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Tuesday, 7 January 2020

(10.04 am)

DR LAU CHI WANG, JAMES (on former oath)

Cross-examination by MR SHIEH (continued)

MR SHIEH: Good morning, Mr Chairman, Mr Commissioner. Dr Lau, good morning.

A. Good morning.

Q. There are only a few areas that I wish to pick up with you. First, concrete strength. Can I ask you to look at bundle C13, page 8376. That's the Concrete Code. Can I draw your attention to 3.1.2:

"Unless otherwise stated in this Code of Practice, the characteristic strength of concrete is that value of the cube strength at 28 days below which 5 per cent of all compressive test results would be expected to [fail]."

Do you see that definition?

A. Yes.

Q. Does it follow, therefore, that the characteristic strength as defined by the code depends upon concrete cube test results?

A. The concrete cube test result is supplied by the supplier, not the site thing, because the concrete supplier provides grade 40 -- for example, in this case, they provide grade 40 concrete to the client, to the

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contractor, and they also carry out a whole series of tests themselves which are also cube tests, and they are always higher than the strength supplied by the supplier. They can only guarantee -- if they supply grade 40, the supplier only guarantees grade 40 strength. That's all. The strength actually tested by them are always higher than grade 40. But if you ask the supplier, if they supply grade 40 to you, can they guarantee grade 60? They wouldn't. That's the only strength they guarantee, grade 40. And this is the grade 40 concrete strength that we are going to use on this site.

Actually, in Hong Kong -- that's the way we practice in Hong Kong. We rely on the concrete grade supplied by the supplier, which is grade 40 in this case. This happens all over Hong Kong; right?

Q. Dr Lau, it is indeed recognised practice to take samples of concrete cubes during construction --

A. Yes.

Q. -- and to test them and to confirm that the concrete strength meets the design requirement?

A. Yes, as a quality control.

Q. So people do take concrete cubes --

A. Yes.

Q. -- from the actual concrete being used in

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construction --

A. Yes, agree.

Q. -- and test them?

A. Agree.

Q. You said in your evidence that concrete cube test results done by testing in a laboratory should not be relied upon because they are always higher than the actual concrete strength used in the structure?

A. Yes, that's right, because of -- even in terms of workmanship, in the preparation of the concrete cubes, the skilled worker compacts the concrete properly and cures them properly in the water tank, under constant temperature, for 28 days, before they were tested in the laboratory. So the strength is always high, no doubt about that.

But the concrete inside the structure was cast by the contractor into the structural formwork, and the strength -- we don't know what sort of workmanship involved in the compaction, we do not know how good the curing is. So in general, generally the strength inside the concrete would be lower than the strength test in the laboratory; right?

In this particular case, we found there are a lot of -- what I want to say is it depends on the workmanship in curing. This is a very important point.

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In this case, because of the honeycomb in the concrete, I think we cannot rely on the strength from the concrete cubes and say that this is the strength inside the concrete. If Leighton wants to demonstrate it is higher strength, there's one way they can do, to actually core concrete from the structure, enough core from the structure, and test it in the laboratory, to demonstrate that it is higher than grade 40. In that case, I can accept that. This is the difference between me and the other three experts. I think this is a very important point. You have to distinguish between concrete strength from concrete cube tests and actual concrete strength inside the structure. This is a very important point.

Q. Dr Lau, all your points are no doubt very important points but let me just take up your points one by one. Insofar as the point that in a laboratory setting the concrete is taken and tested in the form of a cube, this is what is known as a shape factor and so in the design context it would have been taken care of by a factor of 0.67; do you accept that?

A. I think you are wrong as well here, because I'm involved in the preparation of the Concrete Code. This is to convert the concrete strength from the cube into the bending stress into the structure. That's nothing to do

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with the shape, because they are test -- this concrete strength, crushing strength in the concrete cube, and we are trying to convert that into the bending stress into the structure. So that's why we multiply by 0.67, something like that. That's the idea in the Concrete Code; nothing to do with what you said.

Q. But it reflects the fact that the strength of concrete as tested in a laboratory is necessarily higher than the strength of concrete as used on site, so this 0.67 factor has already taken that into account; do you agree or not agree?

A. This is not for that purpose. Not for that purpose. A different purpose. It doesn't -- it's not used that way. It's to convert the crushing strength in the cube to the bending strength in the structure. That's the purpose of that particular clause in the Concrete Code. So it's for different purposes.

Q. And there is, on top of it, a 1.5 factor which is applied in the design context, to take into account, basically -- allow for a margin, so to speak?

A. This is to be applied to the fcu, which is the grade 40 concrete supplied by the supplier. This is to be applied to the --

CHAIRMAN: Sorry, "fcu" stands for ...?

A. "Fcu" is actually the concrete strength used in the

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design, and this fcu comes from the grade 40.

For example, if the supplier says, "My concrete is grade 40", then fcu is 40. That's the meaning of that.

CHAIRMAN: Thank you very much.

I cut across you. So, "This is applied to the fcu, which is the grade 40 concrete supplied by the supplier. This is to be applied to ..."? Because I think, Mr Shieh -- sorry, perhaps you could repeat the question there that's at [draft] 5:25.

MR SHIEH: Can I perhaps just put a numerical example. When designing to use concrete with a strength of, let's say, 400 megapascals --

A. Okay.

Q. -- in fact the actual expected load is 400 multiplied by 0.67 and then divided by 1.5?

A. Now, the first one, the 0.67, is to convert the strength to bending stress, first of all. The 1.5 is to be applied to the grade 40 concrete. That's what we use in all our design. That particular fcu is not the concrete cube strength. The fcu is actually the grade 40 strength which is guaranteed by the concrete supplier. This is the difference between you and I.

Q. And it's a very important point?

A. I tell you, nobody in Hong Kong do it your way. No structural engineer in Hong Kong do it your way.

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Q. Of course I don't do it my way.

A. I know. I want to explain to you that it is important point because I think the other experts are trying to use the concrete cube strength as -- multiplied by all these factors and put it into the structure, which is wrong. This is a totally wrong idea. I want to make it clear in this particular Inquiry. This is not what we do in Hong Kong, not what we do. We never do it in Hong Kong anyway, never.

Q. Can I move on to --

CHAIRMAN: Sorry, can you help me -- how do you do it then?

A. Well, the fcu actually is the concrete -- if we order grade 40 concrete from the supplier, they give you grade 40, it is written in the document, "This is grade 40", but to prove it is grade 40, we do a lot of other tests. The tests are always cube tests. They always show that the strength done by the tests are higher than the grade 40 concrete, which is 40.

Then, in my design, I use the grade 40 in my design. Only 40. I'm not using those cube strength tests from the supplier for my design. They were just data to support the guarantee from the supplier. So, actually --

CHAIRMAN: That's what I've understood. Perhaps I might be disabused. I've understood that the cube tests confirm

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the strength of the concrete, minimum strength of the concrete in the structure itself?

A. Yes, for quality control.

CHAIRMAN: For quality control purposes.

A. Only. They are not for design. They are only for quality control. I think this is a very --

CHAIRMAN: But how would they be used for design?

A. In the design, it's entirely from -- now, let me put the procedure. If MTR want to use grade 40 for the design in the structure, you put down grade 40 in the design document. So, when the contractor come in, based on the design document, they order grade 40 concrete from the supplier. They give you grade 40 concrete. But of course, to prove it is grade 40, they have to do a lot of tests, and all these tests demonstrate that the concrete supplied to you are higher than grade 40 and they guarantee that the concrete supplied to you are actually better than grade 40, but they use grade 40 in your design.

CHAIRMAN: Yes.

A. You don't use something else.

CHAIRMAN: No, no.

A. That's the important point.

On site, they also carry out a lot of other cube tests, as quality control, to ensure that the concrete

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you use are higher than grade 40.
CHAIRMAN: Yes.
A. You don't use those cube strength tests for your design. They are only used for quality control.
CHAIRMAN: That's right. That's as I understood it, actually. I thought that if the design requires grade 40, you know that that's what the contractor says, the contractor is supplying the wet concrete, and then it goes in, but you want to make sure it is at least grade 40.
A. Yes.
CHAIRMAN: And so you have cube tests which will always come out higher because of the easier or the better circumstances in which it's made hard and cured, et cetera. But what it does do is it acts as a test to ensure a minimum strength for the actual concrete in the structure.
A. Yes. You've got the point. This is the point I want to make. But the other experts said, because the cube strength test says it is 80 or 60 -- that's what Mr Nick Southward said -- we should use 60 or 80 in the design. To me, it is totally wrong. This is unacceptable to me.
CHAIRMAN: All right.
MR SHIEH: I don't think that's what they say, but anyway, we can read what they say.

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CHAIRMAN: Okay.
A. But I think this is a main point of difference between me and the other experts.
CHAIRMAN: Okay. Thank you.
MR SHIEH: Dr Lau, the transcripts speak for themselves and I don't believe that you are accurately understanding what the experts are saying, but we can agree to disagree.
Can I now move on -- let me try to put it one more time. Concrete test results in a laboratory, it is accepted that they would be higher than the strength of the concrete actually used on site, but that factor is already taken into account by the conversion of 0.67 and 1.5. Do you accept that?
A. This is to convert the crushing strength of the concrete to bending strength in the design. This is the main purpose for this particular 0.67. We are not -- what Mr Southward is saying is that we use the cube strength test in the laboratory, multiplied by 0.67, and use it in the design or structural assessment. This is something which I totally disagree, absolutely disagree with him on this point. He's trying to say that we use 0.67, multiply to the concrete cube test result, which is very high, 80 or 60, and put it into the structural assessment. This is totally wrong, unacceptable to me.

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We should use only 40, rather than this 60 or 80 multiplied by 0.67. This is a totally wrong concept.
Q. In design, don't you actually use the bending moment?
A. Yes.
Q. Bending strength?
A. Bending strength.
Q. Don't you actually use the bending strength when you conduct your design?
A. Yes.
Q. Can I move to the topic of the trough walls and the yield line analysis.
CHAIRMAN: Sorry, please forgive me. I'm just trying to wrap this around my head.
The cubes are used to -- they are a way of testing the strength of the concrete actually in the structure that's been poured in; correct?
A. Can you repeat your question, sir?
CHAIRMAN: Sorry. These cube tests are a way of testing the strength of the concrete that's gone into the actual structure?
A. It's a way of guaranteeing that the concrete inside the structure is up to certain strength.
CHAIRMAN: Okay. So it's a way of ensuring a minimum grade or strength?
A. Yes.

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CHAIRMAN: Okay. Now, to ensure a minimum grade or strength, you have to give the cubes a grade or a strength?
A. The cube, the grade of the cube, are actually grade 40. Now --
CHAIRMAN: What I'm saying is once you've cured it and dried it, it's going to have a particular strength?
A. Yes.
CHAIRMAN: Which is going to be higher than that in --
A. The structure.
CHAIRMAN: -- the structure. But you could argue that if it's not high enough or not higher enough, then the concrete in the structure is not up to standard. There has to be a comparison between the two.
A. Yes, you are absolutely right, sir. This is the whole point. So the strength for the concrete cube has got to be higher than those you specify for the design.
CHAIRMAN: Exactly, yes.
A. That's the whole purpose for the concrete cube test, to make sure it is higher, not lower.
CHAIRMAN: Yes.
A. So it's always higher, that's why, because the supplier wants to make sure that it is higher.
CHAIRMAN: But if mathematically it works out that it's a lot higher than, say, 40, is it not permissible then

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to say the cube tests show not only a basic strength of 40 but actually show a strength of 50?
A. This is what they are trying to argue, the other experts.
CHAIRMAN: That's why I'm asking: is it not permissible to do it?
A. It is not permissible. I'll tell you why. As I said previously, it depends on the workmanship. To prepare the concrete cube, you have one type of workmanship, one type of curing, but the concrete pour in the concrete, whether type of workmanship and the type of -- in the form of curing which is poorer than those done on the cubes. So we do not expect the strength in the concrete, in the structure, the same strength as those tests in the cubes.
In this particular case, I think they have grade 40 concrete, I have no doubt about that, but because of poor workmanship, we have to be very careful. Even if they try to put those concrete cube test results to put into the structure, you've got to be very careful. It depends on the workmanship.
CHAIRMAN: I'm with you, yes.
A. Because, I tell you, I think they have some point, because in their report they keep on saying forensic engineering, forensic investigation. In the forensic

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investigation, trying to prove -- to find out what caused certain collapse, people do use this sort of analysis to work out what is the strength in the structure. You can do that if we have something like the collapse of a structure. But in our case, we are not doing that. We are trying to do rectification to the structure, to ensure that we have certain factor of safety for the rest of the design working life. So we are talking about two different aspects. They are talking about forensic engineering, forensic investigation. I'm talking about rectification for the structure to last for the intended design working life. So we are talking about two different concepts here.
CHAIRMAN: Very good. Thank you for your help.
MR SHIEH: Let me try one last time and then I'll move on. The 0.67 and divided by 1.5 conversion formula is to convert, as you say, the supplier's grade strength for the concrete into what you call the bending strength, which would be the strength that the concrete would actually have on site; correct?
A. Yes.
Q. So, when I design something and I say to myself, "I want to achieve this particular bending strength for the concrete on site" -- let's call it X -- I don't actually order concrete of strength X because I need to actually

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order concrete of a higher strength so that, upon conversion, according to the formula, it would be converted down to X; correct?
A. (Nodded head).
Q. Is that correct?
A. Actually, if you order grade 40 concrete, they give you grade 40 concrete; yes? If you want to do what you want, you order grade 60 concrete, in that case, because it depends on what you order. You order grade 40 concrete, they can only guarantee grade 40 concrete to you, with a lot of additional tests.
Q. Dr Lau, if I order grade 40, in the lab, if I see grade 40 -- you know, the grade 40 test being fulfilled, I will know that when used on site it is not going to be 40; it would be less, yes? That's what you are saying?
A. No, no, no. If you order grade 40, you understand that it will be at least grade 40, not lower, at least grade 40.
Q. No. I think you are confusing -- I don't know whether it's deliberate or not. Of course the grade is grade 40, but what you are saying is the strength demonstrated by the laboratory test, when used on -- the same concrete, when used on site, is going to be of a strength less than what is demonstrated by the lab test?

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A. Can you repeat your question again?
Q. Let's say the test results in the laboratory show that it is of a certain strength -- let's say 400 --
A. This is not what we do. Sorry. I think you totally misunderstand the whole concept.
Q. That's why I'm not an engineer. Educate me, please.
A. You order grade 40; right?
Q. Yes.
A. You do the concrete cube test to make sure that it is at least grade 40. It's got to be higher. But in your design --
Q. Sorry, what do you mean by "it's got to be higher"? Higher than what?
A. If you order grade 40 concrete, you do the concrete cube test, it can be 50, 60, 70 -- it can be anything above grade 40, always. Always. But when you do the design, you still use grade 40. You don't use the cube test result in your design. Otherwise, it will be very confusing, because --
CHAIRMAN: I see that point. You are saying that these tests --
A. Very confusing.
CHAIRMAN: -- are purely and simply there to confirm minimum grade of 40, and the tests will always come out higher and that's why you apply these mathematical formula to

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them, to bring them down to a level where they equate to what's actually in the structure, but you don't do -- those tests are required for nothing more than confirming minimum grade strength in the structure?
A. Something like that. Basically, what we want to do is -- if you order grade 40 concrete, the supplier guarantees it is grade 40 concrete, you use grade 40 concrete in your design and that's it. You don't use the cube test result which is much higher and use it in your design, which will be very confusing because in that case, in Hong Kong, all structures will have different strength, depending on the cube test result. This will be very, very confusing for everybody. There's no -- well, this is what I mean.
CHAIRMAN: All right. I may be way off the point here, then we'll finish it, but let's say you've got a problem. The grade 40 concrete comes in. It's tested at a figure much higher than 40 in the laboratory; okay?
Then you have a problem as to strengths later and you revisit. Is it not permissible then to say the cube tests, when revisited, showed strengths of such a high level in comparison to what's actually in the concrete structure that it shows that what's in the concrete structure wasn't just grade 40 but was higher and therefore had excess capacity by way of strength or

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resilience?
A. What you said is normally done on a collapsed structure, for example.
CHAIRMAN: Okay.
A. Take two examples. If there's a building that collapsed, say for example, you want to find out what happened, what caused the collapse. Then what you said can be useful because we want to find out what caused the collapse and we do a lot of investigation, coring of the building, to find out the what they call characteristic strength of the concrete and do it in the back analysis, to see what happened.
But in our case, we are not doing that. In our case, we are checking the design for the rest of the design working life of the building. If the designer asks for grade 40 concrete, we should check the structure based on grade 40 concrete, rather than based on all the concrete cube tests, right, from the cube test.
This is not what we do for a normal design of a building. But when they are doing the forensic investigation, I can understand why they want to do it that way. They want to find out exactly what is the strength in the structure. In that case, you still have to core the concrete, to core the structure, to find out

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the strength, rather than using the concrete cube test results, because they are not relevant. They are only relevant as far as the material is concerned. They are not relevant as far as the workmanship in curing is concerned.
Do you take my point?
CHAIRMAN: I do. Thank you very much.
A. I hope I can explain it to you, because this is a big --
CHAIRMAN: No. I understand it. Thank you very much.
A. There's a big difference between me and the other experts on this very point.
MR SHIEH: Can I take up Mr Chairman's question. Let's say when you design, you say to yourself, "I want to use grade 40, I order grade 40."
A. Yes.
Q. And by supplying me with grade 40, the supplier guarantees that it would be at least -- the cube results would be at least 40; yes?
A. Yes.
Q. At least 40. You say, in reality, it may be higher when tested, but at least 40. So let's say I, the contractor, upon seeing the concrete delivery, take a cube for testing. It is just 40; it would pass, correct?
A. Yes.

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Q. Because I have designed my structure to be constructed by using grade 40 concrete, and it is grade 40 so it passed, I would happily build it and it would fulfil the strength requirement that I have; yes?
A. Okay.
Q. If that is the case, then if the cube test result is actually higher than 40, let's say 60 or 80, it must follow, must it not, that it is of a strength higher than what I actually want. Do you accept that? If I want a certain strength and I say to myself, "40 is good enough for me", if the result turns out to be 80, for example, it must be far, far in excess of what I actually need to sustain the structure; is that correct?
A. But the supplier -- if you go to talk to the supplier, "Can I use it for grade 60?", they will say, "No, you use it for grade 40. This is what I guarantee you, that it's a grade 40", because --
CHAIRMAN: I appreciate that.
A. Because I think counsel has the wrong concept. They give you a higher strength doesn't mean that you can use it in your design, because in Hong Kong, the most important thing is you buy grade 40 concrete, they supply you with grade 40 concrete, you use grade 40 concrete in your design. It doesn't matter what sort of

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test you do.
CHAIRMAN: Yes. The way I've understood it is this, that you want grade 40, the contractor agrees to supply grade 40, he supplies the concrete, as far as he's concerned it's grade 40, but it needs to be tested; is it grade 40 or not?
A. Yes.
CHAIRMAN: And the way in which you test it is by these cube tests; okay?
A. Yes.
CHAIRMAN: Now, because of the rarefied circumstances in which the cube tests are conducted and the more bash-about circumstances in which the actual concrete is settling in the structure, you need to equal them out, and you do that with these mathematical equations or statements; okay?
If, later on, you come back and you want to try and get an idea of whether the grade 40 which you ordered was grade 40 or was of some different dimension, you go back six months later and you check everything and you see that all the tests on the cubes show that in fact the grade 40 was coming out at a consistent grade 60. Are you not then entitled to say, "I ordered grade 40, grade 40 was in the design, but when I tested it all and I averaged it all out, what in fact I had for this

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grade 40 requirement was a grade 60"? In other words, this concrete had more oomph per square centimetre than I ordered.
A. This is what they argue.
CHAIRMAN: I know. That's why I'm putting it in a question.
A. My argument is this. In order to do that, the only way that you can do that is to actually core into the structure, get the result, and then test those cores in the laboratory. If they show consistently that it is 60, then maybe you can do it this way. But at the moment this was not done. They only rely on the concrete cube test to tell me. This has no relationship with the workmanship and the curing condition in the structure.
If they are able to do something like what I said, they carry out a lot of coring into the structure, get the sample out and test it in the laboratory and do all this mathematics to show it to me that it is grade 60, then maybe I can accept that, maybe, only maybe.
CHAIRMAN: All right. I appreciate that. You have a fundamental difference of approach here.
A. Yes.
CHAIRMAN: Thank you very much.
MR SHIEH: Just one last question -- I keep saying, famous last words, "last question" -- when a laboratory test is

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conducted, the test -- how should I put it? -- the test is blind as to whether you are testing that sample for the purpose of initially passing it, for the purpose of using it on site, or whether you are testing the sample, let's say, during construction; do you see what I mean?
A. Okay.
Q. The test is blind as to whether you are testing a sample for the sake of design, for the sake of accepting a sample to be used on site, or whether you are testing it for the purpose of, let's say, doing a random check during construction. The test is blind as to your purpose. The test only knows you are testing a concrete cube.
A. Yes.
Q. Trough wall, yield line analysis.
A. Okay.
Q. You mentioned yesterday that there had been no checking as to shear strength in the trough wall.
A. No check, no.
Q. Can I show you bundle DD18, page 18512. This is from the AECOM calculations.
A. Yes.
Q. "Trough wall design":
"400 kilonewton collision load is spread over 2.2 metres."

