviour to be provided for that element."

Were you of the view that the members within this building were within the elastic range Mr Harding?

A. Yes, I'd taken the view that if the columns were designed as pin ended then the remainder of the structure would, yeah, would effectively be within the elastic range. But I do agree that I did not do that calculation.

Q. And if I could now have .6 brought up please?

WITNESS REFERRED TO SLIDE

Q. Paragraph 15 of Professor Jacob's evidence, that's the reference to the code in terms of 'in the case of failure are a risk to life' that you referred to Mr Harding?

A. Yes.

Q. Yes. So is that saying to you that, and to a layman, that all buildings should be designed for ductility?

A. Well you could, you could read that, that way yes.

Q. Well how else could you read it?

A. Well as I say, I think they rely upon the engineer to have some degree of judgement, but if that was taken literally then there's no judgement required. Every element in the building should possess ductility.

Q. So even Professor Jacobs, there seems to be not confusion but a difference in what he's saying in paragraph 13 and paragraph 15 that needs to be reconciled?

A. Yes.

Q. You can go back to slide .18 and paragraph C if you could comment on that paragraph please?

WITNESS REFERRED TO SLIDE

A. Yeah well that, I guess, comes back to what I was saying earlier about the various vagaries in the assumptions that we make as engineers in terms of the stiffness of properties of elements and the effect of the amount of reinforcement in predicting the stiffness of the elements. But I think that's a fairly general statement and if it's saying that we can't believe the results we get from our calculations then I wonder what we are to believe.

Q. Paragraph, subparagraph D?

A. I agree that the columns were heavily loaded. These columns were basically designed solely for vertical gravity loadings. If they'd been part of a ductile frame they would've had, they would've been much bigger. I mean if the beams were 500 deep and 500 wide the column would probably end up being about 700 by 600 if it was going to be part of a ductile frame. So the fact that they were just designed as pin ended columns with a gravity load, they were designed for that load without anything else, and consequently they were, they could be viewed as heavily loaded. I think it's valid that John Henry pointed out that he was concerned about some of the columns in his other building and he actually doubled the load on them, and thereby ended up with bigger columns and I think that's a matter of experience. He's basically made the assessment that if you have vertical acceleration you're going to have a bigger load on the column and you need a bigger column. So, yes, in this situation I accept that they were heavily loaded but I don't agree that because they were heavily loaded they were within the load specified in the code.

Q. John Henry's other building was Landsborough House is that correct?

A. Yes.

Q. Subparagraph E?

A. Yeah well I think the, I accept that it is less than the shear reinforcement in the code, but I think the code statement does say, "Where shear reinforcement is required then the minimum shall be…" and I think that's the, it's actually quoted in Mr Latham's evidence where he says it. And as I say, at the time the contention was that if the columns were designed to be pin ended, the shear in them was actually theoretically zero and I think my view was that shear reinforcement wasn't required.

Q. And finally, F, Mr Harding?

A. I accept that the connection on the north side was limited. I attend to the question made before about the size of the load it was designed to take but yes it was situated outside the main floor plate which limited the ability to connect to that wall, so I agree with that statement that it's limited.

Q. A difficult question Mr Harding but I have to pose it to you. Given hindsight, what would you have done differently?

A. If I was to design that building today?

Q. Yes, or even what would've you done back in 1986 with the tools that you had?

A. So do you mean knowing now what I know, if I knew then what I know now?

Q. Knowing now what you know?

A. Well I mean I would've been designing for a vertical acceleration for much heavier loads in the columns than we had before for sure. And I would be not using mesh in the floor. And I'd be using a much lighter floor, but there's a number of other improvements you could make as well I'm sure.

Q. Would you detail the columns and the beam column joints for ductility?

A. Yeah I think the thing is about detailing for ductility as opposed to making it a ductile frame, you would put the ties at a closer spacing to confine the core of the column but as to what lateral load you design those columns for, I think you would need to, I guess with today's computer programmes you'd be able to model the building a lot better than it was possible to do it then and better define where the ductility was required. That was not really something which the ETABS program we were using at the time had provision for.

Q. The Holmes report refers to drag bars, Mr Harding. In hindsight would you insert a drag bar sufficient to develop the over strength bending moment capacity of the walls?
A. That's not what it was designed for, no. But that's certainly what you would do today, yes.
Q. Finally Mr Harding, there's just a matter that's been troubling me and I'm not clear on this point and what Dr Reay says in his evidence, because you say that he gave the drawings to you in the early days, is that correct?
A. Yes.
Q. He says, "I recall," this is paragraph 52 and I'll just read this part, "I recall at the stage that Mr Harding received the architect's drawings I asked him what the structural lateral load system was." My understanding, Mr Harding, is that it can either be a shear wall stabilised gravity frame or a ductile frame, am I correct on that?
A. They would be the main options, yes.
Q. He is saying that I asked him what it was, but you are saying that he told you what it was. Can we have that point clarified please? What did happen in your view?
A. Well, yeah when I was introduced to the job he gave me the drawings and he said, "It's going to be done the same way as Landsborough House," and gave me a copy of the calculations for Landsborough House and explained that that was how that building was done and that was how we were going to do this one. So that was the reason for giving me the calculations for that particular job. That was how that was done and that was the instruction to do it the same way.
Q. And the final matter that I just need clearing up, Mr Harding, is when you did the ETABS run and the inter-storey deflection was excessive and I think you tried to thicken the walls to make a correction. Then you say you went off and discussed the issue of a south shear wall with Dr Reay is that correct?
A. Yes.

