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Wednesday, 8 January 2020

(10.03 am)

DR MIKE GLOVER (on former oath)

MR CHOW: Good morning, Mr Chairman and Prof Hansford. Good morning, Dr Glover.

Mr Chairman, this morning Dr Glover indicated to me that he would like to further assist the Commission on some details regarding the partial factor of safety, and I indicated to him that I have no problem with that, subject to the agreement of the Commission. I understand he has already written up something on the board.

A. Yes, behind. It's not there.

MR CHOW: Subject to the agreement of Mr Chairman and the Commission.

COMMISSIONER HANSFORD: It would be helpful.

CHAIRMAN: All right, Dr Glover, we are happy for your explanation, but please work on the basis that I'm like the dumbest student in one of Einstein's classes trying to understand the relativity theory.

A. You had a good mentor, sir, if it was the great man himself.

I took the liberty of preparing it this morning, because it takes time to write things up and it obscures the view. What I wanted to try to do this morning is to

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get across some of the algebra involved, because I recognise this room is a room of words. My world is a world more of algebra. But I hope with some arithmetic we can bridge the two, to explain some of the concepts in this dreadfully opaque slide that I showed yesterday, which all credit to Eurocode, they make it as simple as they possibly can.

I want to focus on the gamma F and the gamma M factors. Gamma F is the load factors, gamma M is the material factors. I want to explain very simply how we use them in design and also what the repercussions of that are in a forensic situation. So that's my objective. Fingers crossed that I'm successful with it. It's my arithmetic that will fail, if anything.

The subject is -- leave aside issues about partial factors of safety -- at the end of the day we as engineers and the community at large are interested in one thing: what is the reserve of strength? Is it teetering on the brink or does it have a large margin of strength, a large margin of capacity? So I talk about reserve of strength.

In Eurocode language, they use two words. One is called resistance, and that to you and I would be strength, what is the strength of something, what is the resistance. The other is the actions, the forces, the

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loads. So I've just called them -- apologise for the algebra -- R and A.

So, when we are designing something and we have reached let's call it the optimum design, the balance, you end up with an equation which looks like this, and I do apologise because it is algebra again, but the resistance is divided by a material factor, gamma M, and gamma m is taken as 1.5. I think you've heard that before. That's not new, that's tradition.

Then that is divided by the actions. In other words, you're trying to look at the ratio between the two. But the actions are then multiplied by a load factor, gamma F, which is taken as 1.4 for dead load. So that's the equation, and if it's 1 you've actually got it right on the balance.

COMMISSIONER HANSFORD: Sorry, can you just explain what you mean by "right on the balance"?

A. It means that would be acceptable in terms of the codes. It's a compliance calculation, in that sense.

Now, to try to help change the algebra into numbers, I've invented some numbers which arrive at the same situation. So I've said let's call the resistance 150 units. It doesn't really matter what they are but 150 units. And let's call the actions 71. Now, I've selected those numbers because when I then put in 1.5 as

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the material factor and I put in 1.4 as the load factor, I get back to 1. So we are starting from that base and I'm now the designer. I'm in my design office. It's gone to BD. They've looked at my calculations; tick in the box.

Three years later, I've constructed the edifice. Fortunately for Hung Hom, I've carried out umpteen experiments, tests, investigations, measurements, and I'm in a forensic situation. So I now have information.

So let's look at how that can potentially affect that equation, because that's what it's all about. When we talk about all these partial safety factors going backwards and forwards in the banter -- and I do enjoy the banter with Mr Chow -- I can see the eyes glaze over but it's better if I try to explain it in physical terms.

CHAIRMAN: Yes.

A. So if we go to the next stage, I'm now in a forensic situation. I've got a 3 metre thick slab -- it is 3 metres and 20 millimetres thick -- and I'm thinking to myself I've got this load factor of 1.5 gamma F, and I could understand why I needed that at design stage because I didn't know what was going to happen during construction, because the 1.4 is intended to include things like construction loading and people taking

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a rogue crane across the site. That's what it's for. But now that's gone, so I can look at the parameters that are before me and make judgments.

Therefore, I'm going to look at gamma F, and when I explained to you yesterday -- badly, I think, but no matter -- gamma F is made up of two factors, one to do with dimensions and the other one to do with ignorance, how I analysed it and whatever. So if I just look at the first of these, dimensions, then the make-up of the 1.4 originally is made up of 1.15, the dimensions one, and 1.2 for the analysis one. Hence, 1.15 times 1.2 is very, very nearly 1.4. The code drafters rounded it to a number. You wouldn't want to walk around the world with 1.38 in your mind all the time.

So to all intents and purposes that's what it is. But now I've reached a situation where I know what the dimensions are, and I also know, very importantly, that this is a dead-load-driven design. In other words, the ratio between the live load that's going to come on and the dead load that's there today is dominated by dead load. So that's why I'm focusing on the dead load factor. I could do an equal analysis on the live load but it's just not worth going there for this purpose.

So I'm saying I'm not going to take full advantage of the fact that it is 3 metres thick or 3,010 rather

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than 3,000. I want to leave something left. So rather than using 1.15, rather than using 1, I've said okay, let's go for 1.05 as a factor. I'm not going to touch the ignorance factor, the 1.2, although I feel on the basis of the enormous analysis that's been done, not just by individual parties but by different groups of consultants using different pieces of software -- but I'm not going to go there; I'm going to leave that as it is.

So the net result of 1.05, leaving the 1.2, is that the gamma F reduces to 1.25.

Now, how does -- and I've introduced it into the equation. Everything else remains the same. I haven't said it's got stronger. I haven't said the loadings have changed. But just by changing that, I get 12 per cent reserve of strength.

Go one stage further. All the test evidence would tell me that actually the concrete is a hell of a lot stronger than it was, and so I'm saying: okay, let's assume that it's 20 per cent stronger, because it appears to be 20 per cent stronger without any age factors. My resistance now has also increased by that 20 per cent. It's gone up to 180.

If I then put 180 into the equation, everything else remaining the same, I get 35 per cent reserve of

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strength. That's the crux of the banter that's going backwards and forwards. People are saying, "You can't do that because you are reducing the safety on my structure", but good grief, that is the results in pure engineering mathematics, that's what it is. There's no emotion in it. I could go much, much further than this. I could start, as I said earlier, stripping away at the 1.2, whatever.

The thing I've got to assure you on on all this: it does not change what we call the SLS condition, the working load condition. The design for the working load condition, as I said yesterday, towards the end, is that actually, at the end of the day, the thing that really matters is how the structure performs on a day-to-day basis, provided it's got a satisfactory reserve of strength. So I haven't changed any of that. The working load stresses are exactly the same as they are. All I have done is demonstrate the structure has an enormous reserve of strength, rather than, we were led to believe by some parties, a decrease in the reserve of strength.

I'm sorry to have taken the Commission's time on that.

CHAIRMAN: No. It's helped a lot, actually. Thank you.

A. But I just wanted to get some -- and it is for Mr Chow

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to examine that if he wishes. I think the arithmetic is pretty basic, isn't it, for you and I, that is; yes?

MR CHOW: Yes, it is.

A. That's all I wanted to say, sir.

CHAIRMAN: Can I just ask you one thing. You are talking there about the concrete strength resistance and how it's increased, and tests, so we're all agreed, are we, that tests have been done on the strength of the concrete, not of the reinforced concrete but just of the concrete?

A. Oh, yes, I'm sorry. I used that as sort of an example. But the answer to your question is the 6,000-plus cubes that were carried out demonstrate that there was a shift of strength. The cores that were taken in the diaphragm wall that I have referred to yesterday show the type of correlation you would expect between the design strength, which actually was about 36, I think, and the reality on site, you know, the actual cube strengths, which were turning out at sort of -- I think the mean was 79 and the actual characteristic, this is the 5 per cent or the 95 per cent passing was about 62.

So that's what I'm referring to, the inherent strength within the structure.

CHAIRMAN: That's right. I just wanted to make sure that, as I understand it, it's not disputed that there have

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been numerous cube tests done over the period of time, and the strength of the concrete itself, as opposed to all the rebars and various other things, is not in dispute. That's meant --

A. It doesn't appear to be. I think the real discussion is: yes, that might be the case, but that's not what we do. In other words, that they're quality assurance tests and we don't include those.

CHAIRMAN: I appreciate that, yes.

A. So all I'm saying is if you -- but I'm not relying on that, sir. I'm just trying to demonstrate.

CHAIRMAN: Thank you very much.

COMMISSIONER HANSFORD: Just on that final point, though, the question from the Chairman, if anyone were to suggest that the concrete in these structures is somehow of substandard strength, what would your response be?

A. Substandard?

COMMISSIONER HANSFORD: Yes.