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A. I've seen this, yes.
Q. If you move down to the bottom of the page, you can see, "No links required".
A. Yes.
Q. So does it not show that AECOM has conducted the relevant calculations for the trough wall and concluded that no shear links are required? Does that not count as the requisite shear calculation?
A. I'm talking about Mr Southward's analysis. He only carried out yield line analysis, he did not carry out any shear check. I'm not saying he did not carry out any shear check based on his own analysis. That's all. He's now relying on someone else's elastic analysis.
Q. But if it actually has been done, is Mr Southward not entitled to rely on it?
A. Well, it's only one comment, but anyway I think the more important point is -- what I said in my report is that according to the American Code he used, they said there's a requirement that you should check the shear when you use the yield line method. That's what I mean. As far as I'm concerned, if he checks it, I think he can pass it as well. I'm not saying that he will fail in shear. What I'm saying is he did not check it. That's all. That's what I said. In fact even yesterday I said the same thing.

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But the most important point is not the shear. In the American Code, there's no column behind the trough wall; right? It's just a wall. But in our case, we have a column only 60 millimetres behind the trough wall. So actually the deformation of the trough wall under the impact of the derailed train is even more important. He did not check it, but I checked it for him. I think it doesn't work.

I can demonstrate it on this paper. I did a very simple calculation, just a very simple calculation, and you can demonstrate that his method doesn't work. If you don't mind, I can do it for you.

CHAIRMAN: That's okay at the moment. We have your clear statement of that and then if anybody else wishes to ask you to demonstrate then, Doctor, thank you very much.

A. It's just a very simple calculation, it takes about two minutes, and then you can demonstrate that it failed. That's all.

MR SHIEH: Can I just get it clear once and for all, because in your report you referred to and relied on the American Code concerning the utility of yield line analysis which contains the relevance to having to do strut-and-tie; do you remember that?

A. Yes.

Q. Following on from that, you criticised Mr Southward for

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not having done a checking of shear forces.

As I now understand, you are not suggesting that Mr Southward must personally have conducted the checking, are you? You are not saying that he must personally have done the checking?

A. Actually, I don't really want to criticise Mr Southward, because different engineers have different methods of doing things. I just mentioned that if he wants to use yield line analysis, which is allowed in the Hong Kong Code, allowed, he should do a comprehensive check. That's all. That's all that I want to say.

I think a very important point to note is that I'm very concerned about the stability of the column, just behind the trough wall, and I measure it on the plan: it's only 60 millimetres away from the trough wall, and the trough wall has to be recessed to accommodate the column.

CHAIRMAN: Is remedial work being done in that regard?

A. Yes. What the work done is to have two struts, to connect the two trough walls, to show that any impact from the train will be transferred, away from the column to somewhere else. This is a very important remedial measure. I think it's got to be done. Otherwise, there will be trouble.

Now, there may not be any collision, there may not

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be any accident in the long life of the trough wall, but there may be one, so we need to be very careful about this suitable measure. We need to do something about that.

MR SHIEH: Dr Lau, can I refer you to your report, your COI 2 report, at page 11.

CHAIRMAN: Sorry, can I just ask here -- Mr Southward has done a report, but he wasn't involved in the design. He's come in as an expert to look at the design. Was there any discussion/concern/documentation about the nearness of the column to the trough wall?

A. You mean the documentation?

CHAIRMAN: I'm just wondering. You said it's really important, and what comes across to me is that if a train came and crashed into or fell against the trough wall, then the force of that could cause damage to the column, and the column collapsing could cause damage of far greater extent than would otherwise be the case.

So my question is simply: is this something that was raised at any stage, to your knowledge, during the actual designing of the structure, before anybody came in their Wellington boots and started building it?

A. I don't know, but actually, during the site visit, joint site visit among all the experts, we saw the columns.

CHAIRMAN: Okay.

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A. We saw the columns.

CHAIRMAN: But you don't know if it was ever a debated issue in the designing stage of the work?

A. I don't know. I'm not aware of that, no. Sorry. But the columns, we all saw the columns during the site visit.

CHAIRMAN: Yes, of course.

MR SHIEH: On reflection, I don't think I need to take the point any further. I think I've got what I want.

Mr Chairman and Mr Commissioner, I have no further questions.

It's probably surplus for me to say this but obviously, this being a Commission of Inquiry and the experts having written expert reports, the fact that I haven't actually put each and every point of disagreement doesn't meant we are accepting what Dr Lau --

CHAIRMAN: This is not litigation of the classic kind, it's an Inquiry, and we are obviously not requiring you to follow arid formula. Thank you.

Cross-examination by MR BOULDING

MR BOULDING: Good morning, Dr Lau. I act on behalf of MTR. My learned friends Mr Pennicott and Mr Shieh have already raised many of the matters I intended to raise with you, but I nevertheless have a few questions about

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the approach in assessing safety factors.
A. Okay.
Q. So I wonder if you can help me.
You deal in your report -- that's ER2 at tab 17.1, page 11 -- COI 1; yes, that's the one -- and here we are, are we not, in the section of your report where you deal with safety?
A. Yes.
Q. We can see that in paragraph 32 you give opinions as to the determination of the applicable minimum safety factor; correct?
A. Yes.
Q. You tell us, do you not, that this varies from one place to another?
A. Yes.
Q. And you say it will be difficult to rely on one expert's opinion to set out the relevant standards; correct?
A. Yes.
Q. You also go on to say:
"It should represent society's general expectation of how 'safe' structures erected in that place should be."
That's correct?
A. Yes.
Q. Then paragraph 33, going on, you say:

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"... it is only appropriate to adopt the minimum factor of safety prescribed in the relevant building design codes in Hong Kong."
A. Yes.
Q. I understand that to still be your position?
A. Yes.
Q. Now, Dr Glover is coming to give evidence -- fairly shortly, I suspect -- and he deals with safety in his report, and presumably you would agree with him, would you not, that in the inception and design stages of a project, the inception and design stages of a project, much is still unknown as to matters such as, firstly, the actual future construction loadings and sequence? Much is still unknown about that, isn't it?
If you agree with me --
A. Okay, yes.
Q. Thank you.
A. Let me agree with you at the moment, yes.
Q. Fine. That will do.
Similarly, to have another example, much is unknown, is it not, as to, say, material strengths -- at the design and inception stage, much is still unknown as to material strengths?
A. I'm not so sure about that. Anyway, you can carry on. I'm not so sure about this point.

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Q. I'm putting it to you. Are you agreeing, disagreeing, or you don't know?
A. I disagree, because you specify what material you use, so you must know what is the strength of the material. So I don't understand your question.
Q. Well, I think the question is fairly clear. You seem to be disagreeing with the proposition that Dr Glover is going to come and support in about half an hour.
A. Actually, I disagree with him on this point, yes.
Q. Okay. I also suggest that at the inception and design stages of a project, much is also unknown as to, say, the geometric accuracy of the structure?
A. Well, anyway -- actually, I disagree with him on all these points. Actually, you can carry on. I can explain why later on.
Q. Well, you agree with me on the first one, I think, but there's disagreement, as I understand --
A. No, I --
Q. -- between material strength and geometric accuracy.
A. (Overspeaking).
Q. Can I go on to say that for these reasons, international codes and standards contain, do they not, what are referred to as partial safety factors?
A. Yes, we have --
Q. That's correct, isn't it?

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A. Well, the partial safety factor is intended not for the construction stage. It's intended for the long design life of the building. This is the reason why I disagree with him.
Q. Well, let's see what we can agree. We are in agreement, are we not, that international codes and standards contain partial safety factors? Do they or don't they?
A. Hong Kong also has partial safety factors, but the partial safety factors is not intended for the construction stage. They were intended for the intended working life of the building, for the uncertainties during the long life of the building. It's not meant for the construction stage. This is what I disagree with Dr Glover.
Q. So you accept that the international codes and standards, as well as, you would say, the Hong Kong codes, contain partial safety factors, but as I understand it there's a dispute between you and Dr Glover as to what matters they are intended to cover? Is that where we've got to?
A. The partial safety factors are intended for the design life of the building, not for the construction stage of the building.
Q. Well, I've got to disagree with you there, but there's an issue between us there, Dr Lau.

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A. Yes.
Q. And these partial safety factors, they would include, would they not, for extremes of the variations in the applied loads?
A. During the design life of the building, yes.
Q. And they would also include for what I am told are referred to as ignorance factors; correct?
A. Ignorance factors during the design life of the building. I have to maintain this point. They were for the design life of the building which is 120 years, not two or three years during the construction stage.
Q. I've got to take issue with you there, because what I suggest to you is that these ignorance factors are to reflect the level of uncertainties in the assumptions made in the design; that's correct, isn't it?
A. No, no, no.
Q. They are also there to reflect the sophistication of the analysis methods to be adopted to mitigate these unknowns; that's correct, isn't it?
A. The so-called factor of safety covers everything, right from the construction stage up to the end of the design life of the building. This is what I want to maintain. Not just for the first few years, during the construction. It's intended for the whole length of the design life of the building, which contains a lot of

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uncertainties. That's what it is intended for.
You are trying to tell me that after the initial construction stage we can reduce the positivity factor, which I totally disagree. I totally disagree with you on this point.
Q. Okay. Well, there we are.
Presumably, you would agree this, would you not, that Dr Glover says that insofar as structural safety is concerned, there are indeed safety factors contained in the Hong Kong Code of Practice 2004?
A. Yes.
Q. If we were to look at the Hong Kong Code of Practice -- I think we can pick that up at H8/2818, that's the first page -- but for present purposes if I could go to page 21. H2821. Splendid.
Here we see, do we not, the foreword; correct?
A. Yes.
Q. We can see that it provides guidelines, can we not?
A. Yes.
Q. Then if we look at the third paragraph:
"[The] Code of Practice is based on the limit state design philosophy, which provides a more realistic assessment on uncertainties associated with different loading conditions, material properties, workmanship et cetera. The drafting of this Code of Practice has

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taken into account the local conditions, work practice and development of new technologies in analysis, design and strength of materials."
So we can see, can we not, that the Code of Practice itself is referring, is it not, to various uncertainties; do you see that?
A. Yes.
Q. And they are not exhaustive, are they? We can see that by the use of the word or the abbreviation "et cetera"?
A. Yes.
Q. What I suggest to you is that these safety factors have been derived to consider, amongst other things, the risks that need to be considered at the design stage? That's correct, isn't it?
A. And the design life stage as well, not just for the design stage.
Q. So you are agreeing with me but adding something, and one of the reasons they have to be considered at the design stage, I suggest, is because the scale of those risks at the design stage is at its greatest; that's correct, isn't it, because of --
A. I do not agree with you, because you do not know what will happen to the structure during the long life of the building, so this positivity factor has to cater for all sorts of conditions, including the design stage, the

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construction stage, and the long life of the building too. This is what it intends to.
Q. Okay. I see your answer there. We've got that. Thank you.
A. Because if you look at this particular Code of Practice, it keeps on talking about the design working life, all the time, everywhere. You look at this -- you go to other pages, they keep on talking about intended design working life, which is 50 years only. This code is only for 50 years. When BD checked the structure, they only checked it based on 50 years' design life. But this particular building, the fit for purpose is 120 years, which is much longer than 50 years.
Q. Well, we've got your answer and you have kindly agreed with me that it includes for the design stage and the construction stage, and I think that will do for my purposes.
A. Okay.
Q. Can I also suggest that the partial load factors which are applied at the design stage reflect also the risks to be encountered during the life of the structure -- I think that's something you would agree with?
A. Yes.
Q. -- but these risks are greatest at the inception design stage because of what Dr Glover refers to as the

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unknowns?
A. I disagree with you here. I disagree, because I tell you, if a designer has uncertainties about -- at the design stage, uncertainty about something, he puts in additional so-called construction load in the design; right? In this document, I checked it, there was no such thing as construction load.
Now, if he is uncertain, he puts in the construction load. After that, he can remove it for the permanent design. But in this case, I have not seen one single mention about construction load, no such thing at all. The structure was designed for dead load, live load, soil load and also water load. That's all. I haven't seen one single word mentioning construction load here.
So I have to disagree.
Q. Well, can I suggest that the safety factors contained in the HKCoP are conservative, to cater for the unknowns and uncertainties that may arise during the construction stage?
A. I disagree that it is conservative. I disagree with you about being conservative. The code is written to cater for uncertainties, no doubt about that, but whether it is conservative or not, I disagree with you. I don't think it is conservative.
Q. Okay.

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A. I think all these things are there to protect the public against failure, so we require them to be there.
Q. Okay. I hear what you say.
Let me try this one on you. Once the construction phase is over and the structure is up, presumably you would accept that the nature and extent of any unknowns and uncertainties that existed at the design stage are reduced?
A. No.
Q. Really?
A. Definitely no. This is a new concept to me, honestly.
Q. So you've built the structure, it's up and running, and you are not prepared to accept from me the proposition which Dr Glover will explain in due course that at that stage the extent of any unknowns and uncertainties that existed at the design stage are reduced? You are not prepared to accept that proposition?
A. I'm not prepared to accept that, because there would be more uncertainties during the long life of the building. I don't agree with you at all.
Q. Very well. Thank you, Dr Lau.
Questioning by THE COMMISSIONERS
COMMISSIONER HANSFORD: I have a couple of questions for Dr Lau.
The first question. Yesterday, Dr Lau, you referred

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to -- twice, actually, in your evidence, you referred to the locking effect.
A. Yes.
COMMISSIONER HANSFORD: What did you mean by the locking effect?
A. If I can write on this board --
COMMISSIONER HANSFORD: You can.
A. -- it would help everybody.
CHAIRMAN: You can.
A. Okay.
(Drawing on the whiteboard) Can you see?
CHAIRMAN: Yes, no problem.
A. We have this diaphragm wall constructed, this is supposed to be what we call a top-down construction; right?
COMMISSIONER HANSFORD: Yes.
A. So they excavate down to here. They construct the slab. So, when it is constructed, the dead load of this slab, which is very heavy, as Prof McQuillan said, 90 per cent of the load comes from the dead load, 90 per cent; right? So there's a lot of fixed-end moment built into these two joints.
Then they excavate downwards until they meet the -- this is EWL, this is NSL -- and then they cast the concrete slabs. Then they come in, to put in all the

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columns.
This particular structure, all the bending moment is there already; right? They take up all the bending moment. So the structure deflects slightly. Then they come in to put in, after this is constructed, to put in the columns and walls. Now, these column and walls will be relied on for the permanent stage. But the point is there is already bending moment locked in in the structure, based on the factor of safety of 1.4, the dead load. This is according to the Hong Kong Code.
Now, the point is, if we keep on using 1.4 for all this locking effect, it is very expensive. Very expensive. So, in the updated design by Atkins, it is considered that it may be easier, when they check it, they assume it is only 1.26, 1.26 rather than 1.4. So in that case, the moment here (indicating) will be less. Then they check 1.4 later on with the column -- with all the columns and wall put in, and they already built in moment there (indicating). It's what we call the locking stress, based on 1.26.
So this is what we call -- and also, don't forget that a lot of stress in the completed structure is based on the water and soil pressure acting on this (indicating). Now, when they analyse this structure, we need to know what we call the stiffness of the soil.

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Now, this point is -- because if the structure is there, all the consultants, OAP or Atkins or some other consultants, when they do the stage 3 assessment, we have to assume certain critical load conditions on this structure. The critical load conditions, according to GEO, you need to have a 5 metre difference in water pressure. Now, if you use -- so all the forces in the structure depends on the stiffness.

Now, Atkins assume the stiffness is E equal to 1 times N. N is the value from the static penetration test during one investigation stage. Then when OAP analyse it based on 1 times N, we have more or less the same stresses in the whole structure, and then OAP, in order to make it more aggressive, they changed it again using E equal to 1.5 N. As soon as you use 1.5 N all the stresses inside the structure will be lower. Then OAP try to keep -- and OAP, Dr Glover, criticised Atkins by being too conservative, because they use a different parameter in the computer model, and this equal to 1 times N is required by government. This is required by Hong Kong government. You have the design based on equal to 1 times N.

So you need to understand the whole thing before you criticise Atkins or -- you cannot criticise Atkins just by using equal to 1.5 N.

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This is what I mean. Do you understand what I mean by locking stress?

COMMISSIONER HANSFORD: Thank you. I shall look back through my notes to see the relevance of it, but thank you.

Sorry, I had two questions. My second question relates to any future recommendations that this Commission may make regarding the use of couplers.

We have seen how important the visual inspection is of the number of threads that are exposed, and I don't want to go into here whether or not that represents a butt-to-butt connection. But my question is: do you think there can be a more fool-proof method of ensuring that a correct connection has been made in this type of coupler for the future use of these couplers, a more positive, deliberate means of ensuring a proper connection?

A. A butt-to-butt requirement is for permanent elongation as well as compression; right? So this is a very important point. So it's important that we tighten it up. If we tighten it up, we can ensure butt-to-butt in this case, so I reckon, if you ask me, I would make sure that the workers will use pipe wrench to tighten up the bar against the coupler, to make sure they are in full contact with each other.

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CHAIRMAN: So that it should be a standard action at the end of screwing it in, to just ensure butt-to-butt by using a wrench to tighten?

A. Maybe. Make sure they apply at least a certain amount of force, maybe a wrench with certain amount of force, to make sure they are in full contact with each other.

This is very important --

COMMISSIONER HANSFORD: Because -- and again I'm looking to the future, not back to what's happened on this job, but of course BOSA don't currently recommend any particular torque to be applied?

A. I agree.

COMMISSIONER HANSFORD: And your suggestion here is that they should?

A. They should.

COMMISSIONER HANSFORD: Thank you.

Re-examination by MR KHAW

MR KHAW: Dr Lau, just perhaps two matters I wish to further discuss with you.

A. Sure.

Q. If we can turn to the transcript of yesterday, page 151, line 11 -- that's Mr Shieh's question to you -- the question was:

"But there are one or two big principles that I want to put to you. Within the EWL slab, none of the

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couplers were subject to a ductility requirement, do you accept that, within the EWL slab?"

Then your answer was:

"You mean according to the drawing or --

Question: According to the drawings.

Answer: According to the drawings, it seems to be the case, yes.

Question: So if that is the case, it would follow that couplers installed in the EWL slab only needed to fulfil the load requirement of 529 megapascals?

Answer: If there is no requirement for moment redistribution, yes, I agree.

Question: No, if there is no requirement of ductility, then according to the documents we have seen from the BD perspective, the test to be reached is 529?