Q. Was there a south shear wall in the drawings when you did the ETABS run?
A. Not for the original one, no. The original design didn't have that wall there as I recall.
Q. What did it have there?
A. I think it was just glass all the way around the outside of the building. There was still a stair there but no wall there.
Q. See, in cross-examination Mr Zarifeh to Dr Reay, he says as I understand the exchange that there was a conversation and it may have been for about 10 minutes. We need to clarify this point Mr Harding. Can you recall exactly what happened again please?
A. I can't recall exactly what happened but I certainly remember going to him and saying that we need another wall and I suggested that's where we put it. That was a meeting that we had in the front office, the front room of the office.
Q. And the response from Dr Reay?
A. To, well he thought the architect may not like it because it wouldn't look the same as the Contours building, but as I understand it he was going to check with the architect. But I can't remember whether the response was immediate or how long it took to do that, but the answer was, "Yes we can put a wall there provided it's smaller than the length of the stair," so having made that agreement I re-ran the program using that wall and reconfigured the stair so that the landing for the stair was in the middle of the wall instead of off to one side like it had been.

HEARING ADJOURNS: 3.27 PM

HEARING RESUMES: 3.46 PM

CROSS-EXAMINATION: MR REID - NIL

CROSS-EXAMINATION: MR PALMER – NIL

CROSS-EXAMINATION: MR MILLS – NIL

MR ELLIOTT:
Your Honour the purpose of my questioning of Mr Harding here and of Dr Reay when he gives evidence shortly is just to work through each of the areas of alleged non-compliance with code and best practice and to give them both an opportunity to state their position where they haven't done so already and to make any comments they wish. For that purpose I have prepared a document entitled, "Schedule of non-compliance." This is a document I have prepared, it is not evidence.

JUSTICE COOPER:
It's an aide memoir.

MR ELLIOTT:
It is an aide memoir, Your Honour, and what I have sought to do here is just to set out each category in the left-hand column as far as I can see the relevant provision of the applicable code that is being referred to.

JUSTICE COOPER:
Yes.

MR ELLIOTT:
The alleged non-compliance with the particular provision and then I have summarised or referred to some of the evidence although I hasten to add I haven't referred to every piece of evidence and the evidence is that which tends to support the allegation for the purpose of giving Mr Harding and Dr Reay a chance to comment.

JUSTICE COOPER:
Yes, thank you very much I am sure that would be very helpful. I will just read into the record the number that has been given to this, BUI.MAD249.0588.1.

MR ELLIOTT:

And Your Honour a copy has been given to Dr Reay who I think is in the Commission at the moment so that he can also follow along and he will be in a position hopefully to comment after Mr Harding has given evidence.

CROSS-EXAMINATION: MR ELLIOTT

Q. Mr Harding, I am going to just work through this schedule with you. I am also going to ask you just to clarify a couple of questions about your calculations which have emerged in the course of evidence. As I work through this schedule I am not necessarily going to seek to debate things with you but to provide you an opportunity to comment and to state your position. Can I just confirm that you have in front of you, in case you need it two green folders on your right, do you agree that one of those folders has copies of the relevant codes if you would like to refer to them?
A. Yep.
Q. And Bylaw 105, that is the first tab.
A. That is number 5 is it?
Q. Tab 1 I think you will find?
A. Yep.
Q. And you also have available to you there in the other green folder a copy of your calculations which you will find in the first three tabs in case you would like to refer to those, you agree?
A. Yeah.
Q. Do you have the drawings there as well in case you would like to refer to those, do you agree?
A. Yes.
Q. And you have in front of you a copy of a document headed, "Schedule of non-compliance."?
A. Yep.
Q. Now the first item relates to symmetry and I think you have already commented on this matter with Mr Kirkland just now, is that right?

A. Yes.

Q. Just one point of clarification. Is it your position that the walls were symmetrical in the east-west direction or not?

A. No they weren't symmetrical in the east-west direction.

Q. They were not?

A. No.

JUSTICE COOPER ADDRESSES THE WITNESS – MICROPHONE

JUSTICE COOPER:

Just say what you said again.

CROSS-EXAMINATION CONTINUES: MR ELLIOTT

Q. The question was whether you agreed the walls were not symmetrical in the east-west direction?

A. Yes I agree the walls were not symmetrical in the east-west direction.

Q. Now clause 3.1.1 of NZS4203 says, "The main elements of a building that resist seismic forces shall as nearly as is practicable be located symmetrically about the centre of mass of the building."?

A. Yep.

Q. So are you saying that the walls were not located symmetrically about the centre of mass of the building?

A. No I am saying that that is as far as practical. That is an optimum to be striven for as an engineer if you can, but it is not to say that it is a code requirement that they shall be symmetrical.

Q. My question was just was, is it your position that they were or were not located symmetrically about the centre of mass?

A. No they were not –

Q. They were not?

A. – in the east-west direction they were not.

Q. And so is it your position that there was some element of impracticability that prevented that from taking place?

A. Yes.

Q. And what was that?
A. Well the architectural requirements for the location of the walls wouldn't have allowed a shear wall the same as the one on the north side to be located on the south.
Q. So there were no engineering matters that made it impracticable; they were architectural matters that made it impracticable?
A. Ah, well yes the engineer doesn't usually have the right to put that many walls in just 'cos he wants them, that is correct.
Q. Now the next comment is the centre of stiffness of the designated primary seismic resisting elements were significantly eccentric to the centre of mass. What do you say about that?
A. Yeah that is a consequence of the walls not being symmetrical. As I say I don't believe it is a non-compliance, it is basically, if it's significantly eccentric the code requires that you should not use the static, equivalent static method but that you should use a dynamic method so if you do that you have complied with the code.
Q. Thank you. Over the page, shear reinforcing of columns?
A. Yeah as I say I think you will find that it says, shear reinforcement where it is required shall be no less than, and I think I took the view that it wasn't required but I accept that if you were to put it in then you would need more reinforcing than what we have.
Q. I see, so if the allegation is requirements for minimum spacing, there were requirements for minimum spacing of the spiral reinforcing?
A. Yes I think what I am saying is that the code says, "Where it is I required then the minimum shall be," and I think I was suggesting that it wasn't required.
Q. So you do or do not accept an area of non-compliance there?
A. I don't accept it.
Q. Thank you. Anchorage of spirals on columns. Now, we can show you this clause if you like but –
A. Well that is a standard detail though. That is not something that you would necessarily show on the drawings, that's a – it is covered by the New Zealand Standard on forming reinforcement. It is the same with

any hooks or ties or returns, so, I mean it should have been provided at the end of the tie for sure but I don't see any reason to believe that it wasn't.