A. First of all, I would say anybody can make a statement like that. Could you show me why that is the case?

CHAIRMAN: Fine. What you know about this, about the box, the station box and anything else, the other aspects of the building that we've been looking at, has anything been put before you --

A. No.

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CHAIRMAN: -- has anybody said anything to you --

A. No.

CHAIRMAN: -- or alleged to you that's resulted in your investigation that the inherent strength of the concrete itself is below par?

A. No, not at all, and I think using the honeycomb as a metaphor or a facsimile for the strength is an error. The honeycomb is indeed extremely poor workmanship. I would use the word "extremely" because it can be avoided. I think I said this at the first hearing: the aggregate size was probably too large for the density of reinforcement, and it seems as if an admixture, a simple plasticiser, was not used to enable the flowability of the concrete.

But that doesn't change the strength of the concrete which would have been demonstrated by the cube strengths. Even taking into account the fact that conditions might have been slightly different one way or the other, the cores from the diaphragm wall really represent a very good comparison between what I would say are the on-site curing situations. And I know somebody is going to come back and say diaphragm walls cure in a different way but when we talk about a 3 metre thick slab, the conditions of curing are not that different from a diaphragm wall.

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CHAIRMAN: And the conditions that have been shown in respect of the diaphragm walls show that everything is okay as to the essential strength or resistance of the concrete used in the diaphragm walls?

A. Absolutely, sir, and those cores demonstrate it beyond any shadow of a doubt, I would think.

The other thing is -- I'm glad you've brought up the diaphragm wall because I'm not really sure whether I did it justice yesterday because this issue of the diaphragm wall and cracking in it causing distress elsewhere in the structure came as a completely -- I thought we had dealt with the diaphragm wall a long time ago, but just for the Commission's point of view, I see nothing wrong at all with the diaphragm wall. Indeed, if you go through the checklist of life, you will ask yourself: was the design carried out satisfactorily, in accordance with all the rules? Tick. Was it constructed by a competent contractor? Tick. Do we have all the documentation that we would expect? Tick. Was it accepted by the approval authority? Tick.

So I was amazed that it even came up in the presentations that took place, but I felt I had to go the extra mile to demonstrate that cracking is not an issue in the diaphragm wall, hence I showed those slides yesterday which really was a little bit outside

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of my brief in terms of discussing it, but I felt slightly outraged, to be honest.

CHAIRMAN: Could I just ask you one other thing about concrete. Dr Lau said, and I don't dispute it but I'd just like your comment, if I could, that yes, the fresh concrete goes in, you do your cube tests and it has a certain strength. That strength increases as the concrete settles and grows older, but then, like us, I suppose, at 25 we are running around a rugby field; at 75, even being on the field would amount to a physical activity. You know, we start to lose our strength, and concrete is the same.

A. Yes, yes. No, that's not the case. It plateaus, sir. It's true, you get a very rapid increase in strength. It's like a -- it's a parabolic -- mathematically, it's an asymptote, which means it gradually increases with strength over time and just almost runs parallel to something, so it flattens off. It's like the shape of the curve that Mr Southward showed for the stress-strain relationship of concrete. You get the sort of parabolic shape at the beginning and then it flattens right off.

COMMISSIONER HANSFORD: With no decline?

A. With no decline, no -- I mean, when I say --

CHAIRMAN: Even over 100 years?

A. No, and one of the reasons for that is OPC, ordinary

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Portland cement, hasn't been around for 100 years, but what has been around is pozzolanic cement. If you wander around Roman sites, for example, you can even touch the concrete, and it is concrete, which was made 2,000 or 3,000 years ago, 2,500 years ago, and it's still there. You would really have to give it a big thump. And its chemistry, sir, is different.

So you have these two things working together, OPC and the pozzolanic cement.

I could go into all sorts of things about the fineness of the grain -- of the grinding of the cement, it has a thing -- in other words the finer you make it, the more surface area it gets so the quicker it will set, and that's where you get the difference between, shall we say, rapid hardening cement and very slow cement. I'm not here for that, sir.

CHAIRMAN: No. Good. Thank you. Your comments have helped. Thank you.

(The witness returned to the witness box)

CHAIRMAN: Mr Chow.

MR PENNICOTT: Before Mr Chow continues, just to make it clear that -- I'm sure everybody behind me realises this -- all the diagrams that the experts have been doing from time to time are photographed at the end of the day and then reproduced into the bundle, just in case --

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CHAIRMAN: Good.

MR PENNICOTT: I just wanted to make that clear so that everybody knows.

A. I hope I've got the arithmetic right. I know Mr Chow will pick up any error.

Cross-examination by MR CHOW (continued)

MR CHOW: Thank you, Dr Glover. Are you sure you don't want to sit down?

A. No, I'm better standing up, otherwise I will start coughing.

Q. Thank you. It's really helps for you to set out all the details regarding the partial load factors, but before I move on to the next topic that I planned to do, may I pick up on one or two questions raised by the chairman earlier with you.

Do you remember the chairman mentioned whether you have seen any evidence which may cast any doubt on the quality of the concrete, and Mr Chairman mentioned honeycombing?

A. Yes.

Q. What we can see from the evidence -- if I may just give the page reference: bundle OU5, page 3262 -- paragraph 3.5.11 of the holistic report. What is set out in the report is that 19 per cent of the soffit of the EWL slab being inspected are found to contain

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honeycombing.

A. Mm-hmm.

Q. Given the extensiveness of the honeycombing that we found in the EWL slab, do you agree that it is rather unusual?

A. Well, I think the wording there -- it takes the words out of my mouth -- "very unsatisfactory workmanship". In fact, I've just introduced "very" -- yes, very unsatisfactory, and I think totally avoidable, but I still stay by what I said in terms of the strength quality of the concrete.

Q. Provided the honeycombing is rectified?

A. Oh, yes, absolutely. As I referred to yesterday, any cracking, for example, that you had observed of an extreme nature, then you would deal with it, as a rectification, but it's not a safety issue.

Q. Can I take it that if the honeycombing is not rectified, it would have a detrimental impact on the strength of the concrete?

A. Yes, but when I say "yes" to that, it does depend on the degree of the honeycombing. If it's superficial, the cover, for example -- and I think most of this was the cover of the concrete, in other words below the lowest bars -- then actually that has no -- that's cosmetic -- and fire -- but it has no impact on the strength.

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COMMISSIONER HANSFORD: Fire resistance, you mean?

A. Yes, I'm sorry, fire resistance, yes. So people don't like looking at reinforcement over their head, in that sense.

So, no, I think you've got to take the honeycombing in terms of its proper context.

MR CHOW: Yes.

A. And there's no doubt it has to be repaired, but it is by and large -- I hate to use this language because it sounds quite superficial but that's what it is -- it's more like a plastering exercise, to make sure we've got it back.

Q. Right. Earlier, you also mentioned about the possible cause of this honeycombing. You mentioned about the aggregate size perhaps is not appropriate. You mentioned about perhaps they should have put in a plasticiser to improve the --

A. Flowability.

Q. -- flowability. So it is something to do with the design mix; is that correct?

A. No, not -- no, because the design mix doesn't dictate -- I will correct myself. The design mix is dictated in terms of aggregate size.

Q. Yes.

A. But not the admixtures. The admixtures tend to be much

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more to do with the application. My observation that I would have thought the aggregate size should have been smaller was just an observation, but I think 20 millimetre aggregate with some of the concentration of nine layers of reinforcement would have asked some questions. I mean, the contractor, concrete contractor, probably just picked up the specification and said, "Ah, it's one of those", and the ready mix arrived and they hadn't really interrogated what the impact of the concrete would be on the specific reinforcement that was in place. These things happen but I'm surprised it happened quite so often.

Q. As a layperson, it's rather obvious to me that these honeycomb were discovered because they happened to be located at the soffit of the EWL slab. If there is honeycombing in the core of the concrete slab, then obviously there is no way that one would know the extent of the honeycombing.

Now, the fact that we have an unusual large extent of honeycombing that appears at the soffit of the EWL slab, would it suggest to you perhaps you have to start asking about -- or questioning the quality of the concrete as a whole of the EWL slab, and it wouldn't be prudent for us to simply rely on the cube strength that we obtained from the ready mix that was delivered to

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site to say that, now, because the cube strength shows a much higher strength, irrespective of the quality of the concreting work for the EWL slab, we nevertheless use a much higher concrete strength for the purpose of forensic assessment?

So my question is in view of the extensiveness of the honeycomb and the location of those honeycomb, would it be prudent or not to adopt the apparent higher strength of the concrete?