Answer: I agree. When it was originally designed, there was no anticipation of moment redistribution in the original design. It's only in the updated design that moment redistribution was required."

That is just trying to refresh your memory on what was discussed yesterday.

In relation to this ductility requirement, in fact we can just remind ourselves what was actually discussed in part 1 of this Inquiry. If I can just take you to perhaps one small paragraph of the closing submissions

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regarding part 1 of the Inquiry prepared by Mr Pennicott's team. If I can just show you that particular passage. It's paragraph 184.

This is Mr Pennicott's team dealing with Leighton's argument on the ductility requirement. They say:

"Paragraphs 115, 120 ... advance an entirely new point with regard to the non-applicability of the QSP. It is self-evidently an ex post facto argument conceived by Leighton's legal team. It is submitted that the contention is likely to be incorrect. In a nutshell, Leighton seeks to argue that, aside from the D-walls, the QSP only applies to coupler assemblies with a 'ductility requirement' and, in that regard, point to (a) appendix VIII of BD's conditional acceptance letter which refers to 'ductility requirement' and (b) certain drawings which contain the annotation 'ductility zone'. Such drawings only apply to the intersection of the D-wall and the NSL slab at area A. So, it is reasoned [by Leighton], the QSP only applies to that particular area. Whilst the government's and MTR's response to this new contention is awaited, it is pointed out that the QSP itself provides, inter alia, 'For the purpose of this document ... Seisplice type II (ductility coupler -- use in any location).' In other words, the QSP applies to all ductile couplers and not just ductile

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couplers within a ductility zone."

So apparently it is the Commission's legal team's submission that the ductility requirement as specified under the QSP should apply to all ductile couplers in all locations, not just some specified locations.

Did you have a chance to look at this part of the closing submissions by the Commission's legal team?

A. No.

Q. Having seen these submissions, if you look at Mr Shieh's question again, which is about whether, within the EWL slab, none of the couplers were subject to a ductility requirement, what would be your views?

A. Well, if this particular paragraph 184 is correct, then it should be used all over the place then. Then we need ductility coupler even in the EWL slab.

Q. I see.

The next matter that I wish to just very briefly discuss with you -- it's a matter discussed at page 126 of the transcript yesterday. We can start from line 5. Again, Mr Shieh's question:

"How would the poor worker know whether or not, when he couldn't push in any further, it's because it has already reached butt-to-butt or it's because of some misalignment or some mishap that he couldn't push any further? How was he to know?"

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Pausing here, I just want to make clear that I do not wish to discuss butt-to-butt with you. I do not wish to discuss that topic with you any more. Just to make sure that I will not be more unpopular.

If we read further:

"He has to tell his supervisor and let him decide, let the supervisor decide.

Question: But every time he couldn't screw in further he tells his supervisor, but every bar at some stage he would reach a dead end, so every bar he couldn't screw any further he tells his supervisor?"

Then your answer was:

"I tell you, it's not that difficult to fit in the threaded bar into the coupler. It's not as difficult as you said. It's not difficult. I tell you. We are sitting in this courtroom and imagining that it is very difficult, but it's not that difficult ... Most of the workers can put it in quite easily. On my site there's no problem."

In fact this question regarding whether it was a difficult job in fact had been also discussed in this Inquiry. If I could just very briefly refresh our memory by taking you to the evidence from Fang Sheung. It's COI 1, E1/29.3. That's from the representative of Fang Sheung, Mr Pun. Paragraph 7:

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"Although Fang Sheung is only a small-scale company that makes every endeavour for rewards, it is definitely a credible and reputable company ..."

Then if we move on:

"According to the sub-contracting contract, Fang Sheung only provided bar-fixers to work according to the instructions of Leighton, while all the materials were prepared and responsible by Leighton. If the coupler screw cups fixed to the concrete unit were damaged and therefore making it impossible for the screw heads of the steel bar to be fastened, Fang Sheung would only need to notify the site supervisor of Leighton. The bar-fixers of Fang Sheung would never need to figure out the solutions themselves. Under such circumstances, why would it be necessary for Fang Sheung to engage in fraud? Meanwhile, under normal circumstances ... it will only take the workman of Fang Sheung around 20 to 30 seconds to completely twist the steel bars screws onto the screw cups. However, it would take at least 1.5 minutes to 2 minutes to use a portable electric shear to cut short the screw heads of a steel bar during the operation."

Then we also discussed this point with Mr Kit Chan of MTRCL. If we can just have a quick look at his evidence at COI 2, in the transcript of Day 16, page 41.

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Scroll down a little bit. Yes, I think that's the answer of Mr Kit Chan. Can we scroll down a bit? Yes:

"That's the issue. You want to make sure that you have a document -- now, like I say, I would let it slide for ..."

Sorry, I might have got the reference wrong. It should be Day 14 of COI 2, page 41. We can start from page 40, the last bit. That is evidence of Mr Kit Chan:

"Normally, I think my colleague, Victor Tung, I mentioned yesterday, will check it visually, and also randomly use manual check; right? This is a standard practice in the industry. This coupler installation is so easy job, like capping beam, a plastic cap to a dowel bar in a movement joint, a very simple operation, have been in the industry for many, many years. It's just the expectation. What to expect, what kind of expectation from the government for record-keeping? But now everyone knows that the government want more record-keeping for coupler installation, people start preparing all these records now. In the past, probably they don't expect this [to be a] requirement."

So having seen the evidence of Fang Sheung and also the evidence from Mr Kit Chan of MTRCL regarding whether coupler connection was a difficult task -- now, going back to your answer given to us yesterday --

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CHAIRMAN: Sorry, Mr Kit Chan, again, his position was ...?

MR PENNICOTT: Construction manager.

MR KHAW: The construction manager of MTRCL.

CHAIRMAN: Thank you.

MR KHAW: If we can go back to your answer yesterday, when you told us that it was not a difficult job in answer to Mr Shieh's question, can you tell us why you said that?

A. I don't think there's any big problem in general. Well, maybe one or two bars they have difficulty, but in general I don't think it's difficult to screw in the bar. I don't think it is. They can do it -- I'm not aware of any big difficulty in the screwing in of the bar on my site at all. But anyway, I think I agree with the sub-contractor's view that it is not difficult to screw in the bar too.

CHAIRMAN: No, I don't think we have taken it as the overwhelming evidence that it is difficult in itself. It's not a highly technical, complex operation requiring particular agility or anything of that kind. What we've taken it as being is a fairly -- in terms of engineering construction, a fairly mundane job, but nevertheless you are using heavy materials, you are often using them in cramped circumstances, and not always is every coupler set at correct right angles. Not always are they clean. And there may be occasions when they are even damaged.

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And all of these things will, from time to time, in a day's work, present the team with something less than an easy job.

A. Yes. Maybe one or two bars but in general I don't think it's difficult. In general, I don't think it is.

CHAIRMAN: Right.

MR KHAW: Lastly, just one point regarding the answer you gave this morning, as recorded in the transcript. If I can just take you to -- well, I can read from [draft] line 18:18 of the transcript, where you said:

"Then what you said can be useful because we want to find out what caused the collapse and we do a lot of investigation, coring of the building, to find out the [conversion] strength of the concrete and do it in the back analysis, to see what happened."

Did you mean to say "conversion strength" or other strength here?

A. Concrete strength, I think. The concrete strength of the concrete. What did I say, sorry? Let me have a look.

I think I mean the current strength of the concrete. I believe that's what I said, "current strength of the concrete".

MR KHAW: Thank you. I have no further questions.

MR BOULDING: Sir, I've now got Dr Mike Glover here, who is

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raring to go, but I see the time. You may well want to take the coffee break and perhaps peruse his presentation, which has been delivered, I trust, to you this morning.

CHAIRMAN: I haven't seen it yet.

MR PENNICOTT: We received it a little earlier but I don't think it's reached you yet.

CHAIRMAN: That's fine, in which case a coffee break will be ideal now. Thank you.

MR PENNICOTT: Sir, before we do break for coffee, may I raise one point, because it may be convenient to deal with it now, before we start with Dr Glover.

You will be aware that in the last couple of days, two further witness statements have been served on behalf of Leighton, both from Mr Man Sze Ho who we have heard from previously. So we've got now his third and fourth witness statements. His third witness statement deals with certain of the photographs that were referred to in Mr Southward's presentation, and his fourth witness statement, which I understand the Commission has given permission to put in, as it were, this morning, deals with the method statements and the difference between the two method statements that we were looking at last week.

So far as the Commission's legal team is concerned,

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we do not desire Mr Man Sze Ho to come along to give evidence, live evidence, as it were, about either of those two witness statements. If any of the other parties take a different view about that, then obviously they need to speak up relatively quickly so that we can arrange for Mr Man Sze Ho to come along. As I say, from the Commission's legal team's point of view, we don't require that. As I say, I'm giving everybody else the opportunity, should they wish to take it.

CHAIRMAN: Good. Very well, gentlemen. Thank you. 15 minutes.

(The witness was released)

(11.30 am)

(A short adjournment)

(11.52 am)

MR BOULDING: Good morning again, Chairman and Prof Hansford. I'm now calling Dr Mike Glover, and I assume, as with the project management experts last October, he will be regarded as still on oath from giving evidence last time.

CHAIRMAN: Yes, absolutely.

DR MIKE GLOVER (on former oath)

Examination-in-chief by MR BOULDING

MR BOULDING: But perhaps, Dr Glover, you can remind the learned Commissioners of your name and professional

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address?

A. Yes. I'm Mike Glover and I'm a structural engineering expert on behalf of MTR.

Q. Do I understand that you prefer to give at least your presentation standing?

A. I would prefer that. I've got a slight cold and cough and I feel if I stand up I won't cough quite so much. Not only that, I want to use the board.

CHAIRMAN: In which case, please stand up.

A. So if I suddenly go into convulsions, you'll understand; you've been alerted to it.

MR BOULDING: Before you start coughing, Dr Glover, we have one or two formalities to go through. Perhaps I can take you to your Original Inquiry structural engineering report which I hope we will find at ER2/16.1. Yes, there we are. Is that the first page of your Original Inquiry report dated 6 December 2019?

A. Correct.

Q. Could we go on, please, to page 1634. There do we see your signature?

A. That's correct.

Q. Subject to any amendments in the joint statement and the supplemental joint statement, first of all, are the views and opinions expressed therein views and opinions which you honestly held?

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A. They are.

Q. Insofar as you refer to facts, are they facts which you honestly believe to be true?

A. Correct.

Q. You've prepared a second report, the Extended Inquiry report, and I hope we'll find the first page at ER1/12. Yes, splendid. Is that the first page of your report, again bearing the date of 6 December 2019?

A. It is.

Q. If we go on to page 17, there do we see again your signature above the date of 6 December 2019?

A. That's my signature.

Q. And again, subject to any amendments in the joint statement and the supplemental joint statement, are the views and opinions set out therein views and opinions which you honestly held?

A. Yes.

Q. Insofar as you recite facts, are they facts which you believe to be true?

A. Correct.

Q. I'd like to go on to the joint memorandum dated 12 December 2019. That's ER2/18.1/1. There do we see a manuscript note. Who took that note?

A. Prof McQuillan, and you can see I've signed it on the last page.

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Q. That's correct. We can see it was the meeting of 20 December 2019; correct?

A. That's correct.

Q. If we go on to page 5, there do we see your signature?

A. Yes.

Q. I think there are other documents confirming that it's been signed by Nick Southward and Dr Lau, but do the agreements and indeed disagreements set out therein accurately represent what was agreed or disagreed between you and your fellow experts?

A. They do.

Q. Finally, the supplemental memorandum of agreement dated 2 January -- I think we get that at ER2/19.2, and there do we see a supplemental memorandum of agreement signed some four or five days ago?

Perhaps we could scroll down, and there we see everyone's signature apart from Mr James Lau, but I think I've seen his signature somewhere else. But in any event, does the --

A. It is there, just by signature. It's the squiggle at the end there.

Q. Splendid. Do the contents of that supplemental memorandum represent agreements and disagreements between you and your fellow experts?

A. They do.

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MR BOULDING: Thank you very much. You know the procedure from last time but as you've explained, you'd like to make your presentation and I'd invite you to do that now. Thank you very much.

Presentation by DR GLOVER

A. Thank you very much indeed. I'm afraid my presentation is going to be a bit longer than I intended. The reason for that is as this hearing has progressed, there seem to have been a number of matters that have arisen that really do need to be clarified and explained as best as I can to the Commission, and put to bed, so to speak, because there are a lot of issues just hovering around, and as far as I'm concerned they haven't got a home and they should have a home. So I've done my best to do that in what follows.

To start with, though, I've got to pick up on the discussion that Dr Lau had at the end about risk and load factors and safety factors. I'll go into some detail in this in my presentation. But I want to hit that one head-on because I don't want there to be any misunderstanding in the terminologies that one uses.

(Writing on the whiteboard) If you imagine at the start of a project, inception, you have a list of risks, things you have to consider, and you write them down under various headings like "Design", you look at

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"Operations" -- I'm sorry about my writing; I'm not that good on these things -- and in here, a critical one is "Construction", and coming out of all these things is a whole list of knowns and unknowns.

You do your absolute best to actually establish that. We carry out research as much as we can on the unknowns. Sometimes, to quote an American diplomat, you have unknown unknowns, but you go as far as you can.

CHAIRMAN: Can you tell me what an unknown would -- give me an example because ...

A. An unknown would be, for example, if you have never worked on a particular location, and therefore there is no precedent in terms of the site conditions, et cetera. That is a classic situation where you have to do everything from first principles.

Something which is known but there are unknowns would be here in Hong Kong, in the sense that we know what the general succession of soils are, but we don't know precisely where the rockhead is or other issues, and we know they vary.

So the way we approach a project is we do this list and we actually analyse them -- well, hopefully all organisations do this -- because we want to know knowns, we want to know the unknowns, so we can embark on the right research projects at the right time, so we have

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the information when we need it. Many projects fail because that particular operation is not carried out at the beginning of a project, in the thoroughness that it should be.

But the thing I want to draw attention to, and this is where the misunderstanding between Dr Lau and I I think arises -- it's all in words -- that list goes right the way through to the final demolition of the building. It goes all the way through. So all of these issues are considered.

The point I'm making is a great chunk of those are in that stage there, and when you've gone through the construction stage, those risks have been removed or at best, or worst, mitigated. In other words, you know what you are dealing with. I've spent my life designing things and getting them built, and design is the worst stage of all in terms of being able to have certainty, because the only thing that you are sure of: there are many things that you won't know, and many of them arise during the construction sequence.

This issue about designed load factors and lock-in, I'm not going back to that. It would take too long to explain. I don't agree with the 1.26 factor, it should be much less than that, but I'm not going into that. Those were some of the discussions that we had.

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But the point I wanted to get across with this at the outset is: we consider all the risks, right the way through to the demolition of the construction. A big chunk of those are during construction, and you've got a much better picture of what's before you after construction than you did before. So I would say I can't agree with an answer that says "I don't know the situation better after construction than I did before". You know, it doesn't make sense. It's not common sense to say that.

COMMISSIONER HANSFORD: But the main point was there are still unknowns after construction?

A. Absolutely, and this list goes on, but they are less. You've taken out a whole lot of the risks. That's the point. And at the beginning, that's why I say the risk profile is greatest at the design stage because once you've designed it you've had to compensate for all these things already. It's no good something cropping up here (indicating) if you hadn't thought about it. The design is there, it's constructed.

I can't explain it any better than that. I think I'll return to my presentation.

CHAIRMAN: Yes.

A. Thank you.

Could I have the first slide, please. Just to

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re-state my position, I believe that the structures are safe and they possess a satisfactory reserve of strength. I can go into much more detail on that statement if you wish, but I will develop that.

The structures are, on a structural integrity and performance basis, fit for purpose in that they are -- picking up some of the words that Dr Lau uses -- stable, robust and they are durable.

Both Prof McQuillan and Mr Southward are also of that view, and that's as stated in the joint experts' statement that Mr Boulding referred to. Dr Lau does not agree and has reservations. I say "reservations" because he does not disagree carte blanche, he has specific reservations, and I'd like to deal with them as I go through the presentation, to see to what extent I have properly addressed them, and I'm clearly open, through cross-examination, for clarification on that.

My opinions are not based on considerations of code or contract or statutory requirements. They are just simply my engineering appraisal of the information and data that I have before me.

I would like then to continue with --

COMMISSIONER HANSFORD: Sorry, you had a final sentence there which I think is also rather important.

A. Okay. The structure can be considered safe and fit for

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purpose but it doesn't have to be fully compliant with either the code, the contract or any other statutory instrument. It's a demonstration of physical laws and tests and investigations. It's evidence, basically.

So, put another way, my approach is an evidence-based approach. I try, in my evidence that I am giving to you, not to rely on hearsay or "I feel it's all right". I try to deal with some quantitative facts.

That is one of the problems of the hearing over the last days: there has been a hell of a lot of qualitative statements made, and I think for the layperson that must be virtually impossible to come to terms with, because there's nothing tangible to hold onto. It's, "I feel it's okay." Well, that's not good enough, in my book.

So if I then move on to the second slide, which is the engineering assessment. I want to emphasise these points because this is really a principle of approach. The first thing is it is a forensic analysis. By definition it is. This is not a design exercise. I'm looking at -- not complete, because people will misunderstand my statement -- an as-constructed form, and I'm looking at it dimensionally, I'm looking at it in terms of its material properties, and just the general loadings, et cetera. So that's where I'm coming

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from.

I've already made the point in the second bullet point, which is that the situation at the inception and the design stage is very different from that where you've got all these unknowns and uncertainties I was describing, and these all have to be allowed for, as a designer, at the outset. It's too late thinking about it when the thing is constructed, as I think you will see. But in the post-construction stage, many of these unknowns and uncertainties become knowns and certainties, and they provide a more confident basis for evaluating the safety and performance of the structure, particularly regarding its loading and its materials. I would also add its geometry. Geometry is very, very important in such a very large structure as this.

In addition -- and this is really why we do have the benefit of -- this enormous amount of data that's been produced from the extensive situation and surveys made on the Hung Hom Station, I wouldn't say it's without precedent but it is something which is beyond the normal, and I've taken advantage of that quantity and scope of the investigations in the evidence I will give.

If I could just move to the next slide, please. These are some of the items I've slipped in, as it were. They wouldn't have formed part of my original

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presentation. But because of some of the issues that have been raised, I thought I wanted to do my best to try to help clarify them.

So I will take them each in turn. Could I have the next slide, please. Conceptually, stability and robustness are very difficult to describe, actually, so I've tried it this way. On the left-hand side you have a ball sitting in the bottom of a valley. It's stable. Then on the right-hand side you have a ball standing at the top of a hill. It's stable. The one on the left-hand side, if you were to give it a slight nudge one way or the other, it would come back to a position of stability. However, if you push the ball on the right-hand side, just a small nudge makes it fall into the abyss, and that is the difference. A structure can be, to all intents and purposes, very, very stable, but is it susceptible to a disproportionate collapse situation for a very small input? And that's what robustness is about. It's about providing that provision that says if something is not there or stability is not in place, what compensates for it.

So that's my crude attempt to try to get across to you how you can have stability but perhaps not robustness.

Could I have the next slide, please. The other

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thing I wanted to try to help the Commission with is -- lots of numbers have been bandied about testing for this stress and that stress. This is just a straightforward stress-strain relationship, interestingly enough of a coupled connection, but I won't go into that. That will open up Pandora's box. Just call it a reinforcement bar.

One of the discussions that took place the other day, yesterday I think, was the difference between the stresses that a coupler is tested to, whether it's ductile or not ductile. The non-ductile one is I believe tested to about 520 somethings, and you can see that's that little blip just below there, and that's to do with -- you can see the line is virtually straight from the origin.

You see, I think, that there (indicating), this line here (indicating) is virtually straight, we call it linear, and so that test that's carried out on the non-ductile coupler is really testing its elastic response. But when it's ductile, the coupler has to have a degree of plasticity and so therefore it has to be able to stretch to this point.