Q. So Mr Smith gave evidence that he saw no indication in the drawings of anchorage satisfying that clause, is it your position there was no indication –

A. It is a standard detail, it is not something that you need detail. At the end of the spiral you put a return into the body of the column, that is just a standard detail. So I see no reason to believe that it didn't, wasn't built without that hook.

Q. When you say it is a standard detail do you mean you are relying upon the contractors to put it in even though it is not in the drawings?

A. Oh, no I am relying upon the detailers of the reinforcing bars to comply with the code of practice for reinforcing bars it is not –

COMMISSIONER FENWICK:

Q. Would that have been covered by the specification –

A. It is covered by the New Zealand Standard – I think our specification would refer to the New Zealand Standard, yes Sir.

Q. Thank you, so it is covered.

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CROSS-EXAMINATION CONTINUES: MR ELLIOTT

Q. Are you aware that the foreman of the job has given evidence that not in 40 years has he kept a copy of the relevant standard on site?

A. I wasn't aware of that but normally the reinforcement is provided by a reinforcing steel detailer, not necessarily by the foreman. So the foreman just assembles it. He doesn't bend the reinforcement on site.

Q. Thank you. The next issue is the adequacy of R6 at 250 millimetre spirals in the region of the cranked splices. Are you aware of the particular region that this is referring to? I can show you on the plan if you like but there's a region of cranked splices in the columns.

A. Okay well I'd have to study that but if that's, on face value it would appear correct that that may not comply.
Q. What do you need to see to verify that? Would you like to see the drawings?
A. I'd have to look at 3101. Special Details for Columns and Piers.
Q. All right. Well let's look at ENG.STA.0016.41, clause 5.3.27.1. That'll be enlarged for you.
A. Yes I accept that.
Q. So the alleged non-compliance – spirals of R6 at 250 insufficient to meet that requirement and you accept that?
A. Yes.
Q. Over the page please. There has been evidence of the required level of ductility for the columns and various reasons, some have said the columns should have been designed for ductility and I'm just going to take you through the various grounds that have been put for that statement. The first is in relation to capacity design. So I'm moving down to the next row of this table, we're on page 3, and the particular point is where the capacity design required the columns be designed for ductility. Firstly, do you accept that capacity design applied to the design of this building?
A. I don't accept that it applied to the columns and beams. It wasn't a ductile frame building so that you weren't designing the beam column joints for capacity design. The capacity design applied to the lateral load resisting elements which was the shear wall.
Q. I see. Well in fairness we'd better show you the clause. ENG.STA.0016.24, 3.5.1.1(a). Just read that to yourself please.
A. (witness reads) Yeah that's, again that's if you're designing the building as a flexural ductile frame but as I say this building isn't designed as a flexural ductile frame building. It's designed as a shear wall building. So I don't believe that that clause applies.
Q. Let's enlarge (a), (b) and (c). They're the three clauses available for the method of designing. Do you agree?
A. Certainly.

Q. So firstly Mr Smith has given evidence that clause (a) was applicable. Your position on that is?
A. Is that the ductile structure is the shear wall not the beam column frame. The ductile structure which provides lateral load resistance to the building is the shear wall.
Q. So do you accept that sub-clause (a) applies but you're defining ductile structure as the shear walls not the structure as a whole?
A. Not the beam column frame, yes.
Q. So you're not asserting that (b) or (c) would be applicable?
A. No. Well I'd have to check whether we've designed it as limited ductile or ductile. I believe we designed it as ductile.
Q. Well let's just clarify that because it gives rise to something else Mr Smith referred to. If you look at BUI.MAD249.0272.1. This is page S1 of your calculations.
A. Yeah.
Q. Do you see you've used the structural type factor of S=1.0Z?
A. Yes.
Q. And that's consistent with a ductile structure?
A. Okay, that's right.
Q. Do you agree?
A. Yes.
Q. While we're on this topic I'll show you another page from your calculations which is 0272.28 and do you see down the bottom, second last line S=0.8Z?
A. Yes.
Q. So that appears to be a different structural type factor doesn't it?
A. Yeah that may have been used for –
Q. If you go to the previous page, if you look at page S27 of your calculations I think you'll find that the S=0.8Z relates to your calculations for the south coupled shear wall.
A. Okay.
Q. Are you looking for table 5 with the structural type factors?
A. Yeah.

Q. It's ENG.STA.0018.47. It's page 42 of NZS 4203.
A. Yes so for the ductile coupled shear wall I've used a factor of 0.8 and for the ordinary shear walls I've used a factor of 1. Is that what you're saying?
Q. That's what I was asking you, and you're confirming.
A. I believe that's the case by the look of what you've shown me.
Q. Mr Smith made some comments about how that difference between the structural type factors apparently for the south wall and the north core could lead to difficulties, in particular the south wall yielding and performing in a plastic way while the north core continues to perform elastically which may then affect inter-storey drift levels. What do you say about that?
A. Yes I think that's a factor that you'd expect to happen, that the south wall would yield before the north wall. I accept that.
Q. With the consequential impact on inter-storey drift?
A. Well as I say the, I believe that south shear wall behaved okay. There was no, if, if you were going to get excessive inter-storey drift the, the wall would start degrading, the coupling beams in that wall would

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have had plastic cracking in them. I don't believe that did occur so I don't accept that there was excessive lateral drift caused by that wall.
Q. Professor Mander made some comments about the implications of capacity design being applicable and I'm going to show you a section from his evidence.