A. I can understand your point, and I like the use of the word "prudent". I like to believe I'm a prudent engineer. You've got to weigh these things up. I must agree with you that for a layperson, they would most certainly see a link between honeycombing and somehow a weakness in the materials. But I'm saying, actually, I can't see the relationship between strength and honeycombing. I can most certainly see a link between workmanship and the honeycombing, most certainly, but I wouldn't have extrapolated all the way.

The other thing I have to emphasise, and I should have emphasised this before, in all the assessments I have done of this structure, I have not taken advantage of any of this, other than when it came to the shear strength, because I think we were being posed with a silly situation, a silly problem to solve, that's all.

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But I have not used this in any of the assessments of the strength, and indeed I could start to add some of this in by just using a proportion of the concrete strength, for example, but I chose not to because, to be quite candid, there's so much reserve of strength in this structure that it can stand the test of applying some very, very silly rules in the loads that are applied.

So I can see where you're coming from. I'd like to believe I'm a prudent engineer. Using my prudence, I didn't apply it in the design, but I feel I could quite easily put forward a case to use it, and I most certainly think that if people maintain the position that there are no shear links in certain areas, then I can equally use the argument that it's very obvious that the concrete is stronger.

Does that help to bridge the gap?

Q. I would like to ask one last question on honeycombing, just to make sure that everybody understands. Now, you said there is no real linkage between honeycombing and the strength of the concrete, but as a layperson, if we see a concrete cube full of honeycombing being tested under the same test, I would expect that the strength of that cube would be much smaller or lower than a concrete cube without any honeycombing. Am I wrong?

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A. Mr Chow, you and I can agree on that, that if you test a bunch of stones loosely glued together, it most certainly won't pass the test, yes.

Q. So, in other words, can we infer that if there i honeycomb inside the core of the slab, then we should expect that the concrete strength at the location where there are honeycombs would be lower?

A. Yes. Well, it's interesting. You and I know what causes the honeycombing, and the honeycombing is a lack of flowability of the concrete which is constrained because the spacing between the bars, et cetera, doesn't allow the concrete to flow. In the core of this slab, that is not the case, and particularly at the top of the slab where we are most concerned, it's very visual and very obvious.

So extrapolating honeycombing at the base of the 3 metre slab and then saying, "My goodness me, we've got to declare the concrete inadequate in strength", I'm sorry, I can't buy into that.

Q. Other than the flowability of the concrete, would you agree that it also depends on the workmanship of the concreter, that they properly compact with the kind of rod, vibrator?

A. The vibrators, yes. But there's no reason -- when you're dealing with a 3 metre thickness of concrete and

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the sort of mix we have, it's very flowable for what I would say unconstrained situations, and I have no reason to believe -- when I've observed the surface of the concrete and whatever, there's been no evidence of what I would expect to see if it hadn't been properly vibrated in that sense.

I don't think the competence -- I don't think this is an issue of the operative on site. I think, you know, you've got to imagine he's standing 3 metres above, in other words more than the height of this room, and he's looking through some of the densest reinforcement I've seen at the top. He's got nine layers of reinforcement -- unimaginable, isn't it, really? -- at the bottom, stacked like this (demonstrating), 6 inches centre to centre. Actually the space is more like 90 millimetres square. How does -- he's got this poker, he's got this vibrator, and he's doing his absolute damnedest to get down there and do it. That's what I'm saying.

So this is not an issue of the workmanship of the individual involved. It is a question of the selection of the materials that the operative had to deal with. Now, I've made that sound like a statement. It's not. It's an observation. But I think, if you add all those things together, you will see that it does make sense.

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But the answer to your question is: the concrete strength, as exhibited by the cubes, is a very good facsimile of the strength which is in the structure, and the cubes or the cylinders, the cores that were taken of the diaphragm wall are a good indicator of what you would expect.

Q. Right. The other question that Mr Chairman just raised with you --

CHAIRMAN: Sorry, could I just ask, so that I know and that we know when we come to do this report -- as I understand it, there's no suggestion by Dr Lau that the concrete strength itself is inadequate?

MR CHOW: That's correct, Mr Chairman. But the point, from my recollection, Dr Lau's evidence that he has some doubt as to the quality of the concrete and therefore he does not advise to use a higher strength for the purpose of structural assessment, notwithstanding the cube strength that one obtained from the 5,000-odd cube --

CHAIRMAN: All right. And his doubt as to the quality of the concrete is based on ...?

MR CHOW: The extensiveness of honeycombing and the workmanship.

CHAIRMAN: Okay. Good.

COMMISSIONER HANSFORD: Could I just pick something up here, because this is actually very useful to me.

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So, Dr Glover, you are telling us that the honeycombing at the soffit, at the bottom of these 3 metre slabs is not a workmanship issue, it's a result of the density of the steel and the flowability of the concrete?

A. That's my observation, sir, but I don't think the operative really had a fighting chance.

COMMISSIONER HANSFORD: That's really helpful, because so far in this Commission the word "workmanship" or "poor workmanship" is being used quite widely, and I think it's being used as shorthand, without really understanding what it meant.

A. Poor selection of materials.

COMMISSIONER HANSFORD: Poor selection of materials for the situation that we find ourselves in, with such dense reinforcement at the low level of this slab.

A. Yes.

COMMISSIONER HANSFORD: But "workmanship" would be the wrong title -- "poor workmanship" would be the wrong classification of it, in your view?

A. Yes, very much so, and returning to something the Chairman said about the couplers, it's easy to say the operative's at fault, is slap-dash, it was a Friday afternoon and he wants to get away. That's not the case here. Workmen don't want to do that. It only comes

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back on them in the end so they want to do a good job.

What normally would have happened -- I'm sorry, I am extending my response -- but you would have carried out flowability tests. You would have done them off site. You would have identified these -- you remember I referred to the unknowns and how you rule them out -- you deal with them early, and this was going to be a problem, so you would have carried out flowability trials off-site, using certain admixtures, aggregate size, and you would have gone back to the approving authority and said, "Look, I intend to do this", and show the demonstration. But no, that wasn't. It was just, "Just hammer on, this is the mix, here is the ready mix truck". I am imagining it but you can see the situation, and the poor old operative is there with an army of vibrators, working through the night, the programme is -- we're pushing on, we've got to get on, he's doing his damnedest. Just imagine it. He's up there, he's looking through at least four layers and there's nine layers below. Not only that, the shuttering wasn't properly cleaned, you could see water on it and latents in the photographs. So I think it was just --

COMMISSIONER HANSFORD: The cleaning of the shuttering is a workmanship issue?

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A. I would agree with that, but that would have been quite superficial. You're familiar with these things. You would have got that sort of watering effect on the bottom. It wouldn't have caused the honeycombing. It would have caused the local softening and lack of hardness of the finish, in my opinion.
COMMISSIONER HANSFORD: Thank you. Just to round this off in my mind. You are telling us that the honeycombing can and should be repaired, and indeed it has been?
A. Yes.
COMMISSIONER HANSFORD: And that then remediates the situation to the situation that was expected in the design? It brings it up to the required standard?
A. It brings it up to a required standard, but the fact is that in some of the latents, there would have been a question about the lapping of some bars. They didn't use couplers, they used lapping bars.
COMMISSIONER HANSFORD: Yes.
A. And a lapped bar transfers its strength from this bar (indicating) to that bar (indicating) through the concrete.
COMMISSIONER HANSFORD: Indeed.
A. And if the concrete is not there, clearly it can't do that.
COMMISSIONER HANSFORD: Yes.

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A. So those bars, in a situation -- and I only saw a couple of situations where that would have occurred, where there wasn't sufficient concrete between the two to get the full transfer of load. But there are nine layers.
COMMISSIONER HANSFORD: Yes.
A. And the utilisation levels in the mid-span are something in the teens. So remediation in this case is very satisfactory, and it's almost back to where the designer would have expected it to be, and I would say utilisation levels are probably no higher than 20 per cent.
COMMISSIONER HANSFORD: Thank you. That's very helpful.
A. I wouldn't be making these statements in, shall I say, a more highly stressed structure, but --
COMMISSIONER HANSFORD: This is in relation to this particular structure.
A. Exactly, and all my comments do relate to this particular structure.
COMMISSIONER HANSFORD: Thank you. That's very helpful.
MR CHOW: Dr Glover, another issue that Mr Chairman has discussed with you earlier is in relation to the variation of the strength of the concrete over time. In fact, Dr Lau also mentioned about development of micro-cracks and this is the reason why he opined that over time, after several years, concrete strength starts