So that's really the difference between those two tests. The first test is a test against linearity and the second test is to see to what extent it has

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plasticity. Quite straightforward.

But we are not designing at anywhere near those stress levels. As you can see from here, you continue pulling the bar and it keeps going to in excess of 650, as a number. Our design is based on two limit states. The first limit state is meant to represent collapse, as it were, and that's what we call the ultimate strength. You can see the stress we are using in our design of 400 is well below any of the figures previously. Now, I wouldn't want to change the codes, don't misunderstand me, but I'm just trying to give you a feeling for the margins that we have in the materials that we use, and absolutely correctly.

But the working stress that we would design a structure to would be this other number down here (indicating), 260 or thereabouts, 200, that sort of number. If we were designing to that level, if the whole structure was at that stress level, it would be designed to its optimum level. But life isn't like that, as we've seen with this particular project, and in fact what we are doing with this project is we're down here (indicating), 130 or less.

So when you start to actually add up all of these factors in terms of the safety of the structure and the strength reserves in it, it's quite enormous, and when

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we get into discussions about cracking and deformation, all the standards on cracking and deformation are based around 260. So when you are working at 130 or less, then the opportunities or the chances of such things like large amounts of deformation, vibration, cracking, are massively reduced, as evidenced when you walk around the structure.

So I hope that slide has put some of those things into perspective.

Could we have the next slide, please. Right. Oh, now, I think you should hold on to your seat with this one, judging by the conversations that have taken place already. So I think we should pause for a moment and just get ready for it. I've got two slides on concrete strength and I will go to the board for a third one, really.

Basically, this figure -- I have taken 40, for example, which is the design strength for the EWL slab. It's a look-up table, as you saw from one of the presentations -- I think Mr Khaw showed it -- it's a look-up table. It doesn't come out of some experimental mix designs we do and then we smash the cubes and do whatever. For example, if I was carrying out an experiment on something, then what I would do is I would make my concrete, I would make a sufficient

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number of cubes, which I would test, that would help me then to understand what the strength of the concrete was in the model that I had crushed or broken or whatever. In that case, I would analyse those cubes and I would use either the mean value, which is the average of the scatter, of the distribution, or, if I was looking at an extreme design position, I would be looking at what's called the 95 per cent passing level.

That's what I would do in a laboratory, but what has happened with the standards that have been drawn up, it's basically a "deemed to satisfy" situation. You look in the code, you look at this particular strength, and there is, on another manual, the mix design that will achieve that. Some check cubes are taken to demonstrate that that strength is achieved. Indeed, on this project, those test cubes were well above anything that you would have expected.

So the 40 is not a number which has been derived by any tests or experiments as part of this project. It's a look-up table. Then, thereafter, you carry out -- and Dr Lau is correct, that we carry out a whole series of cube tests, I think on site there were probably 6,000 plus, and that's what my big distribution curve next to it is, where I've got "Actual" is meant to --

COMMISSIONER HANSFORD: Sorry, can you point to it?

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A. Sorry, yes. That's what you would expect to get from the design, if you tested them, and this one here (indicating) is what was found, shall we say, with the cube strengths on site. You can see that conservatively they arrived at a 90 percentile of 60.

The mean, this line ghosted in there, is more like about 85 or thereabouts, about that order.

All I'm explaining here is that's a wealth of knowledge and it seems a bit churlish to say it doesn't exist, particularly when you think about the 40 as look-up table, just by -- you know, it's a conservative figure, and again I'm not fighting against conservatism, I'm fighting against inappropriate conservatism, because that costs money and it costs resources and it costs a waste of endeavour.

So that's a very crude explanation between the difference between design and actual.

Can I have the next slide, please. I do seem to be on a collision course with Dr Lau -- I've been in and out of Hong Kong for a long while, and the concrete technology 50 years ago is very, very different from what it is now. I mean, MTR -- and to draw a comparison between 50 years both in terms of its quality control, even its chemical constituents, and today is totally inappropriate. Sorry, I should have left out the word

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"totally"; it's inappropriate.

Interestingly enough, particularly since the turn of the century, probably a little bit before that, MTR have led the way in mix design in concrete in Hong Kong, and they and other organisations and government, in this respect, have gone a long way to actually changing the mix designs that existed 50 years ago, and one of the major -- I mean, we use admixtures quite a lot -- the honeycombing, for example, was a failure to use the proper admixture and the right aggregate design. I'm not going into that. It's got nothing to do with strength; that's to do with workmanship. But one of the major ingredients in a modern concrete is the addition of what I've referred to there as a pozzolanic material. This is Roman concrete. It's a totally different chemical composition. I think I'm correct on this: a minimum of 25 per cent of modern concretes in Hong Kong contain this material. In fact it's referred to as a PFA which is pulverised fuel ash. It comes as a waste product from power stations, which is quite incredible, really. Maybe we are already in the cyclic economy; who knows?

But what this does is, pozzolanic materials have a slow gain of strength with time, so it's a mix -- and when I say modern concretes are a mixture of materials,

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you've got this composite material which is achieving different objectives, early strength on the one hand and then this slow, relentless increase in strength with time with the pozzolanic materials. That wasn't the case 50 years ago.

I won't go into any more depth on that, I will just go into the next point, which is historically, in terms of the quality of construction in Hong Kong, there is a massive difference between buildings and infrastructure projects. And that gap has closed, in fact it's now the same. But if you were to look back 50 years ago or even less than 50 years ago, you would be astonished at the working practices in a lot of the construction projects for buildings, not in infrastructure, because the controls were much, much more rigorous. In fact it's best that I don't say too much more about that, in other words, because it is a matter of fact and documentation that there has been this situation.

There were some very, very good building contractors. I wouldn't want to group everybody together. Some of the projects we deal with, Hopewell Construction, for example, were magnificent and whatever, so I wouldn't want to tar everybody with the same brush.

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But the fact is concrete technology has changed over 50 years, the quality of workmanship has increased over those years, and the net result of that is: it's wrong to look at things which are that old and say, "Because of that, I now have this situation." It's not true.

This is where I'm going to have to go to the board, with this next bullet point, because there seemed to be a lack of clarity on how the factors which control our design have been evolved, so I'll try to describe my third paragraph by going to the board.

(Writing on the whiteboard) Thank you very much. Let's call it f for strength in situ. Now, we know that the strength in situ is not going to be the same as a cube or -- I'll refer to a cylinder to start with, because cylinders are used more regularly through the world than cubes. One is not superior to the other, it's just practice. But the relationship between f in situ and the design is 0.85, and that's cylinder, and that's used -- in all of the American Codes, for example, you'll find that relationship.

So there's already a reduction factor taking place in terms of you've got the cylinder strength. You don't say it's the same as that. You say it's 0.85. And that's been derived from lots of research over many, many years. This is not new. This is at the heart of

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all of the ACI codes, for example, that simple statement.

To get to cubes, there is a 0.8 factor, and I'll explain why there's a 0.8 factor. That then arrives at 0.67 fcu. That's a cylinder. Now, the 0.8 factor is because of the shape of the specimen. You take a cylinder and you put it in a testing machine and you get a number. If you put a cube in the same testing machine, it's stronger, and the reason for that is because it's a square and the testing regime. But this relationship of 0.8 has been established over the years. So that's why we use that. There's no black magic. That's why we use 0.67.

That's on fcu and we've just had the discussion. That's the design strength. Now, if you take my hypothesis that we've got 6,000-plus cubes out there as well as what I would call working practice elsewhere in the world, then that fcu has now gone from 40 to 60. And the net result of that is that I should really be using something which is much less than this, in fact two-thirds of that. So I'm already now down to something much, much less than that figure.

Now, not satisfied with 0.67, we apply a material reduction factor of 1 over 1.5. So when we design a structure, we don't use 0.67. We use 0.45 fcu.

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That's why you see that. So when we do a design of a column, shall we say, the maximum stress we take is 0.45 times fcu which is the design strength. But if you then say: I've got -- I'm 40 down to 60, then, if I was to use the 60 strength, I would be down at 0.3 fcu, but this time that would be 60. And if I then say, "Actually, I've got a lot of pozzolanic material here and it's cooking away and fantastic and I will just use one of the many growth curves of strength with time", like in the Eurocode, this would be down to 0.2.

So, as I did with the steelwork, the metal, I'm just trying to describe to you the levels of safety that we've built into our structures is absolutely enormous. Now, I'm not standing here advocating massive change to this. I'm just trying to put to the Commission just the sheer arithmetic of what we're talking about. We are not talking about things teetering on the brink. We're talking about modern materials and we are talking about high levels of understanding of the structural mechanics behind it.

I think I've done that.

(Returning to the witness box) I would also want to add two things and I'm not sure how to deal with these, in what order. I think I'll deal with the cylinder strength first.

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Dr Lau referred to he would have great confidence or greater confidence in our hypothesis of increased strength if there had been cylinders taken and tested.

COMMISSIONER HANSFORD: Cores.

A. Yes, cores, but the cylinders.

I tried to find some for the structure, but unfortunately -- or fortunately, whichever way you look at it -- the cube strengths were always so high that nobody had to go back and do some investigations and do some corings. But we are fortunate in the sense that the standard regulations in Hong Kong require diaphragm walls to be cored, to ensure that we have this vertical core of concrete all the way through.

So we have lots and lots of cube strengths, core strengths, for this project, not in the EWL slab, not in the NSL, but in the diaphragm wall. And these are summarised, I think, in a number of the reports but particularly in the AECOM report, and in the AECOM expert report, they do refer to these cores, and with no surprise as far as I'm concerned they show a mean strength of about 79 and a characteristic strength -- in other words, the 5 per cent -- of 62.

Now, the mix design for the diaphragm wall is slightly different from that which is in the EWL slab, but the fact is they are very, very similar. I don't

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know, I haven't been able to find out whether the ready mix supplier was the same supplier or not, but the same regime applies throughout, so if we are looking for, I would say, not a smoking gun in this case but a sort of golden bullet, there you are. There is the evidence on this site that you have a design mix, which in that case was 45 which had to be reduced by a factor of 0.8 because of tremying effects. So you've basically got a design strength of 36 and we're getting cube strengths at 95 per cent passing of 62, and a mean of 79 to 80. I mean, my case rests, really, in the sense of doing a forensic analysis, I emphasise this, I'm not extrapolating this and saying this is what you've got to do in Hong Kong. I'm saying, for this project, I'm more than satisfied that the strength in the structure is at least 60 and with an age factor applied to it now of about three or four years which is quite considerable.

I hope that gives you a better feeling for some of the language that's been used and some of the evidence that perhaps hasn't been presented to you in the way that it could have been.

If I could then move on to, I think -- now, this one I will forgive people for sort of glazing over slightly, but just so you start to absorb that diagram, I want to pick up a few points which have been made generally

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about the analysis of the structure.

A lot has been talked about about the updated design, and in fact there's a reference to the consultants recommending the updated design. Well, that is not the case. We did not recommend that design or the parameters that were used. We were asked to consider what the effect of those parameters would be in the design of the structure. Indeed, MTR recognised that and they allowed -- not "allowed"; crumbs, it's our reports -- in our reports, they did not object to us bringing attention to some of the areas where we believe there was very large -- not "large", sorry -- there was conservatism beyond the level which we would think was appropriate.

Interestingly enough, again, despite what has been said by some of the presenters, Atkins are of the same view, and if you look at Atkins' reports, I think it's in section 16 but I stand to be corrected, they list out a whole series of the designs or aspects of the updated design which they considered to be conservative.

So the idea that all the consultants got together and said "This is what we've got to do" is incorrect. We agree with some of the parameters. We don't necessarily agree with all of them. And this point about the soils that Dr Lau brought up, about N equals

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1.5, indeed the Commission has a report that Arup produced back in September of last year where we analysed all the available data from the site -- in other words, I'm not going by hearsay or whatever, I'm just taking the sheer data and we worked through it -- and we found that on this site, looking at the performance of the walls, they hardly moved, actually, during construction, that you would be looking at E equals much larger than 1.5. But we said, "Okay, we'll go with 1.5 if that settles everything", but no, that wasn't good enough; it had to be 1.

All I'm saying to you is those parameters were not recommended by us. In our reports, we do draw attention to the fact that we do think they are conservative. I wouldn't want you to run away with the idea that they are massively, massively conservative, but they are conservative. So I don't want the Commission to believe that the updated design is something we said, "Yes, you've got to have this."

If you wanted a parallel, I would say it was much more to do with a compliance design, in other words to try to demonstrate that the structure was compliant, and I have no problem with that at all, if that's what the objective was. But it's not my objective for this Commission. My objective for this Commission is to

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bring to your attention why I think the structure is safe and why I think it's fit for its purpose.

I'm sorry about that long speech but I wanted to make that very clear because it could be misunderstood, and I would refer you to the Arup reports and to the Atkins report for reference purposes, if you wanted to get the essence of where we believe the design or the updated design is conservative.

The second thing I want to build on before I delve into the wonderment of partial safety factors is this issue of factors of safety. I've just got to collect my thoughts slightly here. Dr Lau referred to his research in the early 1970s on soils with Prof Nash. Interestingly enough, obviously I was in London at the same time and I was working with another professor, Prof Henkel from Imperial College. King's College was really at the forefront of geotechnical design but so was Imperial and I worked with Prof Henkel, and it was the genesis of a number of non-linear analyses, there's no argument about that. We were more interested in London clay and other people were interested in -- so I do agree with the observation that a lot of the basic research that was done in the early 1970s actually -- that was its genesis.

But then Dr Lau goes on to sort of extrapolate to

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the modern future and refers to software codes like FLAC, et cetera, but these are all geotechnical ones and we would never use those for structural design. They are brilliant, I use FLAC, for example if I'm tunnelling through chalk into soft rock, then it's very good.

So it's a question of appropriateness of the software you use. I think that's possibly what he was trying to get across and I would agree with that 100 per cent and I will show you some of the non-linear analysis that we've done using what I would call an appropriate software system later.

But the other point that he made was about factors of safety being a local consideration, and I would embrace that, particular when it comes to soils, because the soil here in Hong Kong is not the same as the soil in my back garden, and so therefore the rules that you build up for soils -- and I'm referring now to Dr Lau's statement about slope angles in China and Hong Kong -- they are local considerations. But when you are coming to concrete and steel, they are international. You know, you pick up one code and the language might be slightly different but the essence is exactly the same. If I remember correctly, Mr Southward, when asked a very similar question, answered it in a way that I think is probably the best way of responding and that is, if he

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designed a bridge in America or here or anywhere else in what I would call the developed world, then it would be the same. You know, society doesn't, when it comes to things like concrete and steel, it doesn't have a different view, because there is an international consensus as to what it is.

So it is -- excuse my frustration but it's an illogicality because all the evidence is there before you. There are always aspects which they've got this little nuance here or this nuance there, particularly if you are in a highly seismic area, then there would be special rules, and that would be local. But not concrete, not steel, not at its essence.

So that gently brings me on to partial factors. I knew you couldn't wait; that's why I left it there. If I could try to help you with this. Gamma F, the one that's on the far right of the slide, the top one, that is what we call a load factor. So, in other words, let's say we've got a loading of 100 pounds per square foot and that's what we think it's going to be, we would apply a load factor to that of, let's say, 1.6, as an extreme ultimate value. So that's gamma F.

The figure below, gamma M, is the factor that we apply to reduce the strength of materials. If you remember here, this 1.5.

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Internationally, numbers like 1.5 on gamma M and gamma F, 1.4 to 1.5, that's what you end up for what I would say is a standard situation. But when you are looking at a forensic situation, you go back, so you go back to the second row, and you ask yourself questions about what makes up these factors.

Now, the first one is the uncertainty in representative values of actions. This diagram is a direct lift from the Eurocodes. This is not something I've created. This is a figure C3, so it's in there.

The first box in the first column is to do with dimensions, primarily. In other words, how thick is your slab. The second one in that column is to do with the analysis method: have you modelled it correctly? Is the length of the beams correct? Have you got the stiffnesses correct? And clearly, you are not going to get it absolutely right.

Just to give you an indication, on the first one, the first box, at the top, it's normally what I would call a standard, no-thinking type of project. It would be 1.15; yes? Now, that would mean that if I had a slab 200 millimetres thick, my calculations could allow that, if I was to look at it after the event, it could be 30 millimetres more, but that's what it's to take account of, the variability. It's also, interestingly

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enough, to take account of the construction loads that would have taken place and are no longer there. So the idea that you haven't actually written in "Allow for this construction load" is a load of rubbish because it's actually already included in the load factors. It comes in and then it goes out again, used for something else.

But if you then take a 3 metre thick slab and you say it's going to be 15 per cent thicker, 450 millimetres thickness of concrete, you've got to say: why? Why have I got to do that? For construction, most certainly, you've got to have a very, very robust load factor. You've got to make sure you've got stuff in there because the contractor might do something wilful, you know.

But then when you sit back afterwards and you measure the thing and you find it's only -- I think the surveys have shown -- 20 millimetres more than 3,000 millimetres thick, in other words, the variation, you've got to start saying to yourself: why is the design carrying this?

In the forensic codes, and there's an excellent one based in the UK which is the appraisal of existing structures, it addresses this issue and it says: look, taking it down to 1 is a bit silly, really. You've got

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other things in the future to take account of in weight, for example. So it talks about a variation between 1.15 and 1.05; right? Small differences, I know, but actually, when you're talking about something as massive as this and you're worrying about fine judgments, it matters.

The second factor is generally, in the second box down, that's generally taken as 1.2 and it's an ignorance factor. The thing I find most astonishing is it's 1.2, even if you did the calculation on the back of an envelope, or if you use the sophisticated tools that we use. It's exactly the same.

So you can see, just talking about those two boxes, that there's plenty of room to actually sit back and say to yourself, "Hang on a second, these were appropriate for the design stage because I had all these unknowns, but I'm looking forward now to the forensic situation. Is it really sensible to judge it on those bases?"

I've got to emphasise, in my appraisal of the structure, I've not taken advantage of any of this. I'm just telling you again, just as did with the reinforcement and I did with the concrete and I'm showing you now, what the margins of safety are that we are dealing with.

Indeed, when we go down to the second one -- and

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I will spend less time on this because I can see that I've indulged myself enough -- the two boxes here, the first one, or the third one down, is really to do with things like: have I got the right level of cover, is the reinforcement in the right position? And the bottom one is to do with: have I got a rogue concrete mixer out there and has he really put some really bad stuff in? But the net effect of that is the Eurocode now will allow you to vary the gamma M, the most remote figure on the right-hand side, the 1.5, to 1.3 to 1.4, most certainly in a forensic situation, on the basis of the evidence of quality control.

Now, there is no way I am going to say we should be applying 1.3 or 1.4 on this project, but I'm just saying to you that they are not cast in stone, and the reference is not me making these things up. These all come from international, recognised sources of information, which I think on any other project I would certainly bring to bear.

I hope I've still got your attention after that. That was a bit of a battle.

COMMISSIONER HANSFORD: It was very interesting.

A. If I may move on then to the next slide, please. I said I would talk about structures 50 years ago or even now. On the left-hand side, there's what I would call

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a classic -- and this is not meant to represent any particular building, but that's the classic design that even today would be constructed, and that is you are dealing with a superficial deposit of reclaimed land or silt or sand, or marine deposits, and that's sitting above CDG, which is completely decomposed granite, sometimes it's MDG, which is mediumly decomposed granite. But basically it's decomposed granite which increases quite substantially with depth. In other words, the deeper you go, the soil gets much stiffer. Then finally you hit the rock and, as has been mentioned earlier, it's not as I've drawn it, like a nice flat plane; it undulates because it's decomposing.

So when you are dealing with a situation as on the left-hand side, the groundwater level in a lot of these areas is quite high, and therefore to build very, very substantial basements was not something that was done. You try to avoid that as much as possible.