WITNESS REFERRED TO TRANS.201020724.101

Q. So if we can firstly have line 26 to the bottom highlighted please. Firstly, just read that to yourself please, Mr Harding. And the next page, if we could highlight down to line 16 please. You've read that?
A. Yep.
Q. Do you accept that evidence?
A. No, I, as I say I don't believe that the, as I said before, the ductile action, the capacity design, is in the shear walls not in the columns and beams.
Q. Going over the page to page 4 now of the schedule in front of you.

A. Yeah.

Q. The next set of evidence that we've had is that the failure of the columns was a risk to life with the consequence that the should have been designed to possess ductility. I think you've already answered this to some extent but I just want to ask you a quick question or two about that and I'll show you a page from your calculations.

WITNESS REFERRED TO BUI.MAD249.0273.44

Q. Do you agree this is the page from your G41A from your calculations relating to the calculation of transverse reinforcement in the columns?

A. Yeah.

Q. I'll just wait while you get that in front of you.

A. Yeah.

Q. The nomination of 41A as the number suggests that it was prepared and inserted after completion of the remainder of the calculations. Is that the case do you think?

A. Well that is normally why you put an A on yes.

Q. And what that calculation demonstrates is that you've concluded with the non-seismic loading, sorry non-seismic provisions of the Code. You arrive at R6 at 250 transverse reinforcement which is what was ultimately adopted.

A. Yes.

Q. Now you've also calculated the transverse reinforcement applicable applying the seismic provisions of the Code haven't you?

A. Yes.

Q. When you were doing this work were you aware of Bylaw 105?

A. That's, possibly not.

WITNESS REFERRED TO 11.1.5D - ENG.CCC.0044A.86

JUSTICE COOPER:

Perhaps you'd better tell Mr Harding what Bylaw 105 was, in fairness.

MR ELLIOTT:

Yes well I intend to.

CROSS-EXAMINATION CONTINUES: MR ELLIOTT

Q. So Mr Harding, firstly, you have a copy of Bylaw 105 in front of you. This is the Bylaw applicable as adopted by the Christchurch City Council.
A. Right.
Q. Applicable to the design of buildings at the time the CTV building was designed.
A. Yeah.
Q. And there was a particular clause within it – 11.1.5(d) – which I'll ask to be enlarged –
"In events that seldom occur, such as major earthquakes and extreme winds collapse and irreparable damage shall be avoided and the probability of injury to or loss of life of people in and around the building shall be minimised".
A. Yes.
Q. We should also point out to you 11.1.6 which indicates that structural design and design loadings complying with NZS4203 shall be approved as complying with the requirements of the previous clause. Firstly, now that you've seen that in a bit more detail were you aware of Bylaw 105 and in particular clause 11.1.5 at the time you designed the CTV building?
A. Yes I'm aware of what that says and what 11.1.6 says.

JUSTICE COOPER:

Q. You've just been taken to those provisions but the question was whether you were aware of them at the time that you designed the building?
A. Well I can't say what I was aware of 26 years ago, I'm sorry, I'm aware of it but whether I was aware of it then I can't answer you that.

CROSS-EXAMINATION CONTINUES: MR ELLIOTT

Q. Do you recall that when you designed the building you had in your mind as an objective that collapse shall be avoided and the probability of

injury to or loss of life of people in and around the building shall be minimised?
A. Absolutely.
Q. I'm going to put to you if you had, as you did, two options in front of you, as per your calculations of R6 at 250 millimetres or R6 at I think it was 80 or 100 millimetres, is that right?
A. I believe so I haven't got that page ahead of me now.
Q. Better go back to it if I'm going to ask the question so it's –
A. Yes I see that there, yes.
Q. Just go back to that page G41 of the calculations.

WITNESS REFERRED TO 0273.44

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Q. Mr Harding these calculations indicate that the options were on the one hand R6 at 250 spirals using non-seismic provisions or on the other hand either R10 at 100 or R6 at 40 using seismic provisions. Do you agree?
A. Yes that's the detailing for ductility.
Q. So what I'm putting to you is that if your purpose was to satisfy the objective in relation to avoidance of collapse and minimising risk to life safety the only choice you could have made of those two is the seismic detailing. Do you accept that?
A. No I don't. I think that what the bylaw says is that that objective is deemed to be satisfied if you comply with NZS 4203 and that, that was what we were aiming to do. That was the means of compliance to achieve the requirements of the bylaw.

JUSTICE COOPER:

Q. You're saying what "we" were trying to do. What do you mean by that? Who's "we"?
A. As engineers the, you read the means of compliance with that City Council bylaw –
Q. Yes.

A. – is NZS 4203. So as engineers that's the code which we, we use in order to comply with that part of the bylaw.

Q. All right.

CROSS-EXAMINATION CONTINUES: MR ELLIOTT

Q. Mr Harding next is this question of classification of the columns as secondary elements and the applicable clause here that I think you've relied upon is 3.5.14.3 of NZS 3101. We'll bring that up in front of you. ENG.STA.0016.28 and if the words on the bottom left can be highlighted please. Firstly, secondary elements are those which do not form part of the primary seismic force resisting system. Now there's been some evidence that one can draw a distinction between the seismic force resisting system, namely, those parts of the building which will actually be exposed to earthquake loads on the one hand and on the other hand those parts of the system which an engineer will designate or assume will need to bear loads for design purposes.