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to decline. What is your view on his suggestion of development of micro-cracks that caused the decline of the strength?
A. Micro-cracks? I've got to think now what might have been in his thinking there. We do get micro-cracks during the curing period, due to shrinkage, and if you have restraint then the concrete gets poured and it manifests itself in cracks. But there's no evidence of that. You would see it on the surface rather than in the heart of the structure.
I'm not aware -- I mean, concrete is a ceramic. I mean, do your plates have micro -- your plate is a ceramic, concrete is a ceramic. Are you aware of any micro-cracks in your plates? Do they suddenly disintegrate? I'm just trying to find out what he meant, because for something to crack, there has to be an external influence on it or there has to be something to do with the chemistry which is causing it, and I'm not aware of that, and if I refer back to my analogy with Roman concrete which makes up a very significant constituent of modern concrete mixes, then I'm afraid I can't help you answering that question.
Q. I believe Dr Lau also mentioned because of the loading and the stress experienced by the concrete, it develops micro-cracks, so that may be the distinction between the

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plate --
A. Oh, right. That's fair enough. I thought he was thinking the poor old concrete sitting there, it's minding its own business, and then suddenly over time it starts to say, "I have a micro-crack."
No, micro-cracks in the sense of surface cracks are very much part of reinforced concrete, because as I think I said yesterday, reinforced concrete and people -- this is a bit of a shock horror, but reinforced concrete doesn't work unless it does crack, because it's the flexing of the concrete which then allows the concrete to bond onto the bar and for the bar to develop its strain. But these are micro-cracks. When I say "micro", you will be able to see some of them, but they are spread out along a distance, and that's what we are talking about. Do you think that's what he might have meant?
COMMISSIONER HANSFORD: The suggestion --
A. Those would be called micro-cracks and there would be surface you would see them.
COMMISSIONER HANSFORD: I think the suggestion we had from Dr Lau was over many years these micro-cracks would result in the concrete being weaker.
A. No. I can't buy that one. I can appreciate if you get cracks you might get corrosive materials in it and

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something nasty will happen. He did refer to carbonation in 1950s -- or 50 year ago concrete and he's absolutely right about that, and that's because the concrete wasn't dense enough. I mean, I'm not talking about heavy now, I'm talking about dense in terms of -- if you think of concrete, it's a mixture of stones and paste, and the density comes from can I get enough paste and bits of sand into those parts? And that's where admixtures have come in to a large extent and that's where the pozzolanic material comes in.

Our concrete mixes now -- take this in the sense that it's given -- the emphasis is on durability and density rather than on strength, because strength, if you like, is not something that we're worried about so much anymore because we know we can achieve it, as the concrete cubes demonstrate. And indeed, if you like, our specifications on strength of concrete are probably -- are behind in terms of where we are with concrete technology.

So the emphasis for us nowadays is to make sure we have a good, dense mix, to provide that durable cover, because we do not want carbonation, and the carbonation occurs because of this micro-cracking -- maybe this is where he was. You get this micro-cracking occurring because the reinforcement is straining. So you get

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a little crack and that allows the passage of carbon dioxide to reduce the effectiveness of the cover, and that's called carbonation. But actually, if you can make that material which is in the cover dense enough, then you reduce that. And there's a whole series of modern tests that we use to do that on concrete.

MR CHOW: Just to wrap up this part of our discussion, so your view is that as long as the structure is properly built and so long as the structure will not experience a loading which exceeds what the designer has allowed for, then the structure can exist forever?

A. Yes.

Q. A concrete structure.

A. Yes. Well, as long as you don't have any corrosive materials coming into it or whatever. I mean, there has to be a sensible limit to it. But concrete itself, no, I don't see --

Q. All right. Okay.

A. There are no little termites out there, for example, concrete termites.

Q. Right.

A. You're not going to bring that up, are you, concrete termites?

Q. No, I'm not going to bring that up.

A. That's a relief. I'm afraid my knowledge doesn't extend

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to insects.

Q. Before I go back to what I planned to do with you this morning --

A. Oh, dear.

Q. -- can I just seek clarification on what you have just said on those partial factors of safety, just to clarify what really your position is.

Now, so far as you are concerned, for the purpose of giving your opinion, in answer to the Commission's question as to whether the structure is safe or fit for purpose, what you have in mind is the corresponding partial load factors for forensic analysis; is that right?

A. Yes, that's right. I was just trying to explain how I would have approached any other structure, but what I'm saying is I didn't take advantage of any of this.

Q. Certainly, yes.

A. But I wish I could, I mean -- but I didn't have to, because it's safe and it's fit for purpose.

Q. Okay. Thank you.

A. But if you want me to, I will, and I'll demonstrate it's got an even larger reserve of strength.

Q. It's very important that you clarify that, because as Mr Southward told us yesterday there's no textbook definition for safe, safety and fit for purpose. So

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different engineers may have different reference line, different benchmark, and the way I understand Dr Lau's evidence, as far as he is concerned, he would only consider the structure is safe if the structure provides the same level of safety, ie the same factor of safety as required by the code; whereas as far as you are concerned, you consider that the factor of safety which is somewhat lower than the factor of safety required by the code but for the purpose of forensic analysis, so long as it passes your forensic analysis and the corresponding somewhat lower factor of safety, you would consider the structure safe?

A. I'm afraid you've used a word which is an alert to me: a lower factor of safety. No, I don't believe it follows from this I end up with a lower factor of safety, because actually all I've done is I've looked at the actual situation, and the actual situation is the resistance side has gone up.

Q. All right.

A. And so therefore, if I was to look at the factor of safety of the structure as I understand it today, it would be higher than the design, for this particular structure.

So I don't want people running away with that particular idea, because I could see that is a line that

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some people might misunderstand. You and I don't but other people might, so --
Q. Right. One more clarification before I move on. The way you explained to us about forensic analysis, that is to do with the ultimate limit state; yes?
A. Yes.
Q. So even if you consider the forensic or you adopt the way that you did for forensic analysis, you still need to check the serviceability?
A. Yes.
Q. And, when you check the serviceability, you still need to apply the corresponding load factor and material factor as provided in the code; is that right?
A. Yes, I did say that, actually. It doesn't affect the SLS condition at all, in which I don't have material factors, by the way, or load factors. You just take life as it really is, the loads as they are.
So, no, this is to do with ULS, as you and I -- again, I guess this is for other people, isn't it, not for you and I -- is it?
Q. Yes.
A. I hope it is.
Q. (Overspeaking) ... for the Commission.
A. Yes, good.
Q. If I may then start with what I planned to do originally

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this morning.
A. Sorry.
Q. The next topics that I would like to deal with, actually we have covered a little already, is about fit for purpose; right?
In paragraph 5.8 of your first expert report, you said, in your opinion, the important aspect to be satisfied -- I'm starting from line 4 of paragraph 5.8.
A. Yes.
Q. "... the important aspect to be satisfied is whether the as-constructed structure is capable of being used and function as a station safely and without any physical restrictions on its operations and as anticipated by MTRCL. In that regard, the structure should be durable, safe, have sufficient strength, and not deflect or vibrate beyond those limits expected for a station ..."
Now, I just want to seek your clarification. By "durable", in the context of post-construction assessment, are you saying that one has to ensure that we have enough concrete cover to ensure that it is durable, and to have enough concrete cover you would make reference to the requirement of the code; is that right?
A. I have to think my way through this one, because I think -- yes, I would look at the relevant standards

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to achieve the level of cover I would expect for the exposure that the structure would undergo. I hope that made sense, because it didn't come out as a flowing sentence.
Q. Yes. And also, to ensure durability, you would have to ensure that the cracks that are going to develop in the structure would not be excessive, and to decide whether it's excessive, again you make reference to what is set out in the code. In this present case, it is 0.3 millimetres.
A. Yes, you've got to be careful about that, because I think the building code even in Hong Kong recognises that in some situations, exposures, for example, that the crack width calculation, provided you have obeyed certain detailing rules, is a deemed-to-satisfy criteria. People do see the crack width calculation as a science in itself but indeed it's black magic, and that's really why it's only ever been taken as an indicator, it's not absolute.
So therefore, I think this is true of all codes that I'm aware of anyway, national codes, the crack width calculation, if that's what you're referring to, does not have to be executed in all situations; it's only in situations where there is a particular issue to address.
Q. Right.

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A. And I think we are in a situation here -- and we are talking now specifically of couplers, I think -- where that's not the case. It gets a tick in the box and I thought Mr Southward's description of different things, when this question was asked, was entirely correct.
So I'm happy with that.
Q. Right, okay.
A. Sorry, when I said "I'm happy with that", I hope you didn't mean to say that I'm happy with it has to be code-compliant.
Q. "Ah, that is what I understand."
A. I thought you might. What I'm saying is you've got to look at the environment and I believe what the code in this particular situation says makes sense, that this environment is benign and that the "deemed-to-satisfy rules" meant you didn't have to carry out that check anyway, and that's it, and if that's what the code says then that's okay.
Q. The code, from my recollection, because I remember my learned leader has taken Mr Southward through details of the relevant part of the code, about different exposure conditions.
A. Yes.
Q. The code requires that the maximum allowable crack width

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is 0.3 millimetres. If the structure is exposed to exposure condition 1, then the code said one doesn't have to worry about durability, in excess of 0.3 millimetres --

A. Correct.

Q. -- may not have any impact on long-term durability. But for other exposure conditions, then you have to ensure that the crack width under working condition should not exceed 0.3 millimetres. This is what the code said.