As a consequence, you tend to put the building -- perch it, and you would pile the foundations because there's no way you would found on that sort of material, because it's loose. In engineering terms, it's what we call under-consolidated; it hasn't been consolidated. So you would use piles, which are very rigid. You would put a pile cap and you'd do the same with the basement

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slab because the last thing you want to do is go down to get your car in the car park and find the thing has gone into a hole, so you design it suspended.

But what follows from that is the material underneath is extremely soft and loose, for various reasons. One of them is just the natural consolidation of the soil, because it's been loose-tipped, or just over time soils compact more and more, if they have not been consolidated, or, as Dr Lau referred to, you get fluctuations of water pressure which changes what's called the effective stress, and these cause the material to move away from the thing you constructed. I would be amazed if it was otherwise.

But that's not what we've got. We've got, on the right-hand side, a big station box which is well into the CDG. The level difference between the rock and the bottom slab is measured in a few metres. The water is almost at ground level. The water -- to form that slab at the lowest level, the contractor had to dewater all the way down to the slab. He didn't employ divers. He formed the diaphragm walls, he dug down, he took the water level all the way down to the bottom of that slab, and then he cast the slab after preparing it.

This is not a cowboy situation. We've got slides, we've got photographs which describe what's happening.

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Now, that soil has already been dewatered, so during that process, if there had been any issue of consolidation, it would have taken place. I mean, this material is over-consolidated, in the sense that it's got a lot of load on it. It doesn't really want to go anywhere anyway. Taking the water out would have increased the effective stress to some extent, but not greatly.

So the idea that there's going to be a great chasm forming underneath this is just -- you couldn't give it house room.

So on the left-hand side I agree 100 per cent and I would give the highest level of caution. But what I worry about, and I've said this in my original hearing, it's the extrapolation of situations like you find on the left-hand side where it is absolutely correct to do that -- it's the extrapolation to everything else. It's this almost unthinking about these decision-making, "I did it here, I'm going to do it there and I'm going to do it there", without the standing back and saying, "What is the physics of what I'm dealing with?"

So I hope that addresses this issue of voids under slabs and whatever, for this particular project. I don't want anything that I'm saying to be suggesting

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that there should be changes in codes or whatever. If I would make any plea at all, it is to engage a bit more thinking on some of these very, very peculiar and odd situations. This is not normal, what we have at Hung Hom, either in geometry or scale, and it should be looked at differently.

Next slide, please. I think this one has gone away but it came up and I thought, "Oh God, I can see this one is going to run around the block a few times", so I just wanted to tell you what we've done.

Dynamic behaviour was considered as part of the stage 3 assessments. We carried out dynamic analysis of the seismic loading, we actually put a sort of seismic input into bedrock and shook our structure about, and we found negligible resonance in anything. You can put this down to a number of factors and I won't go into another lecture on this, but the large mass and damping of the structure has a very large effect on that, and the idea that running trains would have any effect whatsoever is -- well, it's not even remote. It's just not a consideration, really. So fatigue is not an issue on this structure.

Thank you. The next slide, please. Right. Now, this is the beginning of the presentation I intended to give. So I do apologise for the amount of time this is

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taking, but I thought it was important to go through those issues because they seemed to be hovering around.

COMMISSIONER HANSFORD: It is very important. Thank you.

A. Let's go to these three areas then. So we've got the three areas which I think have been well rehearsed now so I'll move on to the next slide, please.

MR BOULDING: Sir, I hesitate to intervene, but if he's going on to the main part of his presentation -- I see it's about two or three minutes to 1.00 -- you may well think that this is an appropriate break.

CHAIRMAN: It's a good cut-off point, I agree.

Good. Thank you very much indeed. We will return at 2.30. Thank you.

(12.55 pm)

(The luncheon adjournment)

(2.33 pm)

CHAIRMAN: Yes.

A. I'll pick up where I left off, which was the beginning of my original presentation, interestingly enough.

I'd like to go back one slide, just to remind us, if that's all right. So these are the three areas that I will now address.

Sorry, next slide. There are three issues related to the coupler connection. I say "three", it's being quite superficial but I grouped them under three and

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I will deal with them in that way.

Next slide, please. Dealing with the generality and in fact was the focus of the Original Inquiry in January last year, the work that we have carried out in terms of the stage 3 analysis and the very extensive testing that MTR and others have carried out on couplers has given us a very large data set, running to nearly 200 individual samples which have been exposed during the stage 2 opening-up works, which is a very large data set upon which one can analyse the levels of different engagement that were arrived at.

Indeed, in my report, in annex 1, I give an analysis of those results which really -- not "really" -- it absolutely concludes that if you were considering a 32 millimetre engagement, that the failure rate would be no more than 12 per cent of the population. Or, if I was to put that in a more positive light, 88 per cent would pass. In other words, if you had 100 couplers that you had inspected and tested in terms of the PAUT test, 88 of that 100 would pass.

So I don't want -- the problem is people talk about percentages and they talk about pass rates and fail rates and that's what the confusion is so I want to be totally unambiguous here: 88 per cent pass.

On that basis, if you look at the stage 3 analysis,

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both from the Arup reports and more importantly from the Atkins reports -- because, after all, we must remind ourselves they are the designer of this works -- you will find that in no area do any of the areas reach anything like an 80 per cent level of requirement, and it struggles to get to 60 per cent. So, on the one hand, you've got a demand of 60 per cent; on the other hand, you've got a strength of 88 per cent. 88 per cent is larger than 60 per cent; ergo, okay.

Interestingly enough, I think all four of the experts would agree on that, in the sense that from a pure strength analysis of the works, there is no issue in the generality of the coupler connections to the diaphragm walls.

The next slide, please. The exception to that is the coupler connection that we've heard a lot about, in the EWL area A, where there is a different detail, as shown on the diagram. Not to beat around the bush, and I think you know me by now, I don't beat around the bush, I come direct to the point, this connection should be at least as good as any other connection on the works, if not better.

Superficially, there is no reason to look at that detail and say, "Oh my goodness me, this is a disaster waiting to happen." In fact, the opposite is true.

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Indeed, when you read Dr Lau's report and if you were to refer I think it's to paragraph 53 -- forgive me if I haven't got that right -- and you read his introduction to that connection, you actually believe that he thinks this is a good detail; it's going to be better than the rest, and then suddenly he concludes with a conclusion that not that the butler did it but the maid did, which is quite illogical, but no matter. He's entitled to his opinion on that.

But logically, if you look at the detail, you would expect it to be better, and the reasons for that are it's visible. Both sides of the connection are visible. It's an area where the reinforcement is not as congested. If I remind you, generally we have four layers of reinforcement, some of it 3 metres below the surface which is being inspected. This is all very visible, and the operator does have the opportunity of looking on both sides of the connection, which has been the subject of part of our discussion.

So the alarm bells should be ringing. If someone then turns to you and says 68 per cent of these fail -- I mean, you should stop. There's nothing wrong in saying that in your opinion it's 68 per cent failure, but you should then stop and ask yourself: does this actually make sense?

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Now, I'm just an engineer, but one of the things that engineers have to do, they have to face up to problems which have got many facets to them, and in those situations you don't rely on one, single source of information. You seek data. Do you remember I talked about the unknowns being the weakness in any project? You put all your effort into understanding where the unknowns are and then really hammering home, getting the data that you need.

Well, I don't have to look very far on this project to get that data, because the analysis that arrived at the 68 per cent, carried out in good faith, just used a small sample of data. Now, one of the problems with any statistical analysis is the amount of data that you use. If you get a small data set, whatever conclusion you arrive at has got a huge percentage probability or possibility that it's woefully wrong. So small data sets, alarm bells should be ringing very, very loud, and you shouldn't actually progress on that basis.

On this project, if you turn to -- I think it should be the next slide, actually; can you turn on one more? Yes. We'll come back. This is the data set upon that 68 per cent was agreed. This is at the back of my report in annex 2. This is what's called the purpose (ii) data that was arrived at, and there are 11

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recordings there. Four of them were discarded because there couldn't be a reading, a PAUT reading. Two of them were found to be unconnected, and the remaining five were found to be acceptable.

So that's the basis upon the 68 per cent. If you go to the next slide, please, which is the purpose (i) data, which was not used in any of the statistical analysis, for reasons which I'm sure I'm going to get challenged on but as far as I'm concerned this is the additional data, it is random, it was not subjected to the, if I could say, theatre of the sampling that took place, which was more akin to a drug survey in terms of an engineering assessment. But this stuff was just never looked at, and for the life of me I don't understand why, because what you are looking at there is -- the ones which are yellow highlighted, I think there are 12 readings there, the top one was discarded because a reading couldn't be taken, so you are left with 11 valid results which were all in exactly the same area, area A, and indeed I think three of the samples were taken from exactly the same panel that offered up the failures on the earlier slide.

If you look at the engagements, on all 11, they are all in excess, I believe -- I will get corrected by someone if I'm not right here -- of 41 millimetres So

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these aren't borderline cases. These are nowhere near the 37.

So on the one hand we have a data set that is reported to have seven readings of which two fail or five pass, and the same area, we have 11 samples all pass by a very large margin, but they are not taken into account in the analysis. I can't tell you why. It's beyond me.

But if I can go back to the slide -- I do apologise -- that one, that's it. So that's really why I'm saying that on the available data and the judgment really of three of the experts of 32 millimetres being strong enough -- if you go to the next slide, please, and the one after that; that's it -- then you arrive at the only conclusion I can, which is -- the failure rate that I arrive at is 23 per cent, not 68 per cent, and I would go as far as to say, by using the same methodology, I think Dr Wells would have arrived at a better answer than mine -- a better answer, I wouldn't say by very much but of the same order -- and indeed if I used the formula, the magic formula, which is in the MTR holistic report, which has been very maligned, I get a very similar answer.

So the conclusion I come to, which is the one that I would have expected by just observation and common

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sense, that actually that coupled connection in area A is no worse than anywhere else in the construction, and it goes back to I think an observation that you made, Mr Chairman, earlier on, that we are dealing with a mechanical operation. It's a coupler, it's a bar, it's a team of men actually trying to connect something into it, and they repeat this operation many, many times, and there is a statistical probability of the level of workmanship they will achieve. It's not rocket science. You would expect it to be consistent. And there's nothing radically different between any coupled connection in these locations across the project.

That's really why -- and it was interesting that, thinking back to it, Prof Yin did not make a judgment as to whether EWL or NSL was a different data set. He reported that he was told that, whereas Dr Wells has always believed that they were the same data.

So all I'm saying is you have to look for data, you have to challenge data, and very often common sense is the best lead, and what I would call -- one of my colleagues referred to it as a reality check, and all I've done is carry out a reality check.

I think I have nothing more to say on that, actually. I think if we move on to the next subject, if that's all right with you.

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CHAIRMAN: Yes.

A. If we then move on to the next subject. Oh, yes, I'm sorry. I was saying about the level of requirement. So, on the one hand, we have an acceptance level or a failure rate of 23 per cent, or let's call it an acceptance rate of 77 per cent, at no location in area A do you have such a situation, and this slide, if you look at the very end column, albeit with a 30 per cent magic redistribution, you notice it struggles on the first slide to get to 50 per cent. In fact, I don't think it does. Then, on the second slide, if you look at this column here (indicating), you will see that the highest figures are about 60 -- yes, there we are, 62. But generally they struggle to get to 50 per cent.

So, from a fitness for purpose basis, specific to this project -- I'm not extrapolating this anywhere else -- I'm saying that you can see that is the reasons why I believe the structure, particularly area A, is safe and fit for purpose. It's not in my terms a risk. If you were to apply compliance standards to it, well, that's a different set of criteria and I'm not going there.

Next slide. Thank you very much. Now, yes, right, the relevance of the permanent elongation test. Dr Lau raises, I say, a genuine concern because he says it with such emotion. He's not trying to create a bear pit; I think he's genuinely concerned about this. The first thing I've got to make clear to you is: this has never been a strength test and, to the best of my knowledge and belief, it has traditionally been considered to be a quality control test of the product.

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What reinforces my view on that was the CARES certificate that Dr Lau showed in his presentation yesterday, because CARES is a not-for-profit organisation that, if you like, it's like an Agrement Board for products, reinforced concrete products. It's not involved in the actual application of those materials; it really is mostly focused on reinforcement, bars themselves, in terms of their classification and specification, and because couplers are involved in that, they have included that in their certification.

But they are not condoning something in the field. They are just condoning the product. So, again, I'm not taking anything away from Dr Lau's concerns. I'm just saying that's what it has traditionally been seen to be, point 1.

COMMISSIONER HANSFORD: When you refer, Dr Glover, to an Agrement Board type organisation, not everyone here will know what that means, I'm sure. It's

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an independent quality organisation; is that right?

A. Yes, that's right.

COMMISSIONER HANSFORD: I am aware of the British Board of Agrement, but I think it's a bit of an unusual --

A. This is a different thing. You could almost say it's set up by the manufacturers themselves. But actually I've had very good dealings with CARES. They do act very independently in these issues. But they are, as you say, looking after the product, not after the downstream application of the product.

COMMISSIONER HANSFORD: Yes.

A. The other thing I would say, and I haven't put it in this statement, but just to try to put things in perspective, point 1 is not used universally. In fact, I've found it difficult to find where it actually began but I think it began in a DIN Standard, probably many decades above and it just got adopted, but in the States, for example, they would be using, I think -- Canada and the States, they are using more like 0.25 as an acceptance criteria. Same test. Because don't forget that the standard by which these are tested, which is AC133, which is an American standard, is used by them all. But they set their -- sorry, AC133 does not set a standard. It does not set 0.1. It just says, "If you are going to carry out this test, you do it in

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this way", but the authority, the approving authority or whatever you want to call it, sets its own standards. In this case, Hong Kong is 0.1. If you were in Ontario, I think it would be 0.25.

The other thing is, the other authorities recognise that it's not one standard fits all. They recognise, quite correctly -- in fact, some experiments were done at HKU I think many years ago which demonstrated the number has to go up when the bar size goes up. If you think about it, it's the proportion between the area of something and the perimeter of something else, and they can't be proportional, by definition, you know. So smaller bars you would expect to get a lower value than bigger bars. So the bigger the bar, the bigger the movement. But 0.1 applies to everything.

It's again one of these situations that you find yourself in where one size fits all. Anyway, sir, I slightly diverted my thing, but I thought as background that might help you. I think I have alluded to that in the past.

Now, Dr Lau extrapolates, and I do apologise for using the word "extrapolates" but he does, and he does it speculatively, in the sense that you take a bar and a coupler and another bar and you pull them apart, and you then extrapolate what you've observed there, in his

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case he takes an extreme value, and then he says, "That's going to happen over here", and the "over here" is the massive structure of the Hung Hom slabs, where you haven't just got one bar, you've got whole collections of bars. The great majority are perfectly sound, pass all the tests, even the elongation test.

So this rogue bar is part of this wider family, and so therefore that rogue bar will not dictate what the performance of the structure will be. It's the sound bars that do. We use a phrase called strain compatibility, which means that if you apply a force to something, all the elements in it have to strain to the same extent, and the amount of strain you put in any one of the elements is the amount of load that that one takes, so the load gets spread.

So you might get a rogue bar. I wouldn't argue with that. You might get a couple of rogue bars. But clustered around it is this vast family which is going to say, "I'll look after you, little brother, I'll take the load", and that's why I say it's a speculative extrapolation. It's taking the biggest number you possibly can think about and then putting it in the most extreme situation, and it really confuses. It's not correct.

I think I'd better move on to the next slide,

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please. My reason for saying this is speculative is the first thing, and one of the things that Dr Lau says which I agree with 100 per cent is there should be more work in this partial engagement issue, and when I say "more work", more experimentation, more on-site observation. On-site observation is fundamental in these things, and I would agree with that. The manufacturers don't want to go there. I can understand that. But I can see, with the sheer volume of couplers that are used in the industry, it would be to the benefit of the manufacturers if they cleared this issue up, because I can see it returning.

For Hung Hom, I've got to emphasise again, all my observations are only on Hung Hom. I would not want them -- in fact, not "want them" -- I would say they should not be extrapolated to any other situation. I'm just looking at Hung Hom.

But the magic thing is, the project that we've got out there at the moment is the most fantastic load test I have ever seen in my life, in the sense that the structure is already subjected to 90 per cent of the load that it will ever sensibly see.

So, if you were going to get cracks, you would have them now.

CHAIRMAN: What is said, of course, by Dr Lau is that there

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may be cracks but you don't see them now.

A. Why wouldn't you see them now? I mean, are they hiding around the corner? Are they going to pop out?

COMMISSIONER HANSFORD: I think his argument was they are hidden inside the concrete.

A. Well, there would be cracks, potentially, inside the concrete, but they wouldn't be due to these issues, which are due to surface strains. You would get something called a shear crack which is actually within the body of the structure, it never goes anywhere else, and in fact there's a slide I will show later which describes shear cracks. It's all to do with -- in crude terms, it's like a Poisson's ratio effect. In other words, if you push something, it tries to squeeze out, so if you imagine something which is in very heavy compression, it wants to burst out, it wants to spread, and so you get what we call complementary tensions, so you get this compression strut and you get these complementary -- that has come apart. That's called a shear crack. But that doesn't find its way to the surface and it certainly isn't a source for corrosion, because it's within the body of the slab.

When people talk about cracking, you've got to be very precise about what sort of cracking and the cause of that cracking. But the idea that there's a crack in

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there that's hiding around the corner, waiting to pop out when you are not looking -- I mean ...

CHAIRMAN: And of course, when you talk about corrosion, that's the other issue. If it's internal, it's not open to the air and to the elements in any way so it doesn't corrode?

A. Exactly. Sir, you understand exactly. You need three constituents for corrosion. You need iron or steel, you need an electrolyte, in this case water, and you need oxygen. So if you were to put some water -- let's say water managed to get into a small cavity, it's got oxygen, it's next to steel, and it's certainly got water. Once that oxygen has been exhausted, nothing happens, and this is true -- and I think interestingly enough in zone -- category 1 exposure, I think you will see there's a line there which says, "Structures immersed in water". Well, that's because there is no fresh supply of oxygen. You've got plenty of metal, you've got plenty of water; no oxygen.

So the idea that you've got water somewhere doesn't mean to say you get corrosion. You need a constant supply of that water, oxygenated water. If you don't have the oxygen, you don't get corrosion. I only wish people could understand those basic principles better because then there would be less concern, I think, in

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our world, on this issue of durability.

Believe me, I'm not making light of durability. I'm just trying to explain why I believe, in this particular situation, those concerns are not correct.

The other thing I would add is we have carried out very extensive demolitions in the structure. I mean, it's a very forgiving structure, fortunately, but all that vibration that's taken place, if there were they cracks hiding, waiting to break out, the vibration would have certainly brought them to our visibility, and they would do that for two reasons. One of them, they would have caused agitation of the crack, which would not have been visible to the naked eye, potentially, but the other thing is once a crack opens up and it gets dust in it, it becomes very visible, and we haven't seen any of that.

The second point -- I think Mr Southward made this point and I agree with all of his observations -- is the environment that we're dealing with in the location of the couplers is a benign environment. You can pick other locations, potentially, in the box where it is not benign, but not the inside of it, not the position where the couplers are. So I would agree with Mr Southward.

The last point is the one I've already made: the connections occur, the couplers occur, in very large

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clusters, and we are not saying each one of those clusters is all non-conforming.

So those are the reasons why I understand the concern but I do not agree with it, for the reasons I've given.

Next slide, please. The trough walls. Here -- what's my concern? I've been involved in impacts -- and that's what this is, this is an impact loading; this is not a point load at the end of a cantilever, as the modelling shows, this is an impact, it's an energy thing, it's to do with impulse, it's to do with energy absorption, and that's why I said the yield line approach is an appropriate approach, because it's an approach which assumes things are deforming and plastic energy.

COMMISSIONER HANSFORD: This is the case of a derailed train.