A. Yes.

Q. Do you understand that distinction?

A. Certainly.

Q. So what I'm putting to you firstly is that the primary seismic force resisting system there refers to the actual force resisting system which was called upon to bear earthquake loads which would have included the columns.

A. No.

Q. Do you accept that?

A. No I don't accept that. It was excluding the columns. The lateral force resisting system was the walls. There was no reliance on the columns made in resisting lateral forces.

Q. And if we just look at the top of the next column please. We have, the sentence goes on to say, "or are assumed not to form such a part and are therefore not necessary for the survival of the building as a whole under seismically induced lateral loading". So I'm suggesting to you that the columns were necessary for the survival of the building as a whole under seismically induced lateral loadings and for that reason as well could not have been treated, defined by you as secondary elements. Do you accept that?

A. Well secondary elements are those which do not form part or are assumed not to form such a part. So I mean they, they do not form part of the primary force resisting system. Therefore they are a secondary element.

Q. And do you accept, I suspect you do not, columns were necessary for the survival of the building as a whole?

A. Well I said it comes back to your definition of which elements of the building are a risk to life in the event of failure and I have difficulty thinking of any element in a building which couldn't be conceived as a risk to life. I mean it's that judgment as to which elements you believe need to possess ductility and which ones you don't. I mean if you're to read that literally you could say every element in any building needs to possess ductility and I don't think that's the intention of the code.

Q. Every structural element of every building.

A. Well any element could be called a structural element. Every element in the building has, you know, has an element of structure to it.

Q. Shouldn't an engineer who's concerned about safety say, well, I will design all structural elements for ductility?

A. Well as I say that's impractical to do that. You could call every element in a building a structural element if you wanted to take it to an extreme. I think the, the comment that is made to try and clarify that for the purposes of engineers is, is to explain to you which elements they're talking about and when you talk about, it's that same conflict that was in Jacobs' evidence. In some instance it says that the cover-all, that all elements which are at risk, you know, pose a risk to life should be designed for ductility and then later it, it tells you that if the columns can be shown to remain elastic then you need not detail them for ductility, and those same, you'll find that kind of, you need to make a call as to whether the element is critical in terms of supporting lateral loads to the building in an earthquake and I, I've mentioned this before regarding vertical acceleration. We generally seem to be of the view in terms of design and certainly in terms of NZS 4203 that the earthquake loads that are specified are lateral loads, horizontal loads and they are typically the ones which you, you provide ductility when resisting those lateral loads, the object being that you can absorb that lateral load energy in an earthquake by having plastic action occur. It's not intended to apply to vertical loads, at this stage.

Q. Are you aware that the definition of primary elements in NZS 4203:1984 included beams and columns?

A. If you are designing it as a ductile frame they would include beams and columns. That's the choice you make at the beginning, whether you're going to make the beams and columns part of your lateral load resisting system or not and if you choose not to, if you make the, resist all of the horizontal loads with your shear walls then those are the ductile load-resisting elements which you design for ductility. Any other element is not.

Q. Thank you. Now the fourth reason which has emerged as a ground for columns being required to be detailed for ductility is the next point on page 5 of the schedule. "If the columns were legitimately to be treated as secondary elements even then some have said the drift limits were exceeded which therefore meant the columns should have been designed for ductility." Now you're aware that evidence has emerged in the Hyland Smith report aren't you?

A. I have seen the Hyland Smith report.

Q. And do you accept that evidence?

A. Well there's some evidence which says that the columns remained elastic and some evidence which says that they were plastic. If you accept that the columns remained elastic then they need not have been designed for ductility.

Q. I think you've said this already but you didn't do any calculation of V Delta did you?

A. No the, the computer program we were using at the time, if we had put all the columns and beams and everything in the, in the input I don't think it would have dealt with it. So we were simplifying the structure for purposes of getting an output from the computer. So we didn't include the columns and beams in that analysis. Therefore we weren't able to get a calculation as to what the bending moments in them were. But as I said the, the assumption which was made was that they were secondary elements that if – they weren't required to be detailed for ductility. And I'm just stating the situation that if they were, could be shown to remain elastic, then that would not still be required.

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Q. You used the word, “we,” a moment ago in relation to what was being done. You are referring there to yourself and Dr Reay?
A. No I am sorry how did I use the word, “we,”?
Q. We were talking about the V delta?
A. Yes, I didn't do that calculation at the time I designed it.
Q. Now you have made some comments about pin ended, the columns being pin ended?
A. Yeah.
Q. There has been some evidence on that which you need to have the chance to comment on. This is from Mr Smith at TRANS.20120809.95.
A. Sorry are we on page 6 now?
Q. Well I am still on page 5. This doesn’t appear in my schedule I am just dealing with it at this point because it seems relevant?
A. Right.
Q. So this will come up on the screen in front of you.

JUSTICE COOPER:
Should we add this in.

MR ELLIOTT:
Yes thank you Your Honour. TRANS.201208.09.95.

JUSTICE COOPER:
TRANS.20120809.95.

CROSS-EXAMINATION CONTINUES: MR ELLIOTT

Q. Page 95 of the transcript. Starting at line 18 so if line 18 down to the bottom could be enlarged please. This is some evidence given by Mr Ashley Smith. He is referring to a diagram prepared by Commissioner Fenwick which I can bring up for you to look at in a moment.

A. I am familiar with that.

Q. Oh, you are familiar with it, good. Well if you just read to yourself that page and there are a couple of lines on the next page as well which I will refer you to.

A. Yes.

Q. And then the next page please, just down to line 4 thank you.

JUSTICE COOPER:

Sorry whose evidence is this.

MR ELLIOTT:

Ashley Smith Your Honour.