A. Yes.

Q. Are you happy with that as a reasonable requirement?

A. I'm happy that we have a benign environment, whatever you want to call it, and as such cracking does not pose a risk. It's not a wetting and drying situation, and I think that is it, really, other than to observe that I have not seen any of this cracking, and the loading that the structure is currently experiencing is about 90 per cent of the loading that could reasonably be expected, and if there had been any cracking it would have been apparent by now, even in an isolated area, even in one or two locations. But there's no evidence of this.

Q. No.

A. So I'm not sure where your questions go because if we don't watch out, we are not going to be talking about

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this structure, we are going to be talking about clauses in codes, and I've already said that's not -- I'm not considering that. I don't want to give you a judgment on compliance. I'm just answering the question: is it safe, is it fit for purpose, does it pass those tests? And I'm saying yes. I don't want to discuss what the fine detail of a particular clause says in a code, because I don't think it's relevant, not because I don't think it's important in another arena. In this one, it's irrelevant.

Q. Actually, at this point, I just want to clarify with you what you actually mean in paragraph 5.8. But as you mentioned about, we are not in a wet --

A. Wet and dry.

Q. Wet and dry. Perhaps it is a convenient moment that I have to raise this with you.

A. Okay.

Q. According to my instruction, there is a tidal variation.

A. Yes, there is.

Q. The tidal variation, according to Atkins' report, varies from minus 0.2mPD to plus 2.8mPD.

A. Those -- I guess -- I can only imagine they come from historic records.

Q. Yes.

A. Because the sort of diurnal, in other words what you

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would expect every day, is that the slab level is at plus 4, the actual sort of stable water table I think is around about 1, and it varies by about half a metre either side of that.

Q. Right.

A. If you have an extremely high tide, a storm surge or something, then I should imagine it could go as high as that, but I find the 2.8 -- did you mention? -- to be a bit extraordinary, and the lower figure must have been because of some local -- in fact, it would have been because of some local drawdown that the contractor would have been carrying out as part of the construction.

But by and large you don't -- you don't design around those sorts of numbers because they are almost instantaneous. You know, you don't take those as long-term durability issues, I should say to you. You've got to take those loadings into account in your structural calculations, but when -- so, when we are looking at durability issues, we tend to look at the steady state and we take some fluctuations around it but we don't take extremes because, if you take extremes, you end up with extreme solutions which make no sense at all.

So that's why I was trying to put the numbers that we were being given in perspective. They are not the

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sort of numbers that you would expect the structure to be subjected to in a long or medium-term condition. Something extraordinary would have to happen in Hong Kong for that to be the case; because, for example, for the water table to rise to 2.8 metres, when the sea level is about zero, just think what that means locally, whereas the level at the moment is 1 metre.

You know, so I have to push back on that, I'm sorry. I mean, you are giving numbers in good faith. I'm not disputing your earnestness. But I have to equally put them into perspective, don't I?

Q. May I just, for the purposes of the record, give the page reference for this. It's bundle AA, page 527. It's part of the Atkins report which provides the tidal variations from minus 0.2mPD to plus 2.8mPD.

Regarding what you said about the top level of the EWL slab, my instruction is that it is not plus 4, it's only plus 2.85mPD. Would you disagree with that?

A. I'm sorry, if that is the case -- let me take your number for now. I've been advised it's 4 but if you've got better advice then I'm not going to challenge that.

Q. Thank you for your trust, but I can also give a drawing reference in which the level, the top level of the EWL slab is marked on those drawings. It's bundle H --

A. Sorry, is that the level in the station or is that the

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level in --
Q. The EWL slab.
A. Yes, but is that the level in the station or is that the level at SAT, NAT, area A or whatever? Because, you see, in the station, it's at a level, and then when you've got --
Q. Right.
A. Of course, and I just wonder. But anyway, let's listen to what you have to say.
Q. I assume it is for the station --
A. I'm not sure.
Q. I can further confirm that -- we have looked at it during the first stage of our COI, I think 2.85 is a figure which looks very familiar to me, so I suspect 2.85 is the top of the EWL slab of the station.
A. Okay.
Q. Just for the purpose of the record, bundle H, page 552 is a drawing on which someone marked the finish level of the EWL slab as plus 2.85.
Assuming that the top level of the EWL slab is plus 2.85 --
A. Oh, we've got a drawing.
COMMISSIONER HANSFORD: Can we just pause and see this drawing?
MR CHOW: Sure.

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A. Can you point me to the level itself? There's a ground level outside, isn't there?
Q. Yes.
COMMISSIONER HANSFORD: There's a point --
A. I can't really see it.
MR CHOW: Can we blow up a little bit, the top of the EWL slab.
A. There's a level there, isn't there, on the ground? Can I see what that is. Yes --
Q. 2.84.
A. 4.4 is the existing ground, and then we've got a dimension of 5 -- no, that's the slab depth. Rail level -- yes, look, there's your 2.84.
Q. Yes, 2.84.
A. But this is -- I'm not sure where this section is but let's assume that that's -- because you see, what I'm referring to, you see that level there (indicating), 4.03?
Q. Yes.
A. That level there (indicating), that was my reference point about the station being at 4.
Q. I see. But to the left of it we see a level marked off, plus 2.84 --
A. As I said, I'm quite happy to --
Q. It looks like --

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A. I can see where the confusion has arisen, yes.
Q. So let's assume the top level of the EWL slab is plus 2.84.
A. Yes.
Q. And given the tidal variation can go up to plus 2.8, with that tidal variation am I right in understanding that the top part of the EWL slab is subject to wet and dry conditions?
A. The external face of the diaphragm wall?
Q. Yes.
A. Yes. The external face of the diaphragm wall could be, yes. But, I mean, you've got to remember, wetting and drying -- what wetting and drying means is something is wet, you know, you take a bucket of water and you throw it on it, and then you allow it to dry in oxygen, so it's got lots of oxygen coming into it, and then you dry it, and then in a short period after that you throw another bucket of water over it and you get more oxygen in it.
In the ground, it's not like that. You see, the fact -- an interesting thing that people don't realise about waves, for example, the water doesn't move. All a wave is is a circular motion of a particle of water moving round and round. So because the tide actually moves up and down, we've got this thinking that there's

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a huge in-flush of water, but it's not, it's a pressure, and the water locally just rises up and goes down. The level of oxygen in that water is not substantially changed.
This is not the same as my bucket of water, drying it out with a hairdryer and then putting another -- it's not like that. So I'm not sure where you are going.
Q. The short point I'm suggesting --
A. We are talking about the outside wall of the diaphragm wall. I thought we were talking about cracks local to the couplers which are inside the structure, away from the wetting and drying. So can you get the connection between the two?
Q. As I understand --
A. Otherwise we are going to waste our time, aren't we, talking about the diaphragm wall?
COMMISSIONER HANSFORD: Sorry, just pause there. You said "away from the wetting and the drying". It's just wetting.
A. It's just wetting, and the ground is very humid. I really don't know. This is not --
COMMISSIONER HANSFORD: Because it's not drying, is it?
A. No.
COMMISSIONER HANSFORD: Is that right?
A. It's wet. You don't need standing water for something

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to be wet.
CHAIRMAN: Mr Chow, perhaps you might put the question that you would like to put now and then we can see where that takes us.
A. That would help.
MR CHOW: What I'm getting at is that from my understanding of Dr Lau's evidence, you will recall that yesterday or the day before, when he talked about the crack distribution within the joint, he mentioned about the cracks on the outside of the diaphragm wall, and to him there's a concern of water getting in and causing corrosion to the reinforcement inside.
Yesterday, in your presentation, you mentioned one would need oxygen, water and iron to cause corrosion.
A. Yes, and a constant flow, by the way, of oxygen and water, not the same water.
Q. Not the same water.
A. Yes. The key is, for something to cause corrosion, there needs to be a constant supply of the things that make the corrosion. If you deny one of them, you don't get it. So if you deny oxygen, you don't get corrosion. Put a nail into water at home, for example, and leave it there, and carry out with a number of different ones, you do it over a number of days and take the nail out and see what the degree of corrosion is, and you will

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find that it will plateau.
Q. Just to use your example, if I drop the nail in the water for a certain number of hours in the day, and then I bring it out into the air, and the next day I do exactly the same thing over a period of a year, are you suggesting the nail would not rust?
A. No, that's not correct.
Q. So the nail will rust because it's subject dry and wet conditions?
A. Yes, when you take the nail out, it will be wet, and it absorbs oxygen, it will start again.
Q. Very well. So for the part of the EWL slab structure, which is subject to tidal variation, so during a certain period of hours in a day, it is in direct contact with seawater or saline water; is that correct?
A. It never dries.
Q. It never dries.
A. The environment in that level, there's always a level of dampness just because of the poor water pressures in the soil.
Mr Chow, please, ask me the question and I'll answer it. Do I believe there is a crack that could sensibly connect into the CJ; is that what you are asking me? Because I think that's what you are.
Q. I am not asking you about the CJ yet.