A. A derailed train, yes, agree. I can say I don't agree with the force, to start with, because dealing with high-speed trains, as I've had to, and derailments, the idea you get this massive lateral load horizontally is not there. The reason -- and these are almost like derailment kerbs, you will appreciate that, and the train comes off and it glides along. It's not this sudden punch that you see in -- I wouldn't have

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approached this particular problem. If this was an issue, this is not the way to deal with it. I think in terms of resolving it, the structure is very substantial, and what amazes me, the earth backfill is not taken into account, nor, in fact -- if you are doing a dynamics problem, you don't say there's a piece of concrete there but it's not considered by the authorities to be structural, therefore I'm going to leave it out, or the soil might not be there. Those things don't enter into my thinking. That's what's there, that's what's being constructed, it's not going to change. I've got this impact load. How will it be absorbed?

It will be absorbed in three ways. Number one, the earth is a very, very good absorber. The slab itself will absorb energy and gradually go plastic. And thirdly, this oversite concrete at the top -- which interestingly enough is also connected to the columns, so if we were so worried about the columns, why do we have the oversite concrete cast around the columns? Why isn't there a gap? So I can't take the risk seriously, to be honest. As far as the mathematics are concerned, I've told you what I think. I wouldn't have approached it this way, but if you had approached it in the way that I would have done conventionally for a derailed

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train, there certainly wouldn't be an issue. I'd just like to leave it at that. I don't agree with the 35 per cent anyway, which I could go into.

I'm not giving you the sort of focused response you would like from me, I think, on this issue. I'm just saying I don't see that there is a major issue here. I can understand we don't want to knock columns down with a big building above, but we've overstated the problem.

Could I go to the next slide, please. This is the conclusion, really, on couplers. As far as I can see, all the coupler connections have been shown to be adequate and the structure is safe. It is incorrect to assume on the basis of the results of the elongation tests that cracking will occur, for the reasons I have already explained.

Interestingly, I was reflecting on this when I was getting this presentation together, I think what gets lost is how do you deal with cracks on the site? Because you get cracks and some of them are larger than you would like them to be, particularly on bridges, for example. We don't put them on the safety list. We put them on the rectification list.

So really the cracking comes under the same heading as honeycombing, in the sense that it is something we

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don't want to be there. I mean normal cracking, okay, but if there's excessive cracking, you repair it, and you'd approach it in exactly the same way as honeycombing.

The unfortunate thing, or fortunate thing from my point of view, is we can't observe any cracks to repair, and there isn't this massive load like a herd of elephants being let loose in the station which is going to create it, because the train loading actually is pretty small and runs almost on top of the diaphragm wall anyway, so you are not getting this loading coming from the span increasing the bending moments.

Then the last point I've just made, basically, with the trough walls, I think it's a lot of fuss about nothing, to be honest, but if we could move on on that.

Next slide, please. Now the shear link reinforcement. Yes, I mean, I agree with all the observations that have been made about the shear link reinforcement, in terms of numbers of bars, anchorage of bars, spacing of bars, whether they are there or they are not there. But the fact is the structure has been so robustly designed that there's very few areas that actually require shear links anyway. So rather than debate whether the actual details in the generality of a construction are there or not, it really isn't the

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point. The point, I believe, is in these limited areas, is it fit for purpose, is it safe? That's the question.

So rather than getting into that discussion as to what might have happened in area B or area C which is irrelevant, let's just focus on the area we are interested in, which is area A.

I would emphasise, actually, that the approach which is being adopted in assessing the shear strength is one of compliance, the reference to the code, saying the code says this is the permissible stress for the concrete, therefore this must be the answer, and if it's not that, it must be unsafe. That's not correct. It can't be correct from a commonsense point of view. From a compliance point of view, it's absolutely correct. But I'm not looking at it from that point of view.

If I move on to the next slide, I hope that the following three points will help demonstrate that there is, from a safety point of view, no concern. I say that you can address the problem from a safety and a fitness for purpose criteria by considering any one of these three, either separately or collectively. You can say, "Well, actually, it's not reasonable to assume there's no reinforcement there", because the exceedance of stress that we are talking about is very small. We are talking about -- I do apologise, I haven't got the

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number in front of me -- but it's no more than about 10 or 15 per cent more than the code would allow. It's not, "My goodness me, it's double or triple." No, it's quite a small judgment.

So if there was just a nominal allowance for the shear reinforcement, the problem would just go away, and indeed the areas other than in the SAT are not huge.

So that's one which you could solve it which I think on the basis of the photographs that Mr Southward shows, they are quite compelling. Those are areas which were reported not to have any links. But what you would obviously have to do is you would actually have to look at the source of those photographs and make sure that they did represent what they were meant to represent, but that's by the way.

Moving on to the second point, and I think this is really the crux of the issue as far as I'm concerned in terms of why I think it's okay. I took you to this issue of design strength and actual strength, aging factors, et cetera, and in my opinion I see no reason why an enhanced concrete strength should not be considered in those areas, and indeed, if you did that, I think you will find that the problem goes away, particularly in the SAT area.

Then the last one is my points this morning about

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cavities suddenly appearing under slabs 18 metres underground with 16 metres of water plus acting on them, which have already been dewatered during their life. I don't think that's credible, and if the slab doesn't have a cavity, then you don't get the shear stress and whatever.

The other thing is a lot of the models that have been used on the project are what I would call two-dimensional models, they are just slices through the structure, and as Mr Southward pointed out there are three dimensions to a structure, and if you isolate it to -- let's assume this is the bottom slab of many slabs but the other slabs are all connected by walls. If you just analyse this one in isolation, you've lost the sort of the gathering effect, the sharing effect of the others, and I believe the analysis that was carried out was indeed only a two-dimensional analysis.

So I would add then to that third one, when I say "sensibly conservative", in other words I do not want models which are not conservative but I want models which represent physically what is there, and that would mean three dimensions and soil underneath, particularly water pressure.

So that's my opinion on the shear links. I think it is safe and it is fit for its purpose.

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The next slide, please. I want to deal with this one in two parts, to deal with the issue of the horizontal construction joint which we refer to in the shorthand as the CJ. There is no doubt, and interestingly enough Dr Lau also supported the view that this is a workmanship issue, it's not an issue of safety or fitness for purpose. The other thing is that the contractor, MTR, the designer and also the approval authority have all agreed that a nominal dowel detail will satisfy and will solve the workmanship problem. With that army of people willing to do the works and to solve the problem, I really find it very difficult to make a comment, particularly since everybody has put so much effort into the work, into the method statement in terms of its construction.

The one thing I would draw Dr Lau's attention to is that although the pilot holes which are being drilled do use a normal what I would call masonry drill, the actual core is diamond-cut, and you wouldn't know that you were going through the rebar. That's a big difference.

COMMISSIONER HANSFORD: So what you are telling us there is that whilst you would get a clear indication you had hit reinforcement from the pilot drills --

A. Yes, correct, yes.

COMMISSIONER HANSFORD: -- once you got into the core drill,

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which is the 32mm drill, you would have no idea?

A. Correct, but the tolerance on that is you've drilled a hole which is basically 16 millimetres in diameter, and because of its percussive nature it's going to be slightly larger. You are using that as the centre for your 32 -- the risk of you hitting anything important is much reduced.

COMMISSIONER HANSFORD: Okay. Thank you.

A. Whether you should do it is a different thing.

The next slide, please. I said I would do this in two parts. I call this a new issue because this came out of nowhere, as far as I was concerned, in the hearing, and that was any question at all about the diaphragm wall which was raised. So, if you forgive me, I've done a little bit more analysis on the joint so that we can hopefully put this one to bed as well.

In the following slides -- I'm going to use some slides which are in my reports, previous reports, about the stress regime in the joint, but then I'll hit, in the last slide, this issue of the cracking that Dr Lau brought up which I found quite disturbing, really, because I think it was misleading, but I want to bring it back into focus.

So if we could just take each of the bullet points in turn. The actual failure mechanism of the

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construction is 10 meganewtons, if that means anything to you, but actually the ultimate strength that we would be looking from it is much less than that; it's only seven. So the actual arrangement of the diaphragm wall, even though the diaphragm wall is the weakest part of it all, it still has adequate, oodles of strength to it.

The second thing to point out is that even at failure, even when the diaphragm wall is buckling and collapsing, there is virtually no stress at all at the CJ.

Then the last point is the cracking and I'll come on to that. Go to the next slide, please. This is the scrappy calculation I referred to in our first hearing, Prof Hansford, where I said this is how I think it's going to fail.

COMMISSIONER HANSFORD: Yes, I recall.

A. If we go to the next slide, that's in fact what happens. You get a compressive flexural failure at the junction between the soffit of the EWL slab and the diaphragm wall, that large green area, and you can also see --

COMMISSIONER HANSFORD: Perhaps -- sorry to interrupt -- for the benefit of the chairman in particular, you could just explain what these colours all mean?

A. Yes, of course. The colour code is this (indicating). Blue is very, very low stress and, as the colours move

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towards red, it gets higher and higher. So when you look at the area around that joint there, which is -- this is the EWL slab and this is the diaphragm wall, you can see you get that compressive flexural failure at that point. Then the thing to point out is you get this very strong compression strut running diagonally from that point to that point (indicating), and that's where -- can you go back slightly, please. I pressed the wrong button, I think. That's it, thank you. Which is exactly how we would expect it to happen.

Does that help you, Professor?

COMMISSIONER HANSFORD: I was okay with it, but I felt the chairman might need some help.

CHAIRMAN: It certainly does. Thank you.

A. It is that compression strut which is the fundamental way in which the structure performs.

Next slide, please. Looking at the top of that joint, this (indicating) is where the construction joint is, there (indicating), and you can see the stresses are all concentrated below the joint, at this corner (indicating), and that's because this structure works because of these bars, and those bars are acting like dowels. You can see the stress concentration here, in the centre, where the hand is, and you can see there's hardly any stress in the concrete there, and this is

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a thrust line which is pushing on the bars and then resisted by the top of the diaphragm wall below the construction joint. Construction joint does absolutely nothing.

COMMISSIONER HANSFORD: Perhaps before we leave this slide, or you might tell me it's better to come to it in a later slide, the dowels that are going to be inserted as part of the special measures, what will they do to that?

A. Well, they will increase the number of dowels. I mean, there's going to be one here (indicating).

COMMISSIONER HANSFORD: But what will they do to the stress levels?

A. Not very much, because they only represent something like 1 per cent of the total dowels which are already there, but if people are happy with it, I'm not ...

COMMISSIONER HANSFORD: Okay. Thank you.

A. It's not -- as I say, I'm telling you the structure as it is is safe and fit for its purpose. What other people want to do for other reasons is entirely up to them.

COMMISSIONER HANSFORD: It's not something to die in a ditch for.

A. I would rather not die in any ditch, but particularly on this one. I think I've said my piece in the past and

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I'm just the honest engineer that does the analysis and gives other people bad news, I guess, in some respects.

This is entirely predictable, the dowel action is exactly as we predicted. We did lots of hand calculations to demonstrate this, but that was not good enough for some people. So we've gone the whole mile and done the non-linear finite element using some of the best code we can in the world.

So the stress levels, and that's at failure, so this is already something like 50 per cent higher than the ultimate tensile stress. The stress levels, if I go to the next slide; thank you very much -- this is what the stress looked like at the time of the ultimate tensile stress. This is when the structure should be falling apart. Again you can see the stress levels are remarkably low at the CJ. All the action is happening down at the bottom, with the diaphragm wall.

The last slide, and this is the penultimate slide -- we can look forward to having a bit of a rest, I think, after this -- what we've done overnight, we have assembled three sections through that joint that you've been looking at, and we've considered them for different levels of applied loading. Remember that the failure load that we would be looking at -- you see we've got -- the largest one is 6 there -- we would be going up to

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about just under 10.

I'm not showing you the full thing. I'm just showing you three situations which are pretty pertinent.

The first one is one of the lower loaded pieces of the structure, as it is today, and we would expect to see some minor cracking at -- there's some minor cracking at this joint, which should be entirely where you would expect, because this is where the high stresses are, at that intersection, but nothing to be concerned about. I think we've shown on the code what the crack widths type of things that -- yes, I mean it's 0.1mm, that sort of size, very nominal. And you've got to remember, when we show one crack on here, it's because it's magnified. In fact what will generally happen, because this is reinforcement, because the modelling can't model right down to a single fraction of a millimetre, so that one crack probably represents a cluster of three or four, all smaller than that, but just locally to that area. Because something that people don't understand: reinforced concrete only works when it cracks. It might come as a surprise to people, but the only way that you can mobilise the stress in the bar is if the concrete cracks and grabs hold of it as it moves apart. So cracks in reinforced concrete are exactly what you would expect.

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What you want to avoid is having excessive cracks in the wrong environment. Excessive cracks can be acceptable from an engineering viewpoint, as long as they are in a benign environment and they don't cause visual distress, because one of the fitness for purpose criteria that Dr Lau didn't include, interestingly enough, although it's a very comprehensive list, was aesthetics, and actually one of the key considerations in structures is that actually, when you look at it, you mustn't feel uncomfortable, and if you saw a large crack in the wrong place, you would feel uncomfortable. But if it's not in a position which causes you distress, then cracks can be much wider than 0.3 millimetres and not be of any structural significance whatsoever; all right?

If we just go across to -- sorry, I've pressed the wrong slide again -- the middle one represents a situation after SLS. Remember serviceability limit state? And this represents a situation of stress which is in excess of what the structure is enduring now or indeed is in excess of what it would endure later. You can see the clustering of cracks, as you would expect, is spreading a little bit further, but again nothing to be alarmed about in that sense, and remember my comment earlier: one crack here really should be read as

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a cluster of cracks, all smaller.

Then the last one I've shown there, the slide there, this one (indicating), this is ULS. This is when the whole structure should have fallen down and collapsed, and again you can see the degree of cracking is nothing to be unexpected, and this is ultimate limit state. The only ones that you should bear any credence to are this one and this one (indicating), and there's no case to answer.

The other thing I would point out, in those first two slides, is that the construction joint is up here (indicating). There's no cracking. I mean, there's no water. Where's the water going to -- how is it going to get in there? So I say to you, even when you get to the ULS, where is it coming from?

The other thing is you've got to remember that the EWL slab is at a level of 4mPD. In other words, it's 4 metres above ordnance datum, let's call it sea level, and the groundwater level outside at around about 1. So the groundwater is almost at the bottom of the slab. It's nowhere near the joint at all.

COMMISSIONER HANSFORD: Just to be clear, the dowel bars that are going to be inserted as part of the simple measures will make no difference to this; is that correct?

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A. No, not at all. I mean, it's -- no. I wouldn't even bother to model it. It's of no consequence.

COMMISSIONER HANSFORD: Okay.

A. But it's not being inserted for reasons of structural integrity, I understand. I mean, I'll leave that for others to address.

My position is very straightforward. It's the structure is safe and it's fit for its purpose, as it is today, and if others wish to do something to it for other reasons, for compliance or whatever, then it's not for me to say that. I've been given or given sight of a very reasonable method statement. I think reasonable measures have been taken to avoid some of the things that I was concerned about. So, to be quite honest, I'm not going to ask my neighbour to stop playing football if it's not interfering with me. That's up to them and it's outside of my brief.

COMMISSIONER HANSFORD: Understood. Yes.

A. I think this is the last slide. This is a personal view. Well, these are all personal views rather than collective, but I feel quite strongly about this one, and I think I've been consistent in my meetings with the Commission on this. I've got severe scepticism of what I will call automated monitoring systems in situations where I do not believe they are necessary. My reason

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for saying that is because they cause more problems than they solve.

And they come under two headings, those problems. The first heading would be operationally. Any system you will have to put this here, because the movements are minute -- I mean very, very small, and I don't know what we are looking for. What is it? Are we looking for -- generally, when I've used monitoring, it would be something where you are plotting something and you are seeing a trend and you say, "My goodness me, let's stop", like if you are digging a deep hole and you are measuring the ground outside, you've made your predictions and you are matching it, you're monitoring it very, very carefully, and you've got a trend line. In a situation like this, I'm not sure what that trend line would be. For the life of me, the structure has been there for three years, four years. It's deflected about as much as it's ever going to. So what am I reading? What am I trying to find?

So it's going to be very -- so anything that's there, unless it's, you know, made of polystyrene or something and just for show, it's not going to do anything. So with very sensitive machines, sensors, you'd get noise, you'd get noise on the signal, you would get a malfunction in one of the devices, and then

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all hell lets loose. You know, suddenly trains have got to be stopped, people have got to go and find out what's gone on, so you get a public relations disaster on your hands.

I'm being practical about this and it might not be music to many people's ears, but I think the best solution, and this is what we do on -- my bridge in Scotland, for example, is festooned with measurement devices, but there we are measuring towers which are 200m high in high winds and we want to know how much it moves. But even there we rely mostly on visual inspection, and what the station really needs is a planned preventative maintenance/inspection regime, which means having a look at particular items of construction which one considers from the analysis that's been carried out to have a particularly higher stress than the other areas.

Take area A, for example, with the couplers. It's been raised as a concern. Well, that would be an obvious area to inspect regularly, but I can't see what any instrumentation will do for you.

CHAIRMAN: Can you not -- sorry to interrupt -- calibrate the monitoring equipment so that it only records movement at a particular level?

A. Yes, but that level is going to be so small that it's

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actually within the noise of the thing. I mean, all electrical, electron devices are not precise, they have a noise to them, so there's an error in that. If what you are trying to measure is actually very comparable to the error, then I'm not sure what you are doing.

CHAIRMAN: I see. Yes.

A. What you could do, if you really were going to be severe, is you could dig up the structure again and put some strain gauges on it, but the trouble is the stresses are already in the bar; it's already stressed to 90 per cent, so what are you going to be measuring?

CHAIRMAN: And I would suppose -- this is a layman talking again -- if you only calibrated to start recording measurements at a fairly high level, then the criticism may well go out that if you had recorded at a lower level, you would have picked up a problem much earlier?

A. Correct, and why didn't you do it three years ago? You can only pick up things into the future, and because the structure is so dominated by dead load, the loads are already there, which is my point about the cracking, which is my point about the stress levels that we've got in the structure now.

COMMISSIONER HANSFORD: Presumably, Dr Glover, you would only be measuring any future movement, you wouldn't be measuring any movement that's already taken place?

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A. Correct, yes. And one of the points that Dr Lau has made which is very true, if there was such a thing as shear failure, and I can't see it, but that -- a shear failure is something which occurs quite quickly. It doesn't go under -- a punching shear situation, it's explosive suddenly, but if there was a shear problem, you would start to get the shear cracks I started talking about, but you can't see them because they are in the body, but you might get some slight distortion. But I really think it would -- I can understand how the public might say, "You are hiding something", but to be honest, my advice is trying to protect the government and the public from what I would say are misunderstandings of the data that's coming out, and it's much better if there are regular inspections which are properly recorded and what I call a preventative planned maintenance regime is set in place for the station. That's my advice. I certainly wouldn't engage in some of the more sophisticated devices like fibre optics, et cetera, because I just don't think they are applicable in this situation.

CHAIRMAN: Also, could I ask this. Let's assume for a second you did -- we are talking theoretically -- start to spot these minute cracks arising and stresses, minor stresses.

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COMMISSIONER HANSFORD: What would you do?

CHAIRMAN: What do you do, that's it.

A. That's exactly the point. You then descend upon that point and you carry out very focused investigations on it, and they don't involve measuring things. They measure -- sorry, when I say measuring, measuring dimensions or whatever. You look at it, you look at the patterns of the shape, you actually go back and ask yourself is there something extraordinary happening, is there a particular load that's suddenly come on, has somebody taken a herd of elephants down there recently or something, or has there been a very unusual train load with unused nuclear waste running down, because they are quite heavy trains actually, someone illicitly used the railway? So that's what you could do.

The interesting thing is, you see, even if you installed such a system, whatever that system might be, you are not going to turn it on and walk away from it. You are still going to have to carry out inspections. I mean, maybe people think you don't. Well, you must do.