CROSS-EXAMINATION CONTINUES: MR ELLIOTT

Q. Have you seen that –

A. Yes.

Q. It is only down to the first couple of lines Mr Harding. You don't need to worry about the next section.

A. Yep.

Q. So my question is, just do you wish to comment on the evidence Mr Smith has given there about whether the beam column joints were detailed as pin ended or not? Do you maintain your position that they were?

A. Yeah I am not – that was the intention of the design yes, but I mean the connections between the beams, you know between one beam and another is not pin ended there is continuity between one beam and another but I am suggesting that where the column joins the beam that the concept of the design, wish to design it is pin ended and my understanding was that if the columns were to remain elastic then that was acceptable.

Q. Do you accept that it would not have been the effect of the design that they would be pin ended, they would have in fact –

A. I agree that it wasn't detailed significantly to be pin ended, that the vertical reinforcement in the column did continue through the joint, yes.

Q. Turning to page 6 now of the schedule. The first issue there is beam column connections. Now, would you agree that if, I know that you don't accept this, but if the columns were required to be designed for ductility it would follow that the beam column connections should also have been designed using the seismic provisions of the code as well?

A. Well, I would have to think about that I mean, if, as I say when you say columns designed for ductility, whether that means that we are designing the frame as a ductile frame. If you were designing it as a ductile frame as I said earlier you'd actually have to have columns which were bigger than the beams, they'd be massively bigger columns. So I don't read it as being that you are trying to design it as a ductile frame I read that as being that you are trying to provide a degree of ductility to the column and that is by way of closing down the spacing of the spiral reinforcement to provide confinement of the core of the column and that's so that if you end up with a hinge in there at all, if it is in the column then you have got a degree of confinement to the joints so yeah, I think it's, you'd have to look a bit harder as to what you are trying to achieve there so I don't accept that on the face of it, no, without some further thought.

Q. The next issue is whether the minimum, that's non-seismic transverse reinforcement requirements the beam column connections were met, and if you just have a look at the points that have been inserted there. Clause 9.4.2, 9.4.5 and 9.4.6 are referred to. So you have those in front of you, page 67 of NZS3101?

A. Yeah well again I think you are implying that this designed as a ductile frame as if the ductile frame is what is providing the lateral load resistance to the building and that is not the case. That is not what we designing it for.

Q. Well just have a close look at the heading to that section, 9.4, which is in principles and requirements additional to 9.3 for joints, not designed for seismic loading –

A. Yeah.

Q. So we are not talking about seismic provisions here we are talking about non-seismic do you agree?

A. Yeah but for non-seismic loads, there is no shear force in that joint. It is a bending moment, you know if that beam column, it is a beam joint basically with a column below it so that the, if you are not talking – if you are talking non-seismic actions there is no load in the columns, there is no bending moment in the column.

Q. Well the effect of Mr Smith's evidence was that the transverse reinforcement of R6 at 250 was insufficient to meet the requirements of the clauses that I have referred you to, do you accept that?

A. No, no I don't believe that that is relevant that clause for the situation we are designing for.

Q. Thank you and over to page 7. Clause 9.4.8 of NZS3101 relates to confinement. This has been referred to earlier. You may have been here. The clause says that this spiral reinforcing shall be no more than 200 millimetres. Do you accept that the transverse reinforcement of R6 at 250 was insufficient to meet that requirement?

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A. That, yeah I, I mean I would've thought that confinement to the beam column joint, if you're talking about its capacity to resist vertical loading, is you're getting confinement by all the horizontal bars in the beam, that's confining the joint. So I struggle to see, if you're not, if you're talking just vertical loads I presume are you?

Q. Well I'd need to check the evidence but I can't take it further than the proposition that's been put there, insufficient to meet those requirements of the clause?

A. Well…

Q. I think, if it assists, I think this is a matter which Dr Reay accepts as non-compliant. Mr Palmer will correct me if I'm wrong.
A. Okay, well it certainly, it's at 250. If it has to be 200 because of that clause I don't, you know, without me studying that further I guess if Dr Reay accepts it and that appear to be what it is, I do have difficulty though. I think that that clause applies, oh, that's for non-seismic loadings?
Q. Yes it is. You said to Mr Rennie earlier on that you thought it was from the seismic section but it's not is it?
A. Yeah. "With the exception of joints connecting beams in which case the joint reinforcement may reduce to one half of that."

JUSTICE COOPER:
Q. What are you reading from?
A. I'm reading 9.4.8. Yes but no you're correct it should not be further than 200. So you're right, it should be 200. I accept that.

CROSS-EXAMINATION CONTINUES: MR ELLIOTT
Q. Were you aware of that clause when you designed the building?
A. Not that it should be, I can't tell you whether I was or not, it's too long ago.
Q. The next matter relates to the diaphragm. Distinguishing the diaphragm I think from the diaphragm connection to the walls and you'll see in the next row of the table on page 7 there's reference there to clause 10.5.6.2 of NZS3101?

WITNESS REFERRED TO DOCUMENT
A. Yeah.
Q. And 5.3.32. Now this comes from Mr Jacobs' evidence. You've obviously read that recently because Mr Kirkland took you through it?
A. Yes.
Q. You appreciate this point?
A. Yes I –
Q. Or do you (inaudible 16:39:21) the code?

A. – I mean it’s not designed as a two-way slab. I think that the mesh met the requirements of the day. I don’t, certainly we wouldn’t use mesh in a floor today but that was, the specific design that was done of the floor at that time and I believe that the mesh met the requirements.

Q. Now next on that issue, you would've seen earlier on that I asked Mr O’Loughlin to look at clause 3.4.6.3?

A. Yes.

Q. Of NZS4203:1984? Do you want to see that or do you recall that it says that diaphragms must be designed using the forces set out in the parts and portions section of the code?