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A. Just ask me what it is, otherwise we are going around in circles.
Q. All right. According to Dr Lau's theory, there are cracks on the outside of the diaphragm wall, and because of the tidal variation there is a concern for excessive crack width, because that will cause corrosion to the steel inside the diaphragm wall, and at that zone, the tidal variation zone, that is the region where we have the connection. Do you agree with me that that is a concern?
A. No. And I repeat what I've just said, just to make sure that everybody understands why that is: the diaphragm wall has been designed in accordance with all of the standards required. It's been constructed in accordance with all the standards. It has all of the quality assurance tests. It has been passed by the approval authorities. It's undergone the highest level of inspection. If you start to question that, Mr Chow, then you should question every single diaphragm wall in Hong Kong. Is that what you're doing? Because if you want to go into that, then I'm quite happy to do that, but that's what you are saying.
I submitted yesterday some non-linear finite element analyses which demonstrate it is implausible for the scenarios that are being described to occur, and apart

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from that I fall back again on what I've just said about the quality of construction of the diaphragm wall.
Now, if you want the Commission to open up the discussion onto the diaphragm wall, that's for the Commission to decide, not for me, but I've just told you what my position is. And I'm sorry to be quite so strident, but it's a non-issue and we have some important issues to discuss and I'd rather move to those.
So I didn't mean to be offensive but I just wanted to be clear.
Q. Not at all. I don't mind at all.
CHAIRMAN: Could I just ask you here, so we know where we are going, because I appreciate you are saying, according to Dr Lau, his investigations have indicated the likelihood of cracks on the outside of the diaphragm walls. That's what you say.
MR CHOW: Yes.
CHAIRMAN: Those cracks, being on the outside, would be in direct contact with the earth, and because of tidal variations, so the water levels in the soil become greater and lesser, there is a concern for excessive cracking which may lead to corrosion to the steel inside the diaphragm wall.
Now, has government taken any steps by way of

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remedial action to deal with that particular problem?
MR BOULDING: Sir, can I just intervene, and I apologise for this, but it is important to point out that Dr Lau has not carried out investigations to come up with that particular conclusion. It's all theory.
CHAIRMAN: All right.
MR SHIEH: One can be concerned about anything under the sun.
CHAIRMAN: Absolutely, which is why what I want to try to find out is starting from square one, so to speak. Nothing has been done, no remedial actions have been taken, based on Dr Lau's opinion?
MR CHOW: Mr Chairman, from my recollection of Dr Lau's evidence, he seems to suggest that the installation of dowel bar would somehow reduce the stress level inside the connection and that would help. And I would understand his evidence as suggesting that because of the addition of the dowel bar, it will reduce the crack width, and therefore improve the situation and that would help or mitigate or reduce the concern with corrosion.
CHAIRMAN: All right. One of my major concerns obviously is that we don't bypass these things unwittingly. So what's being said then is that Dr Lau is concerned, based on his knowledge and his expertise, not on actual

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investigation, that there are cracks on the outside of the diaphragm wall, and because of tidal variations, they may become excessive.
COMMISSIONER HANSFORD: No, because of tidal variations, water may seep into them.
CHAIRMAN: Yes, thank you, causing corrosion. But he is of the view that this perceived long-term problem of corrosion will be satisfactorily dealt with by what I'll call the dowel bar remedy.
MR CHOW: That's what he said, yes.
COMMISSIONER HANSFORD: I think probably then it might be useful for the Commission to get Dr Glover's view on that --
MR CHOW: Yes.
COMMISSIONER HANSFORD: -- sort of scenario.
MR CHOW: Yes. Perhaps I can ask Dr Glover --
COMMISSIONER HANSFORD: Yes.
MR CHOW: Dr Glover, you recall that part of Dr Lau's evidence is that there is a need to install dowel bars, and the reason that he gave in evidence is that because of the cracks on the outside of the wall and the installation of dowel bar will improve the situation, so as to somehow eliminate the concern. What is your view on that?
A. Mr Chow, you know I don't want to say this; right? But

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I think it's patently obvious that adding a dowel which contributes something like 1 per cent, less than 1 per cent, to the strength of something is not a mitigation. And indeed the installation of such a dowel, with all the good intentions that it has, causes vibration and all the other bits and pieces.
So I don't see the link, as a professional engineer. You might wish to ask the same question of Mr Southward or Prof McQuillan, but I don't see the link, I'm sorry. It might be a lack of understanding on my part, but for the life of me I just can't understand.
MR CHOW: All right.
CHAIRMAN: Could I follow that up. What you are saying is that the dowel remedy, if I can call it that, you don't see it as being of any significant assistance to whatever problem may be --
A. To whatever the list of remedies that are being sought, I don't see that it contributes. This particular one, this cracking of the diaphragm wall, it has no relationship whatsoever.
CHAIRMAN: Then that's my second question: are you yourself, on what you know of the structure overall, particularly the external side of the D-walls, concerned about the issue of possible corrosion over an extended period of time to the steel inside the D-walls?

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A. No. The diaphragm wall has been designed competently, constructed very competently, lots of photographic records of what was constructed. We now have the benefit of the cores of the concrete which demonstrates it's very dense. Remember my comment earlier about density of concrete being the most important thing in terms of corrosion protection. No, I don't have concern.
MR BOULDING: Sir, I don't want to be pedantic but it is an important point. My learned friend Mr Chow put the question on the basis -- and perhaps I can read:
"Dr Glover, you recall that part of Dr Lau's evidence is that there is a need to install dowel bars, and the reason that he gave in evidence is that because of the cracks on the outside of the wall ..."
Now, it's important to point out that his evidence was not that there were in fact cracks but there might be cracks. I see the professor is nodding.
COMMISSIONER HANSFORD: Yes.
MR BOULDING: It's an important point, in my submission.
COMMISSIONER HANSFORD: Yes. I think his evidence -- please correct me if I've got it wrong -- is that there is a risk that cracks might occur.
MR BOULDING: That's my recollection as well, including that of my learned junior.

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CHAIRMAN: Thank you. Yes.
MR CHOW: It also refers to a number of diagrams from finite element analysis which show the pattern of the cracks. I think this is as far as we can go.
A. If I could just put that diagram into perspective, and I'm sure Dr Lau wasn't intending to mislead anybody, but if you looked on that diagram -- it was from Atkins, I think, wasn't it? It's quite clear -- at the bottom, it says "ULS conditions", in other words failure conditions. That was misleading, to show that slide.
The other thing is Atkins do acknowledge that that analysis was done in a bit of a hurry and they hadn't completed it, whereas the analysis I showed you yesterday has had the opportunity of more consideration.
So I would say it's a pity that that is in the bundle but if anybody ever was to refer to it, I think they've got to look at it and see that it's related to the failure mechanism of the structure and not at all what the working life cracking pattern would be. So it was very misleading to show that slide, without putting that -- I wouldn't say caveat but that explanation, and that explanation, as far as I'm aware, was not made.
Q. Yes. I actually take your point. I will take your word for it. If it is referring to ultimate limit state, then perhaps it has to be made clear.