CHAIRMAN: Of course. You mean you can't ignore it.

A. So you're going to do that anyway. No, of course not.

CHAIRMAN: But --

A. Sorry, Chairman. Because these instruments wouldn't be

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picking up the issue you've just raised which is the little cracking. They might pick up the crack in that one location or something, but what about the pattern? It goes back to the test on the coupler again. If you just get one reading, it's not telling you anything about the mass, and what you are interested in, in inspections, is the correlation between that and that, and you can only do that really visually.

COMMISSIONER HANSFORD: You referred to a planned preventative inspection maintenance regime.

A. I call it a PPM just for shorthand.

COMMISSIONER HANSFORD: Yes, sure, okay, fine. Is that, in your view, over and above what should be happening anyway?

A. I'm not familiar with the procedures in MTR for railways, but that's certainly what we have in the UK. We have our regular bridge inspections, as you well know.

COMMISSIONER HANSFORD: Yes.

A. I'm thinking about those in the first five years as being an extra-over. I would expect those inspections to take place anyway, the ones you and I talk about.

COMMISSIONER HANSFORD: Yes.

A. And I'm talking about these being specific and focused on those areas which have raised concern.

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COMMISSIONER HANSFORD: Okay. An additional sort of enhanced --

A. Call it enhanced, that's a good word, actually, because that's what it is.

COMMISSIONER HANSFORD: Enhanced inspection regime.

A. Focused on the areas of concern.

COMMISSIONER HANSFORD: For something like the first five years, perhaps?

A. Yes.

COMMISSIONER HANSFORD: Thank you.

A. The reason I say that is because I think after five years people will be satisfied that the thing has been operating now for a long period of time, we've shown due diligence in trying to get to the bottom of the issues. I don't see it being longer than that, but I do agree with you, and in fact I think we're speaking the same language here, that there has to be a background of inspections going right the way through to the future.

COMMISSIONER HANSFORD: That would be my expectation. Thank you.

CHAIRMAN: Thank you very much.

A. I think that's the last slide, isn't it? Yes. Thank you very much, Prof Hansford and Chairman.

MR BOULDING: Dr Glover, I have no questions arising out of the presentation. The procedure now is that you will be

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cross-examined by counsel for various of the parties, starting with Mr Pennicott, I suspect, then government, then Mr Chow, and then finally Mr Shieh for Leightons. The learned Chairman and Prof Hansford can ask questions at any time they'd like. Then, depending upon what you say, conceivably I might have one or two questions for you at the end. So please stay there.

A. Thank you. Is it all right if I continue to stand? I feel more comfortable this way.

CHAIRMAN: Absolutely. You make the choice yourself.

MR PENNICOTT: Sir, I'm happy to press on. I don't think I'm going to be very long, or I'm happy to break, depending on how you feel.

CHAIRMAN: Oh, I see.

MR PENNICOTT: I don't think I'll be much more -- I had estimated previously about half an hour to three-quarters of an hour. I think I'm going to be a lot shorter than that now, in the light of Dr Glover's presentation, to be frank.

CHAIRMAN: Maybe we will press on for the time being.

MR PENNICOTT: If I may, sir.

Examination by MR PENNICOTT

Q. Dr Glover, can I first of all take up the last point you have just been discussing with Chairman and Prof Hansford, that is about future monitoring.

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A. Mmm.

Q. In the joint report, for the first part of the Inquiry --

A. Ah, right, yes.

Q. -- you may recall that's where this hare started running, because the experts on that occasion -- it's annexure E, sir, to the interim report where you have actually set out the whole of that agreement -- at paragraph 5:

"... agreed as follows.

'All agreed that a load test was unnecessary because it would yield no meaningful result and long-term monitoring would be a better approach to allay public [safety] concerns."

A. Yes.

Q. So that was what was, as it were, signed up to and agreed at that stage.

A. Yes.

Q. That led to the Commission, at paragraph 391 of the interim report, making a recommendation that:

"The Commission recommends ongoing monitoring of the station structure during operation of the station, so as to provide reassurance to the public."

Do you see that?

A. I do.

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Q. Now, as I understand it, what we are now doing, or what you are doing, and I'm bound to say --
A. Suggesting.
Q. Suggesting, and I'm bound to say straightaway, Dr Glover, Prof McQuillan agrees with you.
A. Oh.
Q. -- is putting a bit more detail on the monitoring.
A. That's right, and I must -- we put forward that proposal at that time because we did not have the benefit of the stage 2 -- in fact any of the holistic works.
Q. Quite.
A. We had only had a sort of microscopic approach, looking at certain aspects.
Q. Yes.
A. But now we look at the total picture, I think it would be overkill to do more than I'm suggesting.
Q. Yes.
COMMISSIONER HANSFORD: I can't remember the interim report without looking at it. Can we go down a bit, because I think we go on in paragraph -- no, we don't. It's in a previous section.
MR PENNICOTT: No, that is the only recommendation.
COMMISSIONER HANSFORD: There's a previous section where reference is made to the expectation that the movement will be extremely low.

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MR PENNICOTT: That's right. It's the next sentence actually on 391:
"... the Commission notes the advice it has received that it is unlikely that any significant movement will occur."
That's I think what you had in mind.
COMMISSIONER HANSFORD: Thank you, that's the point.
CHAIRMAN: But on the other hand, if you are looking at the recommendation -- and again we are talking theoretically at the moment, to test the parameters -- you don't want a test which unnecessarily and inaccurately and erroneously causes public alarm --
MR PENNICOTT: Quite.
CHAIRMAN: -- as opposed to public quietude, because the press would be entitled to ask, "Oh, there have been 37 little beeps below whatever measurement is used", and that gets into some newspaper and they are talking about 37 incidents of stress or something appearing, and nobody wants to go down to the Tube station anymore.
MR PENNICOTT: Yes.
CHAIRMAN: That's overkill, but I don't think it's that much of an overkill, actually.
A. I agree with you.
CHAIRMAN: It's not necessarily that you actually have to have alarms running and saying, "Please get out of the

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station, it's going to fall down." It's a question of freedom of access to information and misreading of information, et cetera, et cetera.
MR PENNICOTT: Yes.
CHAIRMAN: Whereas visual testing is a monitoring of itself.
A. Correct.
CHAIRMAN: You say it's more likely to be accurate as to anything that really needs work done.
A. Yes. At a very simple level, you can see that the inspector comes along, he takes a photograph from exactly the same position, the same angle. He highlights whatever he might have seen on the first occasion and he sees if there's they difference. If he does see something which is a little bit untoward, he would go back at a quicker interval. It's a responsive -- it has to have the rigour of regularity, but then if there is a concern, you speed up that particular issue in that particular location. I mean, that's the way it's done around the world. This is not new.
CHAIRMAN: All right. So that would be a very normal, accepted --
A. Yes.
MR BOULDING: Sir, I hesitate to intervene, but it may well assist Prof Hansford, when he was talking about what he

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had in mind, if we went to chapter 11 of that report and started looking, I think, at paragraphs 459 and 460. I hope that might assist.
COMMISSIONER HANSFORD: Yes, thank you. I knew there was a bit more written somewhere. Yes, that's right, thank you, that's exactly it.
CHAIRMAN: Because one of the problems -- and I'm musing out loud -- is that what can happen to the average person in the street, which includes me, except I've been educated now because I've been here now for several months, is it becomes actually a little bit blown up and almost becomes a sort of Samson and Delilah type of Armageddon where you imagine the station suddenly, the columns coming down and the whole thing crashing down and the public fleeing out, you know --
A. Screaming.
CHAIRMAN: -- being crushed to death and everything, as opposed to what in realistic terms it would be, even if one took Dr Lau's scenario without any equivocation at all, it would be some form of cracking requiring internal works, but may be something very small falling off a roof or something, but nothing of any --
A. Consequence.
CHAIRMAN: -- real consequence --
A. Correct.

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CHAIRMAN: -- to the public at large all at one time.
A. Yes.
CHAIRMAN: Thank you very much.
MR PENNICOTT: Mr Boulding having helpfully pointed us in the right direction, if we could just have a look at paragraph 460 of the interim report. It does indeed say:
"The Commission accepts the advice provided to it by independent structural engineering experts that the east and west diaphragm walls and EWL slab and NSL platform slabs should be instrumented to detect movement during the operational phase of the station. Instrumentation should be by means of fibre optics or other approved measures. Movements should be monitored and reported to the government."
I think it's that aspect of it, Dr Glover, that you are now, as it were, suggesting perhaps is a stretch too far?
A. I'm sorry. I think it is going too far and the reason for that is we've done so much more analysis and it's just fortuitously I included a slide about vibration fatigue. Quite honestly that demonstrates yet again that the levels of detecting movement is going to be very, very low, to a level which I couldn't have anticipated back a year ago. I just hadn't done the

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work.
MR PENNICOTT: Sir, and your notification, and I'm sure Dr Glover will be pleased to hear it as well, my instructions are that Prof McQuillan takes the same view as Dr Glover regarding the type of monitoring going forward that would be appropriate, and I know because I've seen some draft of his slides already and had a discussion with him about it. So I think certainly Prof McQuillan and Dr Glover are ad idem on that particular point.
CHAIRMAN: Certainly, as an aside, not often but it's sometimes an unintended side effect of testing new medical equipment on individuals, but if it's too refined, you frighten the patient to death, because he's strapped up with some sort of monitoring system, and every time he looks, it's in the red, and he thinks he's dying. So I can see trying to get something that is accurate that doesn't cause alarm.
MR PENNICOTT: Getting the balance right.
A. Yes, getting the balance right. I would call it an appropriate response.
MR BOULDING: Could I just invite your attention to paragraphs 461 and 462 as well. Thank you.
MR PENNICOTT: Indeed.
A. I would use the word "appropriate response", really.

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The suggestion is what I call an appropriate response to the results of the investigations we've carried out.
MR PENNICOTT: Yes.
CHAIRMAN: Yes.
A. I'm certainly not suggesting just walk away.
MR PENNICOTT: We understand.
CHAIRMAN: No, no. In fact, as Mr Boulding has pointed out, those last two paragraphs, they flow on nicely from that, so it's a form of monitoring that takes into account what we've mentioned in those paragraphs.
MR PENNICOTT: Yes.
COMMISSIONER HANSFORD: Well, we drafted them as such.
MR PENNICOTT: Of course.
COMMISSIONER HANSFORD: But can I just be clear -- I will ask the question of you, Dr Glover, but you may not know the answer -- has any instrumentation been put in place to date?
A. Not that I know of. There were some railway instrumentations when they were running trains, as far as I'm aware, but nothing specific to the structural integrity.
COMMISSIONER HANSFORD: Right.
A. That would have to be confirmed by MTR, but no.
COMMISSIONER HANSFORD: Right. Thank you.
MR PENNICOTT: Sir, I just thought I would deal with that

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first whilst it's fresh in everybody's minds.
CHAIRMAN: Thank you very much. Very practical, and could be important or it is important.
MR PENNICOTT: All right.
Dr Glover, thank you for that and thank you again for coming back to the Commission to give evidence. I forgot to mention it earlier.
A. Thank you for inviting me.
Q. Dr Glover, just a few points arising out of your reports, if I may. Can I ask you, please, to be shown paragraph 5.2 of your COI 1 report. We will put that up on the screen if it's easier.
A. 5.2, yes.
Q. This is the paragraph that I showed or read out part of to Dr Lau yesterday.
A. Yes.
Q. It's where you refer to the degree of post-construction surveys, inspections and opening-ups and so forth.
A. Mm-hmm.
Q. You say, towards the end of that paragraph:
"... none of the findings have exposed any fatal flaws in the construction ..."
And I think that's something that Dr Lau was happy to agree with. Then you say this:
"... despite the analysis and testing being

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subjected to very high acceptance standards."
What did you mean by that, "very high acceptance standards"?
A. I'm sorry, I've lost that particular sentence. Sorry, yes, the last clause, yes. I've got it.
Yes, what I was saying was we've found nothing -- maybe the English is poor, but the intention is: nothing has been found and we have gone to enormous lengths to find the smoking gun, and the smoking gun has not been found, if you want that in sort of plain English.
COMMISSIONER HANSFORD: It's not acceptance standards, it's the --
A. The rigour by which we have conducted the tests and the studies. I do apologise for that. It must have been a midnight paragraph.
MR PENNICOTT: That's fine. That's why I'm here.
A. I hope that makes sense. That does read with it, I think.
Q. Okay. Can I just ask you this, Dr Glover. Do you, from an engineering perspective, see any difference at all between safety on the one hand and fit for purpose on the other?
A. Yes, I do, and the reason for that is fitness for purpose as far as -- you have safety, which is to do with being secure, the structure is strong enough and it

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doesn't deflect too much or whatever, but the other aspect of fitness for purpose is it's got to operate -- it's got to have the right sort of characteristics in terms of deflection, which is a crossover between the two.
But a structure can be safe but I couldn't use it as a railway because it could deflect too much. Does that help you?
Q. Yes.
A. I mean, so there is a separation, and I thought Dr Lau's sort of long list, you could see there are many which are common and there are also many which are completely different. If you take my one about fitness for purpose in terms of aesthetics, for example.
Q. Yes.
COMMISSIONER HANSFORD: And it could not be fit for purpose unless it was safe?
MR PENNICOTT: Unless it was safe.
A. Correct.
Q. Not necessarily the other way around.
A. Thank you for that. That's a very good way of looking at it.
Q. The next short point. You tell us that it was Arup that initiated or suggested the application of the binomial approach to the statistical analysis.

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A. That's correct.
Q. Was that something you were personally involved in or was that some of your colleagues at Arup?
A. Some of my colleagues at Arups. It's one I shared in.
Q. When you say you shared in, you personally believe that that is the right approach?
A. Oh, yes, absolutely. As I've said, I'm not a statistical expert but I use statistics quite a lot, and as a firm we use it quite a lot, statistics, the application of statistics.
Q. Because, as I understand it, whilst it may have been Arup that suggested or proposed the binomial approach, it was others that, as it were, set the bar for the pass or fail mark?
A. Oh, yes. No, no, ours was just a suggestion on methodology.
Q. All right. Could I ask you, please, to be shown paragraph 7.11 of your first report, the COI 1 report.
A. Yes.
Q. I think, Dr Glover, to some extent you've probably covered this in your slides, but at 7.11(i) --
A. Yes.
Q. -- you say:
"From the results of the extensive testing of the coupler connections by MTR and others, I am satisfied

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that a coupler connection with an engagement length of 32mm will achieve the full strength of the connection, and satisfy the full range of strength tests specified by the relevant code AC133."
And that's obviously something that Prof McQuillan and Mr Southward agree also?
A. Yes.
Q. Then if we could go to the very last subparagraph, that is (xv) --
A. Yes.
Q. -- you say:
"For the above reasons" -- and obviously there are extensive reasons set out which I am not going through -- "I conclude that the coupler connections in all parts of the HUH Station are both fit for purpose and safe, including [the important words] the EWL coupler connections in area A."
A. Correct.
Q. As I understand it, as you've explain in your slides and you explain in your report, that is because you cannot see any differentiation between the fixings in B and C and area A?
A. No. And if you look at the data, that's the conclusion I think other engineers would arrive at, or most other engineers because there clearly are exceptions.

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Q. And also that's underpinned, as you've explained and I just want to make sure I've got this clear, because of the limited number of samples taken from area A and your view that the purpose (i) tests in area A could sensibly have been taken?
A. I believe they were never even looked at, from this point of view.
Q. The consequence of doing that would have been to reduce the failure percentage down to something like 23 per cent?
A. 23 per cent, yes.
Q. All right. Then we can reach our own conclusions as to doing the work in area A as a consequence.
A. Yes.
Q. Okay.
A. It is on the basis of the 32 millimetre engagement.
Q. Indeed.
A. I don't want that to be misunderstood.
Q. I understand it's on the basis of 32 millimetres. You made that very clear.
CHAIRMAN: I'm just wondering, 15 minutes now, 4 o'clock?
MR PENNICOTT: Yes.
CHAIRMAN: We will need to stop this evening fairly sharp, just a minute or two before 5.00.
MR PENNICOTT: All right.

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CHAIRMAN: Thank you.
(3.58 pm)
(A short adjournment)
(4.15 pm)
MR PENNICOTT: Dr Glover, could I then, skipping over the elongation test which I was going to ask you about but I will not in the light of what you told us by reference to your slides.
Could I ask you to look at paragraph 7.15 of your COI 1 report. This is in the section where you are dealing with the single data set --
A. Ah, right.
Q. -- which I've touched on a moment ago, in fact. You start off by saying at 7.13:
"Coupler connections are widely used in the construction industry, and as such they may be considered as a standard product with an experienced workforce available to execute the construction thereof."
Then, passing over the next paragraph and going to 7.15:
"Notwithstanding, it is a reasonable judgment to make that the variation in coupler engagement will be, in an engineering judgment context, for a particular site and conditions applying thereto the same throughout

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the construction process whether that be [in] the EWL area C or the NSL area A and observation of the similarity in the distribution of the data from the NSL and the EWL supports that conclusion."
Now, Dr Glover, I understand what you say about it being a standard construction process and standard materials, and so forth, but the points have been made, have they not, that so far as the sub-contractor in area A and -- the steel fixing sub-contractor in area A and steel fixing sub-contractor in area B, they were different, do you regard that as relevant, different sub-contractors?
A. I do consider that to be relevant, but I did not understand them to be different.
Q. The works in area A were carried out at a different time than B and C, about a year or so apart; would that be relevant?
A. It would be relevant if the workforce had changed substantially, but the construction sequence was continuous. I mean, for example, if it had been phased in the sense that a piece of work had been completed and then it had been returned to another part six months later, you would have lost what I consider to be the conventional wisdom of how to build on that side, but that's not the case. Construction was continuous.

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Q. All right. Would you agree that the slab that was being constructed in area A is a lot less deep, thick, than the one being carried out in areas B and C, that is --
A. Yes, I would. I would accept that.
Q. Would you agree, therefore, that the conditions were materially different in terms of fixing the rebar into the couplers?
A. No, I wouldn't agree with that. I would say, when you say materially different, that conjures up a picture to me that they are dramatically different. But the physical operation was still going to be the same, particularly at the EWL level.
The one situation which stands out is the points that I made, I hope I made, when I gave my presentation, that I would have expected area A to show a better performance than the other areas, but that doesn't mean to say that I would have expected to see it dramatically different.
You would have had the same basic distribution, it's just that it would have been, I would have thought, slightly better.
Q. That's for the reasons you gave?
A. It's less congested.
Q. Easier access, less congested?
A. Most importantly, both sides.

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Q. And being able to visually --
A. Touch it, yes.
COMMISSIONER HANSFORD: The point being that the holistic report -- not the holistic report, the verification report demonstrated or suggested it was materially worse and that --
A. I see, yes.
COMMISSIONER HANSFORD: -- appeared to be an anomaly; is that the point?
A. Yes, I mean, when you have lack of -- I hope I'm answering your question; stop me if I go off down the wrong track.
If you have isolated bars that you can see, you can touch both sides of it, particularly if they're in the vertical instance as they were in the trough walls, is very difficult to -- certainly in the trough walls, they couldn't be unconnected, because they would fall over. If it isn't screwed in, it's just going to fall out. There's no doubt I was expecting that area A would have a better performance. When I say "better", I mean not enormously different but that certainly a less -- a better performance than in the other areas. But again your words, "materially", no, not materially different in the sense that they would say, "Oh my goodness me, this is a different situation". No, I would have

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expected it to be mild because it is the physical operation of screwing something into something else, it's related to human strength, it's related to the way one positions oneself. It's almost an ergonomic problem and as a consequence I believe, my opinion, that you would tend to get a statistical distribution on the amount of engagement that would have taken place.
MR PENNICOTT: Yes. You mentioned the trough walls during the course of that answer.
A. Yes.
Q. Would you accept that that work or the conditions in which that work was carried out in the trough walls was very different to the slabs in area A, B and C?
A. I see some similarities in the sense that I described to you, which is that with area A you could see both sides of it, but the most important thing is the size of the reinforcement was so much smaller. I mean, we are talking about -- compared to area A and areas B and C, the reinforcement is a phenomenal size, and it's vertical.
Q. Yes.
A. So it's visible, it's touchable, inspectable, at all stages of construction. It's not hidden up by subsequent layers. That's why I was surprised that it was penalised in the way that it was. But I believe

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that was not because of a structural reduction factor. It was because there was a lack of records, and I can relate to that, but 35 per cent is very arbitrary.
Q. Yes.
A. I'm sorry, I'm not sure if I'm really helping you, Mr Pennicott.
Q. It's just that we know that the reduction factor of 35 per cent has been taken from the investigations that have been carried out not in the HHS, not in the trough walls.
A. Yes, I wish you had asked me that to start with, because that would have helped me meandering around. No, it was an arbitrary decision to do that, because there is no relationship between the 35 per cent. I mean, it isn't. The fact that they are the same -- I assumed that the fact that they were the same number was coincidental. I didn't think people would have extrapolated it from one and put it in the other, because there is no technical basis for doing that.
Q. No. Quite. All right.
A. But there is an arbitrary basis. If you want a number and it's floating around, you select it, I guess.
Q. It's arbitrary?
A. It's totally arbitrary.
Q. There's nothing else, that's all there is, on one view?