A. Yes.

Q. Were the diaphragms designed using those forces?

A. I would have to study the calculations again. I would've assumed so.

Q. Yes?

A. But I accept that your figure in there, that 30 kilonewtons that you talked about –

Q. I’ll come back to that. I’m just talking about the diaphragms themselves. You need to consider that so would you please consider that, and you can come back and confirm your position once you’ve considered it.

A. Okay.

Q. The next issue over the page, now this is from some questioning to you from Commissioner Fenwick. The reference is TRANS.20120731.90

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WITNESS REFERRED TO TRANS.20120731.90 – LINES 4-20

Q. If you’d like to read this to yourself.

A. Yes.

Q. You’ve accepted there that capacity design applied to the walls and to the diaphragms haven't you?

A. Yes it’s something which we’ve been giving consideration to since the earthquake but I don't believe that that’s something that’s been routinely used by engineers to design the diaphragm connection for the capacity design force, although I accept that when you think about it that is a sensible load to use but it certainly was not designed for a capacity design force, it was designed for the load from the computer output so the shear reinforcement in the wall was certainly designed for capacity design and that's traditionally the effect that you use in a shear wall building.

Q. You've used capacity design in relation to the north core and the south wall haven't you?

A. Yes, in terms of the shear reinforcement in the wall.

Q. But you haven't considered that capacity design should be applied across the structure as a whole taking into account the north core, south wall and the diaphragms connecting them?

A. That would be true.

Q. But you accept that you should have done that if applying capacity design don't you?

A. I don't believe that that was standard practice at the time. The difficulty I think is that the, perhaps there's not as much time spent in the codes explaining what the requirements for the diaphragms are as there are explaining what the requirements are for walls. So it's not something that's necessarily brought to your attention when you're designing a building.

Q. Do you accept that on this building it would have been artificial just to design each of the north and south walls using capacity design but not the section in between?

A. Well I think the, what happens here is that when you are modelling the building, the ETABS model assumes a rigid diaphragm and it's always been pretty much believed that diaphragms are rigid and I don't think that the, well I don't think the Code is necessarily as clear as it could be in specifying what the requirements are for diaphragms, if they are to be designed for capacity design or not. But I'm agreeing that this was not, capacity design was not used in the diaphragm if that's what you're asking me.

Q. We'll talk now about the diaphragm connection to the north core. Firstly, just assume, I know that you've given some answers contrary, just assume that capacity design did apply to the walls and the diaphragms.

If that was the case it would also have needed to have applied to the connection between the diaphragms in the north core and the south wall wouldn't it?

A. Yes.

Q. And on that scenario your loads used to calculate the capacities required for the diaphragm connections would have needed to have been greater than the loads which it would take to cause a plastic hinge to occur in the base of the walls.

A. Yes.

Q. But that wasn't something that you took into account or applied.

A. No it was not.

Q. You were here earlier during the evidence of Mr O'Loughlin when I asked him about the calculations relating to the diaphragm connection. You heard that evidence.

A. Yes.

Q. And is it right that in calculating the loads you used to determine the necessary capacity the diaphragm connection at every point, north and south walls, you used forces derived from the equivalent static method.

A. Well that was the question you asked me previously and I, or whether it was parts and portions and I can't answer that. It was, yeah it was, it probably was the equivalent static method or the dynamic analysis, whichever gave the bigger figure, rather than the parts and portions I suspect.

Q. Have you seen the evidence of Mr Banks?

A. No I haven't studied that. I've seen that the calculation has been done as to what the drag bars should have been designed for but I haven't studied that.

Q. His evidence is yet to come but it appears, I'll check this with him but it appears when he used the parts and portions provisions he arrived at a figure of 724 kilonewtons as being the applicable force/load required at the north core connection, whereas you've calculated 300 kilonewtons haven't you?

A. Is he talking about a north-south earthquake or an east-west?

Q. East-west. East-west.
A. Well if its east/west and 700 is the total load then some of it will be to the north wall and some to the south wall.
Q. No I think that's what he apportions to the north wall.
A. Oh well I take that as your statement. I don't know I haven't looked at that.
Q. Well if he did reach the figure of 724 apportioned the north wall, east-west direction, based on parts and portions it's obviously more than your figure of 300 kilonewtons.
A. Well if that's the figure he's used yes but, as I say, if it's 700 for the whole building then there's less proportion to each wall but, 300 is the figure I used.
Q. The proposition which I put to Mr O'Loughlin which I think he agreed with was that if you did use your loads from your equivalent static method rather than parts and portions forces which you would have used to determine the capacity of the connections at the north core and south wall at every level of the building may have been less than the minimum required by the Code. What's your position on that?
A. Well I've already agreed that we didn't design it for capacity, to develop the over-strength bending capacity of the shear wall. I've accepted that. So the figure we would have used would have been less than that. Whether that's what the Code requires I can't really comment on that.
Q. On page 9 – Mr Kirkland might want to correct me but I don't see a need to get a comment on compliance following installation of drag bars.
A. No well I haven't looked at the design of the drag bars. This is the post, the remedial work?
Q. Yes.
A. I can't comment on that.
Q. Then we have the spandrel panel separation. This is a matter referred to in the Hyland Smith report, do you recall that?
A. Yes I do.
Q. Do you agree that there was no seismic gap specified in the drawings in relation to the spandrel panels?