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COMMISSIONER HANSFORD: Can I suggest we don't take the word, we have a look at it.
MR CHOW: Yes, please.
COMMISSIONER HANSFORD: Can we go to Dr Lau's presentation.
A. I've got my fingers crossed that my memory is good.
Was it in his report?
COMMISSIONER HANSFORD: I thought it was in the presentation but I will stand corrected.
MR CHOW: I think it is in his presentation as well.
COMMISSIONER HANSFORD: Yes, I thought it was.
A. There it is. It's that one.
MR CHOW: Maybe page 44.
A. Yes. If you look at the bottom, the left-hand branch of it --
Q. Yes, that's right. Actually --
A. -- it does say "ultimate limit state".
Q. Yes, figure 5.4 does state that it's the crack pattern at ultimate limit state.
MR PENNICOTT: Yes.
CHAIRMAN: Okay. Can you help me?
A. Ultimate limit state, sir, is the ultimate design, in other words it's about to collapse. Crack patterns are irrelevant for ultimate state of collapse. We don't even ...
COMMISSIONER HANSFORD: Can I just --

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A. So it shouldn't -- to use that as a basis for saying, "Oh, my goodness me, it's all going to crack", is misleading, to the extreme.
COMMISSIONER HANSFORD: The purpose of my intervention is to make sure the chairman understands what is being said.
A. Sorry, yes. First of all, does Mr Chow agree with what I said was correct?
COMMISSIONER HANSFORD: Sorry, can I just continue?
So what this is telling us is this is the cracking pattern at ultimate limit state, the theoretical cracking pattern at ultimate limit state; yes? The ultimate limit state is the condition at failure, and the analysis has shown that it never reaches anything like the loading required for failure, ultimate limit state.
A. Correct.
COMMISSIONER HANSFORD: In layman's terms, is that correct, Dr Glover?
A. It is, and we don't -- we are only interested in cracking patterns at the ultimate limit state, to understand the failure mechanisms. We check crack width at what we call SLS, which is the working load level, and the crack pattern which is appropriate to that was the one that I showed yesterday.
COMMISSIONER HANSFORD: Yes.

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A. And it was the one between -- I showed three images. One was at ultimate limit state and the other two were related to various levels of service, and you saw the levels of cracking were very limited and they were indeed concentrated at the bottom of the EWL slab and there was no sign of distress in the higher-up parts.
I'm sorry to give a long answer but I wanted to be precise.
COMMISSIONER HANSFORD: No, no. We need a long answer.
CHAIRMAN: All right. Thank you. That makes a lot of sense. Thank you very much.
MR CHOW: Dr Glover, can I just finish my question on paragraph 5.8 before the morning break.
A. Please.
Q. In paragraph 5.8, you also mention about "not deflect or vibrate beyond those limits expected for a station".
A. I'm sorry, Mr Chow.
Q. The last part of paragraph 5.8, you list out a number of factors that you consider. The last factor is "to ensure that the structure will not deflect or vibrate beyond those limits expected for a station".
A. Yes.
Q. Now, the limits expected, it's concerned about expectation, do you agree that it would relate to a particular expectation in the locality at which the

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structure is located?
A. Are you referring specifically -- okay, generally, yes, but when it comes to deflection, we have checked all the deflections in the structure, taking account of all sorts of factors, but deflection just isn't -- I mean, if you remember the conversation we had about monitoring, it's not an issue. Vibration, we've carried out dynamic analyses to find out if there is anything that could disturb.
So, as far I'm concerned, it satisfies those requirements.
Q. So again in relation to these two aspects, you would check against the requirement set out in the local code, in our present case the Concrete Code?
A. Yes, in terms of deflection it's a slam dunk, I think the Americans call it. Why, do you think there's a concern? Oh, good.
Q. I'm just seeking clarification from you.
A. I'm sorry. Right. Okay. I confirm that.
MR CHOW: Mr Chairman, this is a convenient moment for the morning break.
CHAIRMAN: Yes, certainly. Thank you very much. 15 minutes.
(11.25 am)
(A short adjournment)

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(11.46 am)
MR CHOW: Dr Glover, I would like to move on to another topic, the partially engaged coupler connections.
It looks like the experts are in agreement that there is initial slip if the coupler connections have not been fully tightened.
A. On the basis of the laboratory tests, yes. That's self-evident, yes.
Q. According to the test results, the permanent elongation -- now, I would put the term "permanent elongation" carefully because I understand that the expert view is that this so-called permanent elongation actually represents mainly the extent of the initial slip.
A. Well, it's bedding-in of the thread, basically. When you do up a bolt or whatever, one or two of the threads are actually in contact and the others aren't, and this is because of the machine tolerances. If there wasn't the tolerance, you wouldn't be able to do it up.
So what you are seeing is the initial plasticity of the points that are in contact closing up and then all of the threads come into action.
So that's what it is. It's not slip. It's a bedding-in of the thread, but if you want to call it "slip" then I'm happy with that.

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Q. So the proper description of that is bedding-in?
A. It's bedding-in, yes, of the threads.
Q. According to the test results, we observe that for the one that shows the worst situation is this so-called permanent elongation can go as far as over 0.5 millimetres.
A. I mean, the record is the record, but when we carry out such experiments as that, one always uses the mean. We never use the extremes, for reasons I have explained previously.
Q. All right.
A. You can use the extreme if you wish, but I prefer to stick with the convention, which is the average, and the average is I think something like 0.28, which is quite dramatic, dramatically different. One might be three hairs' breadth and the other one is two. But that's what we're talking about.
When we start bandying numbers around, it's very important that we bring it back to something tangible that people can understand. 0.1 is -- a hair is wider than 0.1. It does depend on ethnic group, mind you, but that's generally -- I won't say which is which, but mine is probably around about 0.1.
COMMISSIONER HANSFORD: Perhaps the other comparison is the number of sheets of paper.

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A. Yes, that's a good one.
You have to have good eyesight to measure.
COMMISSIONER HANSFORD: I know. I can't --
A. You can't. You can't physically measure 0.1mm with the naked eye. In fact, if you have a pen, the old Rotring pens, the ink pens, 0.1 -- you remember those? 0.1, it always clogged up. You could never use it sensibly because it was just too thin.
Sorry about that.
MR CHOW: I think the concern here in relation to this so-called permanent elongation is the effect on this, using your term, bedding-in on the crack width, when it is loose in the structure.
Yesterday you also mentioned about strain compatibility.
A. Yes.
Q. Let's see if I understand you correctly. What you are basically saying by reference to strain compatibility is that when all these coupler connections are cast in the concrete, for the sake of discussion, let's assume 30 per cent of those are not properly tightened at the time of pouring of the concrete, this initial slip or bedding-in effect would not manifest itself, because we still have another 70 per cent of the properly done couplers which are holding the concrete together. In

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layman terms, is that what you are talking about?
A. Yes. That's a good model.
Q. Just for the sake of discussion, if the working load starts to increase, at some point, notwithstanding the 70 per cent of the properly connected couplers, at some point one would start to mobilise the remaining 30 per cent --
A. Yes.
Q. -- of the not properly connected couplers, and at that stage, even with a very small force, this bedding-in, the effect of this bedding-in, will start to kick in; right?
A. Mm-hmm.
Q. As far as I understand, at the moment, no one has ever looked into the combined effect of this phenomenon; is that correct?
A. I'm not aware of any experimentation in that respect. It is, as I observed earlier, a question of looking at each situation. People have not raised this issue in the past, I think that's true to say. But no, there's no -- there has not been any comprehensive research carried out on the issue that you described. You can re-create it mathematically, as you and I can do, but ... yes.
Q. But the situation that I've just described, it is real,

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it's not far-fetched situation?
A. Yes, as I said, I think it's a good model. It does depend on how quickly that slack is taken up, you know, that bedding-in.
Q. Certainly, yes.
A. But the answer to your question is there is no research that I'm aware of into this phenomenon.
Q. I would like then to move on to a slightly different angle regarding these defective couplers. Now, you recall that in area A and area HKC, we have these different types of so-called configuration --
A. Yes.
Q. -- where we have a starter bar coming out from a capping beam, and the starter bars are to be connected with the other horizontal reinforcement by way of couplers; right?
A. Correct.
Q. Now, you mention, as an engineer, you will first approach it by looking at the physical process of forming these coupler connections.
A. Mm-hmm.
Q. I'm sure you are aware of the view of Dr Lau. Dr Lau is of the view that we should take those couplers that we have described, taken out from capping beam, as a separate family of data from the rest of the couplers

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in the EWL slab, and the reason given by him is because the working conditions between these two groups of couplers are different.
Now, from the evidence that we received perhaps during the first round of the Commission's Inquiry, we know that when BOSA produced the couplers -- the threaded rebars and the couplers, they are well protected. Do you recall there are two types of cap, the blue cap and the red cap?
A. Yes.
Q. The red is for I think the ductile couplers, with the corresponding threaded bar, and the blue is for the non-ductile cap.
So we see that the condition when it is produced from factory is rather good. The Commission probably has also seen a video recording of a visit to a factory, BOSA's factory, in which, during the visit, BOSA demonstrated how the threading process was done, and right after the thread was produced there is a device, cylindrical device, to control -- to screw it in and control -- I believe that device is to ensure that the thread length is of a certain length.
From the video, we observe that the process of screwing this device into the threaded bar is quite easy and quite smooth. In other words, as a layman