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A. You could. You could also say -- you could open the works up slightly, if you were that concerned.
Q. Yes, quite.
A. And you wouldn't have to do very much to do that, would you? No, I think the 35 per cent is slightly penal, from what I would think, but is it really of a physical consequence? As you can see from my response earlier, I think the problem has been approached from the wrong direction.
Q. All right. Just one last point on the trough walls. You will have heard that one of Dr Lau's concerns is in relation to the columns and the potential damage that might be caused or deflection that might be caused by a train derailing, having impact with the trough wall and then adversely affecting the column.
A. Yes.
Q. Do you have any observations about that?
A. I have a number of observations about the detail that was constructed. I mean, if you are that worried about it -- and I have not studied the drawing in any great detail so I could be wrong here -- but you certainly would leave an air gap, wouldn't you, behind that wall and the column?
Q. There is certainly a gap. I think Dr Lau told us 50 -- 60, I think it was.

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A. There's a distance between the two but whether it's a gap I'm not sure, because the whole area is filled with soil, and also the oversite concrete which was put on top of the soil is in rigid contact with the column. But this is before the work. I would say that if you really were -- and I'm not suggesting anyone wasn't concerned about the columns, I'm sure they were, but if that was the case you would have certainly left a void, not a gap, a void, between the wall and the column, and you would have gone further than that, you would have made sure that the oversite concrete that's at the top of the wall certainly didn't contact the column, and I have only been to the sidings once but my observation was that the oversite concrete was indeed cast around the column, so why would you do that if you wanted to isolate them?

The other thing is -- I'm sorry, sir, I'm going on a bit.

Q. I asked you to comment.

CHAIRMAN: No, carry on.

A. To be quite candid, you would have put an isolation zone around the column, wouldn't you? You have sleeved it with a gap, not just on the front face but all around. Because, as I've pointed out, why I think it's -- I hate to say this because these things get misunderstood. Why

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I believe the solution lies in looking at the model more appropriately is because if you wanted to isolate something, you would make sure that it didn't pick up all sorts of secondary effects. Now, that column is not isolated. None of them are isolated. They've all got the soil compacted around them and the oversite concrete contacts them. No allowance has been taken of the fact that the oversite concrete, which I know in official terms is not structural and therefore is neglected, but when you do a dynamics problem, you include everything. That means that if a train does impact, that oversite concrete and the soil will press on anything it possibly can, which includes the adjoining walls and the column.

If you were that concerned about it, you would put not a sleeve, you would put a gap all the way around the column, and I'm not aware that there is one, and the drawings are not that clear, actually, so I back away from actually saying definitively that it's not there, but that's what I've observed. I hope that makes sense to you.

MR PENNICOTT: It does make sense and I think our understanding is the same as yours in terms of what was physically there in the first place.

A. So, as far as I'm concerned, one is looking at that particular problem from the wrong end of the telescope.

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Q. Yes.

A. One's looking at it purely as a piece of concrete, cantilevering in free air and not really taking account of the real situation that one has before one, and also not really understanding what a train derailment looks like.

Q. Okay. That's sufficient for my purpose.

A. I'm sorry, I don't want to cut off --

Q. No, that's fine. I just wanted your comments on the column and we've got those comments. Thank you very much. That's very clear.

A. Thank you.

Q. Just a couple of questions on the construction joint and the ... (unclear word) --

A. Okay.

Q. -- and the dowels which you covered in slide 28. Could we have a look at your slide 28, please.

You see in bullet point 2 --

A. Yes, sorry.

Q. -- Dr Glover, you insert the word "nominal" before the words "dowel detail", and I assume you have used the word "nominal" advisedly. Why do you describe it as "nominal"?

A. It's nominal when you consider the mass of the construction that's there. I mean, it's a 1.2 metre

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wide wall, you know, 4 foot wide (demonstrating). It goes on for hundreds of metres, and one's inserting a 25 millimetre diameter bar, I believe, at something like 600 centres, when you've already got the sort of reinforcement I've already indicated, and those of us who've gone to the site know how much reinforcement is there, and they act as dowels.

So I say "nominal" because it's -- in comparison to what is already there and the scale of the project, it's nominal.

Q. Structurally insignificant?

A. Structurally certainly insignificant in terms of the safety criteria of the structure.

Q. Right.

A. But as I've said, people have got other criteria which they wish to apply, and as I said earlier I'm not going to stop my neighbour playing football in his garden.

Q. Understood.

A. It's their decision. But you've asked me, this Commission, to give my opinion on safety and fitness for purpose and I've said that the existing arrangement certainly satisfies those two requirements. I do apologise to anybody for any offence on "nominal".

Q. Not at all. I just wanted to make sure I understood what you meant and I do now.

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A. Okay, that's good.
MR PENNICOTT: Thank you. Just give me a moment to see if there's anything else.
Thank you very much, Dr Glover. I have no further questions.
WITNESS: Thank you very much.
CHAIRMAN: Yes, Mr Chow.
Cross-examination by MR CHOW
MR CHOW: Thank you, Mr Chairman.
Good afternoon, Dr Glover.
A. Good afternoon.
Q. As you may be aware, I represent the government and there are a few topics I would like to discuss with you this afternoon.
Dr Glover, the first topic I would like to explore with you relates to the partial factor of safety that you have taken us through earlier in your presentation. If I may refer you to the relevant part of the Concrete Code, at bundle H8, page 2840, please. Go down a little bit. Clause 2.3.1.3.
Dr Glover, you know that the various partial factors of safety are set out in the Concrete Code?
A. Yes.
Q. As to what those factors account for, I can only refer to what is set out in the Concrete Code, and the

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particular provision I would like to discuss with you is 2.3.1.3. You see the paragraph starting with that symbol gamma f; do you see that?
A. Mm-hmm.
Q. This is where they talk about what factors that the partial load factors account for, and what is set out here in this part of the code is that it provides that the partial safety factors take account of unconsidered possible increases in load, inaccurate assessment of load effects, unforeseen stress redistribution, variation in dimensional accuracy and the importance of the limit state being considered.
Do you see that?
A. Yes, I do.
Q. Do you agree that the description given here is exactly the same as the description provided in the British code, BS 8110?
A. I'm sorry, I can't confirm that, but I should imagine it looks very familiar.
Q. You can take it from me because I have compared the two versions.
A. Okay, I've got no problems with that, but could I just add: you are using "gamma f".
Q. Yes.
A. The point I was making earlier is that gamma f, although

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it's referred to as a partial safety factor, it is a combination of two other partial factors, and the description that's written there is a -- I hate to use the phrase again -- but one size fits all. It's when you start to break down the gamma f that you start to get the constituent parts. So gamma f is the result of considering other factors. As long as that's understood.
Q. Yes. May I take you through the details of these various factors, just to get an appreciation of if there's any reduction, the extent of reduction that we can have; right?
A. Yes.
Q. The first item referred to here, regarding "unconsidered possible increases this load" -- now, in respect of this factor, the fact that we have now carried out a post-construction structural assessment, there remains risk in relation to possible increases in load; correct?
A. Yes, because that's why we have the load factors.
Q. That's right.
A. If I can just draw attention to the fact that you've got the dead load factor there and you've got the live load factor there, and each one will have a different risk level associated to it.
Q. Certainly, yes.

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A. But the one that I -- and I have not challenged the live load one, if that's where you wanted to go. I'm saying that is a factor to take account of the future, because that's about the operations.
Q. Right.
A. No, the issues I was focusing on particularly are the dead load factors, particularly as applied to this structure, and in those considers -- when you say the "unconsidered possible increases in load", then when you are talking about the dead load gamma f, then it's not really conceivable that you are going to have -- let's call it a 40 per cent increase in the loading, are you? But if it was a live loading consideration, then most certainly I wouldn't be challenging it in a post-construction stage, but I most certainly would be looking at the dead load, because I've now got information which I didn't have.
If I don't watch out, I'm going to answer all your questions before you've even asked them. I think I should stop. I'm sorry. You know me well enough. I'm sorry. I will shut up.
Q. I am coming to that. As you are talking about dead load, I notice that in the various factors listed out here, one of those factors relates to the dimensional accuracy, so that would go to the question of dead load?

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A. Yes.
Q. My understanding, in fact it's my instruction, is that MTR's consultant, Atkins, in carrying out the stage 3 structural assessment, Atkins has not actually gone out to site, taken measurements as to the real dimensions of various structural members so as to take into account the possible variation in structural dimension. If that is the case, do you agree that the risk in association with variation in structural dimension remains notwithstanding the fact that the stage 3 structural assessment is concerned about post-construction structural assessment?
A. But inspections have been made, Mr Chow. They have been made. I've seen the surveys. So the knowledge is there. Whether they are taken account of in the analysis or not is another matter, and I did say in my presentation that I wasn't seeking to actually apply these to the Hung Hom analyses. I was just saying I believe it would have been appropriate if I had chosen to, and if I had chosen to I would have ended up with even less levels of stress.
So I hope I'm answering your question. I mean, is it? I haven't used the opportunity to use a reduced dead load. I have not done that in the analysis. I've just pointed out that it would be reasonable to do so

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and, as a consequence, the stress levels in the structure would be even lower than we have given.
Does that help you?
Q. Let's see if we can cut short my questioning. Can I ask this: do you think, in the present circumstances, the fact that Atkins took the partial factor of safety as set out in the code was reasonable? I mean the stage 3 structural assessment.
A. It's not unreasonable to have done that. I'm saying that they could have approached it in a different way, but as part of the updated design we had these discussions, and some of our suggestions were not taken forward into the updated design. My understanding for that was that the updated design should be -- let's call it a compliance analysis, in the sense that it was intended to demonstrate to the clients -- government, MTR, et cetera -- that the as-constructed structure was indeed satisfying their requirements.
But if I was doing a thorough -- you know, an absolute grass-roots approach on safety and fitness for purpose, I most certainly would have reduced the dead load coefficient from 1.4 to something more around 1.25, thereabouts, and that would have been based on measurements of dimensions and a justification, and I wouldn't have taken full advantage of it all. I would

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have still left something back in there.
But if you stand back from here, if I had a 200 millimetre thick slab, I designed a 200 millimetre thick slab, the variations in the concrete thickness, it wouldn't be unexpected for it to be 20 millimetres, maybe, you know. So therefore the highly variation on thin things is quite high, but the likely variation on thick things is very low. And all I'm saying is that in the way in which the codes are written, for good reasons, it is one size fits all. It doesn't ask me to consider those things.
All I'm saying is, in a forensic situation, you look at the physical facts of what has been constructed. That's all.
Q. Let me see if we are an agreement on the following. In relation to the partial load factor for the design load, the fact that the structure has been built, someone could have gone down to site, taken exact measurements of various members and then work out the value of the exact dead load and enter into the usual calculation. Certain degree of risk would have been taken away in relation to loading. But one cannot objectively assess the extent of reduction of the partial load factors; can we agree on that?
A. No, because there are two limit states we consider. One

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is strength, and therefore, when you carry out the strength assessment, it is perfectly reasonable to use physically what you observe. The check is that that reduced load has then got to be used within the serviceability calculations, and if the serviceability calculations show that you are not overstressed, then everything is okay, you haven't reduced the factors at all, because the risk -- risks are all measured from the point in time that you view them and, when you start with something, your risks are much larger than when you are further on into the period. So, therefore, you've actually reduced the amount of risk that you were exposed to at the outset.
Q. Yes, precisely. Perhaps I didn't make myself clear. Because certain degree of risk cannot be removed because --
A. We can agree on that.
Q. And when it comes to the partial load factor, we appreciate that perhaps some kind of reduction can be applied in structural assessment.
A. Mm-hmm.
Q. But the point that I'm trying to get your agreement is one cannot objectively assess the extent of reduction? In other words, we can't precisely point to a particular value; instead of 1.4, now we use 1.27 instead of 1.3.

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Can you agree with me on that?
A. We're agreed on a principle, which is the risk profile is less.
Q. Yes.
A. Okay. So that's good. So that's a qualitative statement, it's not a quantitative statement.
Q. Correct.
A. You are then saying what is the basis of arriving at that quantification; yes?
Q. I'm saying there is no objective way to determine this, to quantify the reduction.
A. Okay. Well, I would refer you to the partial factors that make it up. As I drew attention to in the appraisal of structures, for example, it does say that the variation could be the difference between 1.15 and 1.05, and you do that on the basis of your expectation of the variation in that load going forward.
Now, if you go to my 200 millimetre thick slab, I would say that's at quite a high risk of being exceeded and so therefore I wouldn't be reducing the load going forward. But when I'm dealing with something which is 3 metres thick, then I would be going for the 1.05 as the partial factor, not 1.15, and the justification would be that's where it is. That's what I've -- that's the physical fact.

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Q. Right.
A. But I would most certainly look at the serviceability stresses. I mean, I think you know this, Mr Chow: when the codes were re-drafted back in the early 1970s, they were drafted from the working stress basis upwards. So, in other words, one didn't want to change the stresses at working stress level. So the factors were actually back-fixed so you got the same answer.
So the most important thing in our structures, interestingly enough, is the serviceability state. In other words, the stresses that the building is under now.
Q. All right. Let's see if we can simplify the matter. The point I would like to make is that -- now, we can see from the code that the partial safety factor for load actually encompasses a number of factors that it's supposed to take into account; right?
A. Mm-hmm.
Q. Not just one factor which goes to, for example, the dimension of the structure in order to determine the actual weight of the structure. It encompasses other risk factors which includes inaccurate assessment of load effects, unforeseen stress redistribution, all these factors remain the same, even in a post-construction structural assessment?

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A. Okay. Let's take them one at a time. We're agreed on the risk profile reducing.
Q. Yes.
A. That's good. I've now got this structure in front of me and I can see it and I can touch it, so I know what the dimensions are.
Q. Yes.
A. I can look at the geometry of the connections and whatever and I've got greater assurance that they are as I determined. So, yes, I've got greater confidence.
But the point I made earlier about the 1.2 factor which is in there -- and I'm not sure why you want to concentrate on the partial factors anyway, but I'm quite happy to talk about it all day if that's necessary -- but the 1.2 factor that is in there for the sort of issues you're discussing or describing is in there for the analysis accuracy, but the same figure applies whether I was to do it on the back of an envelope or I use the very, very sophisticated analysis.
That's what a code is about. It's a one size fits all. It doesn't give me brownie points for rigour. It does that.
All I'm saying to you is -- we can continue with this conversation -- I would like really like to get to the point you want to make, because otherwise we can

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talk about this for a long, long time. What is it you want me to answer?
Q. The point I've actually made already is that even if some risk in association with load, design load, has been removed, but it is very difficult to quantify the extent of reduction in the factor of safety.
Can I suggest to you that given the difficulty in quantifying the reduction, it would be reasonable for Atkins to adopt the same partial factor of safety in stage 3 structural assessment?
A. I've already said that it was reasonable for them to do that. You and I agree that the risk profile is less and there is a basis for suggesting that that is a conservative decision; yes?
Q. Right.
A. You and I are not agreed on how we quantify that, but I've already expressed how I would have done it.
So my answer to your question, which very succinctly you put that, that's good, I can answer it -- I think it's reasonable for Atkins to apply the 1.4 for dead load, in that analysis. It's also equally appropriate to say that that is a conservative decision, in comparison to the situation that was at the design stage.
Q. For the same reason, when it comes to the partial

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material factor -- perhaps we can quickly look at the corresponding provision, 2.4.3.1 at page 2843.
A. Yes.
Q. Here it sets out what the partial material factors accounts for. Again, a number of factors, not just single factors. It provides here:
"For the analysis of sections, the design strength for a given material and limit state is derived from the characteristic strength divided by gamma m, where gamma m is the appropriate partial safety factor given in clauses 2.4.3.2 and 2.4.3.3. Gamma m takes account of differences between actual and laboratory values, local weaknesses and inaccuracies in assessment of the resistance of sections."
Now, again, a number of factors are involved --
A. Mmm.
Q. -- and for similar reason that we have discussed earlier in relation to the partial load factors, it would be reasonable for Atkins to adopt the same partial material factors in stage 3 structural assessment?
A. I would agree with that in this instance. My reference to the latitude that we have in Eurocode, it was because we could be assured of a higher level of quality assurance, and that most certainly isn't the case here. So I would actually support 1.5 in this instance.

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Q. Right. Thank you.
Earlier, you also mentioned about the load and risk in association with construction sequence, that sort of thing; in a post-construction structural assessment we no longer have this risk or uncertainty in relation to construction sequence.
If I then -- you are aware of the updated design; right? Under updated design, there is a new set of design parameters. As I understand it, when MTRC and the government derived this updated design, they have already taken into consideration the fact that the construction stage has already passed, so the unusual loading or more critical loading cases that existed during the construction stage have not been considered under the updated design. If that is the case, do you accept that the uncertainty arising from construction sequence and construction stage has already removed and actually the effect has been taken into consideration when the updated design was agreed between MTRC and the government?
A. Okay. You know that concessions, or let's call them concessions, in the updated design were pretty cosmetic. Okay? They did not did not take account of what we observed as for the soils, for example. The recommendation from the -- sorry, I'll take out the word

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"recommendation" -- the observation from the consultants was that using E equals 1 N was inappropriate. We obviously went further than that on the basis of the evidence we had. So that part of the construction was not taken into account.
The other thing is that although some lock-in was taken into account, we were very clear that that stage of construction should have been considered as a serviceability condition.
I'm not sure if I'm answering your question but what I am saying is yes, it was post-construction but the updated design principles did not reflect what we had learned from construction.
MR CHOW: Thank you.
Mr Chairman, I only have one short matter I think I can deal with in one or two minutes before we break for the day.
CHAIRMAN: Okay, but we will finish at 1 minute to 5; all right? I'm not watching the time, of course.
MR CHOW: I am keeping the time.
Dr Glover, in relation to the partial material factor, do you agree with me that the factor of safety does not account for defects in the concrete?
A. It takes account of irregularities and variations, but if you are referring to things like honeycombing and

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whatever, no, that is an obvious defect and has to be put right. The works have to be rectified, reinstated actually -- maybe "reinstatement" is a good word because it reflects the fact that the structure has to be put back to what you assumed it to be.
Does that answer your question?
MR CHOW: That answers my question. Thank you very much.
Mr Chairman, I think this is a good point to stop.
CHAIRMAN: Good. Thank you very much indeed.
We leave the matter then until tomorrow morning, Dr Glover. You will have to return. Thank you very much.
WITNESS: Thank you.
CHAIRMAN: Good. Tomorrow morning, 10 am. Thank you.
(4.56 pm)
(The hearing adjourned until 10.00 am the following day)

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