A. No there was a gap of 10mm specified.
Q. Was it noted as being a seismic gap though?
A. Well it doesn't have to be noted as a seismic gap. It's detailed as a gap and that's all that we're required to do.
Q. Now turning to matters which I've described as best practice. These are just matters which have emerged from witnesses who have not necessarily commented directly upon Code compliance but have said well these matters didn't amount to best practice so, again, I'll just run you through them. Firstly, Professor Priestley talked about the diaphragm connection and said that, "One should seek to achieve a

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sufficient connection between the floor slab and the wall." He said there was, "A lack of adequate connection." He said, "It was clearly inadequate to achieve a sufficient connection. The lack of design connection between the slabs and the wall at lines D and DE was remarkable." What's your response to that?
A. Well there was a connection between the slabs, sorry, between the diaphragm and the north core. The essential connection is by the floor slabs connecting to the beams and the beams interlocking with the reinforcement in the shear walls. Now there are four of them, I think, you say D, and D and E. I assume the other ones must've been B and C. So B and C are where the beams are connected and I'll accept that on D, and D and E there is not the same connection and that is I think the reason for the retrospective connection. So whether the original connection was adequate or not I guess comes back to what force you are designing it for. But I don't see, I think that they've done the right thing by if there is an improvement that you can make by improving the connection on line D, and D and E, then you should do so to remove any doubt. But I don't necessarily accept that it was inadequate in the first place.
Q. I don't know whether I actually put this to you but what about the proposition that the connection at the north core was non-compliant with code at the time of the building permit, do you accept that?

A. No, I can’t accept that at the moment. Not on the basis of the information I have. I’ve, it could definitely have been improved, I accept that and that’s what they’ve done by connecting to the walls on line D, and D and E, but I don’t accept that you have to have a connection on D, and D and E in order to support the load specified in the code.

Q. We’ve dealt with eccentricity, on page 11 the issue of robustness, now this comes from a few sources in particular the proposition is that robustness which means the ability of the structure to sustain damage without causing progressive damage to the building as a whole, and Dr Hyland and Mr Smith said that the secondary beam and column frames lacked the level of robustness expected of frames designed to cope with the cyclic drifts of earthquakes. Seismic design provisions of NZS 3101 would've improved robustness. Do you accept that?

A. It would have improved it, yes. Robustness is a difficult thing to identify just what you mean by that. I mean it’s, I guess one way of looking at it is that there’s a secondary line of defence if the first one fails, and I guess that’s really what we’re talking about. So I think we’ve learnt since the earthquakes that buildings move a lot more than we thought they would and things happen that, not just in terms of vertical acceleration but also one building hitting another, and by buildings deflecting by more than you think that they should, so I agree that in the future we should be giving that a lot more thought. But I don’t accept that, at the time we designed it, it was believed that it was sufficiently robust to be safe.

Q. It was believed by you and Dr Reay?

A. Yes, I believe that the concept of a gravity frame stabilised by a shear wall is a valid and robust form of construction is what I’m saying.

Q. You may have answered this point but the next issue there is redundancy. Do you agree that the building lacked redundancy, in particular if the columns or beam column connections failed then a whole or partial collapse would result?

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A. Yeah that's the thing I was just talking about with the second line of defence and I must say I've struggled with that myself looking at many buildings. I mean most buildings you have only one, one line of defence. You only design one set of elements to do each job. I guess the redundancy we have here is it's not just one wall. If one wall fails there's other walls there that are able to take the load so I think if there was a reduced number of walls I would accept that but with the number of walls we had there I, yeah it's a value judgement but I don't believe that's true. I believe that the building was acceptable with the number of elements it had.

Q. Professor Priestley made the comment that the lack of ductile detailing, that's the use of the non-seismic provisions of the Code for transverse reinforcement of columns, was not best practice, especially given comments made in the 'bible' (Park and Paulay) in the mid 1970s that ductile detailing of columns is recommended, especially where there are high axial load levels. Do you accept that the lack of ductile detailing for columns was a failure to comply with best practice?

A. I think that the ductile detailing that you're referring to again in the Park and Paulay textbooks and recommendations really applies where you're designing a beam column frame and that that beam column frame is providing the lateral restraint to the building. That's where you definitely do need the ductile detailing and there's no argument that in a beam column frame, well it's not just ductile detailing it's a ductile frame. Where the lateral load resistance is provided by walls the whole reason for doing that is so that you don't have to do ductile detailing for every other element. So it comes back to that, you know, elastic secondary frame staying elastic that we were talking about earlier but I think that if that was the case then the ductile detailing wasn't required so I think, again I've been struggling with this ever since as to what we could have done. I think really in terms of the vertical acceleration if the columns had had the ductile detailing in them they would have been tougher but I'm not sure that they would have had a higher capacity. I think that once the covers spalled off and you're left with the ductile core of the column the axial load capacity of that is probably still not enough to hold the building up so I think, really, it comes back to having your structural elements big enough to take the loads that are specified and I think if you have vertical acceleration you'd need bigger columns. That by itself is what we should have had. Now whether they should've also had ductile detailing. I think myself all buildings that I'm designing and have designed for some time I've always used ductile detailing in columns, whether the codes required it or not and I do believe that's what we should be using just as a matter of course but I don't believe that was best practice at the time. I think even now I don't know whether we have that as a best practice requirement. I would like to think that it was but I don't believe that it is at the moment. Just for non, I'm talking about non-seismic elements, gravity elements. I'm certainly accepting if it's a seismic resisting frame it needs it but for a gravity element well…

Q. Have you used seismic detailing for columns in every building you've designed apart from the CTV building?

A. Say again.

Q. Have you used seismic detailing for columns in every building you've designed apart from the CTV?

A. I can't say that no.

Q. Just to conclude, I see the time Your Honour, there is a related point over the page if I could just conclude with that. Professor Priestly, I think you've replied to this Mr Harding but Professor Priestly also refers to excessive spacing of the transverse reinforcement and the minimum requirements being inadequate to achieve ductility but I think you've replied to that haven't you?

A. Yeah.

HEARING ADJOURNS: 5.01 PM

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