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understands it, if the threaded bars and the couplers are freshly from the factories, and if it is well protected, then one should not have any problem in screwing in; it should be quite easy.
So do you agree that that will be similar to the situation of the couplers in the capping beam, because the couplers taken out from the capping beam and the threaded bar have never been embedded in concrete, so the condition of the thread and the couplers should be very good; do you agree with that?
A. Most certainly.
Q. Whereas for the other group of couplers which had been cast in concrete, concrete in the diaphragm wall, the evidence that we received from the earlier round of Inquiry is that Leighton has to expose it by way of some equipment, high-pressure water jet, and after this process, the steel fixers told us that certain couplers were damaged. Some are misoriented. And we actually see photos showing a stack of damaged couplers being removed.
From that, would you agree that the working condition would be very different? The condition of the couplers, the condition of the threaded bars for the rest of the EWL slab would be very different from the condition of the threaded bars taken out from the

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capping beam?
A. Yes. I have made the point, I think a couple of times, that my expectation of area A, that the performance would be of the same type of family but it would be superior, and that's really why I said I could not believe the 68.5 per cent failure rate for something which, the way you've just described it and indeed the way Dr Lau described it in his report, should be better, and there is no -- the evidence says it is better, but please continue.
So I agree with you that area A is a better situation than you would have expected anywhere else. Is that what you want to --
Q. No.
A. It's not what you --
Q. We will come to that later on.
A. All right.
Q. At the moment, I just want to see whether you agree with me that because of the difference in condition of the couplers and the threaded bars, these two sets of data have to be treated as two separate families and should not be combined for the purpose of statistical assessment?
A. Let's continue. I've already said that geometrically it's different. The same coupler is being screwed

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together. I believe it's the screwing together of the operative which is the key factor, and I think he should have done better in that environment than he would have done anywhere else.
So I think I'm saying to you the operation is the same, basically, it's a screwing action of an operative, and therefore I would expect to find greater similarity in the performance, but I would have expected it to be shifted slightly up in terms of a better performance.
Q. Right.
A. Is that what you want me to say?
Q. Because it's easier to be screwed in?
A. Yes, yes, I keep saying it; it's a better situation, yes.
Q. So you wouldn't expect the defective rate to be the same, as you have just mentioned, the couplers --
A. Sorry, I didn't say that. I said I would have expected them to be of a similar distribution but better. That's what I said. So I would have expected area A to be superior in performance generally.
Q. So you would expect the defective rate would be lower as compared with the general defective rate in the rest of the EWL slab?
A. I would have an expectation that it would be marginally superior, yes.

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Q. All right.
Then I would like to discuss with you the way to assess -- now, given that these couplers coming out from the capping beam are two-sided couplers, I think the term that you use is double-sided couplers.
A. Yes.
Q. I believe you would agree with me that there are two weak points on each side, potential weak points; right?
A. Yes.
Q. So, for the assessment of probability of defective couplers, if one considers that so long as there is one weak point which failed, would you consider that coupler connection as defective?
A. Correct. It's pretty standard probability theory and it's a question of arithmetic, yes.
Q. So if we have two weak points, given that -- perhaps we can take the example that you used, in paragraph 7.29 to 7.32, where -- this is the calculation that you did by combining purpose (i) and purpose (ii) couplers --
A. Yes.
Q. -- in area A.
A. Mm-hmm.
Q. For the purpose of the record, of course, we are advised by an expert in statistics that one cannot mix purpose (i) and purpose (ii), but I do not want to go

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into a detailed discussion with you --
A. It would be fortunate for you that we don't, I think!
Q. I don't want to get into --
A. I understand.
Q. I don't think it would be helpful.
A. I'm sorry, I didn't mean to say that in a disparaging way, but I just think it wouldn't be to your advantage.
Q. What I would like to discuss with you is rather the methodology, the arithmetic that you did. Assuming it is proper, as what you have done, to combine purpose (i) and purpose (ii), what you did in this section, paragraphs 7.29 to 7.32, is that you find, for example, on the slab side, there is a failure rate of two out of 18 samples.
A. Yes. I mean, it's a statement of fact.
Q. Yes, which works out to be about 11 per cent; right?
A. Yes.
Q. So the pass rate for that side of the couplers is about 89 per cent?
A. No, I don't say that. That's what is called the nominal rate, in other words that's what you observe, but you then have to apply extreme probability theory to establish what the likelihood of that is and it will always be worse than that.
Q. I'm trying to work out for the benefit --

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A. I'm trying to help you as well.
Q. -- of the Chairman and Prof Hansford. So on the basis that there are two failures out of 18 samples on the slab side, you then work out your so-called nominal failure rate, which works out to be 11 per cent.
A. Yes.
Q. Then on the basis that on the capping beam side there are two failures out of 11 samples, the failure rate would be about 18.2 per cent?
A. Mm-hmm.
Q. So what you did, by applying the probability theory, in order to determine the overall failure rate, you multiplied these two percentages. Now --
A. No, I didn't. I don't multiply them, no.
Q. You don't directly multiply them. First of all --
A. No, sorry, I don't use those actually at all, those numbers. They are just indicators. I take the total data set and then you apply it to the probability distribution. I just gave those for example. In fact, they are all good news. They are all just demonstrating how superior area A is to everywhere else.
So I don't multiply them together. I take that as a data set and then I put that into the binomial theorem, and that's what gives me the probability of any one bar being in exceedance of 37 millimetres or 32 or

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wherever I want.
Q. Yes, 37 millimetres.
A. Yes, 37 millimetres is the one, because -- sorry, you ask the question.
Q. In paragraph 7.32, you arrive at a pass rate of 72 per cent; right?
A. Correct, yes.
Q. When I run the numbers, it does appear to me that what you did is basically you combined the passing rate of the two sides and you arrive at 72 per cent as a combined pass rate.
A. No, no, no. 72 per cent would apply to the failure rate of one side, one bar. That's what the 72 per cent is, I believe. I'm trying to remember what this is here. So, once you work out the probability on one side then, as you correctly say, the probability would be the multiplication of the probability on this side, 70 per cent, times 70 per cent on the other side, which gives you 50 per cent, 0.5. So the probability of a failure of that connection is, for a 37 millimetre engagement, is 49 per cent.
Q. I see.
A. And so --
Q. This is the point I am trying to elicit from you. Basically, because there are two weak points on each

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side --
A. Yes, I agree. There's no argument.
Q. -- to determine the overall probability, you need to multiply or combine the corresponding --
A. Yes. It's P1 times P2 is the joint probability and that's standard probability theory. In fact, it's arithmetic --
Q. It's good enough for me for the present purpose. Prof Hansford, you wanted to ...?
COMMISSIONER HANSFORD: I'm happy.
MR CHOW: So if I may now go to paragraph 7.38, please. In paragraph 7.37 and 7.38, what you do here is you take the pass rate for 32mm engagement length, and then -- under paragraph 7.38, you take that for a single-sided connection, a pass rate of 88 per cent, which means 12 per cent failure rate, and then the second bullet point you said:
"For a two-sided connection in areas A and HKC, a pass rate of 77 per cent ..."
So, basically, the 77 per cent is the multiplication of 88 per cent and 88 per cent?
A. Yes.
Q. That is what you --
A. I hope that works -- yes, that is the intention.
Q. Do you accept that by doing this kind of simple

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probability calculation, the result that you obtained would not be of a confidence level of 95 per cent as an expert in statistics would expect?
A. No. Sorry, I would challenge that, because the data set is such that the data set is the data set. No, I'm sorry, that was a leap of logic which, I'm sorry, I didn't quite follow. But no matter. I think my statement is I don't agree.
Q. Okay. I will now move on a new area, the shear links.
A. Okay.
Q. When Mr Southward was questioned, you were in this courtroom so you have heard details --
A. Correct.
Q. -- in relation to the investigation, and my learned leader has also shown to Mr Southward the method statement for the investigation. Do you recall that part of the exchange?
A. I do.
Q. We know from the method statement that the process for the investigation is that, first of all, one would open up a square shape of 300 by 300 millimetres, and then, according to the method statement, if one finds a shear link, then depending on the location of the shear link, one would then further open up two perpendicular strips of area with a width of 200 millimetres. Do you recall

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that part of the evidence?
A. (Nodded head).
Q. And also the method statement, at the end, says that if no shear link is found, then one can consider further dig into an upper layer; do you recall that?
A. I do.
Q. I note that in your report, basically, your view is that this L-shaped opening-up approach is appropriate in the circumstances?
A. Yes, I would agree.
Q. Am I right to take it that when you say "appropriate", you would no doubt agree that with this sort of opening-up, if shear link has been installed as per the design, it would have been picked up under such an opening-up scheme?
A. I would agree, the L shape is an appropriate response. You don't have to open up a square. I think it does depend on the width of the L, and I did comment, I think, not adversely, but make the observation that I wasn't sure whether the width of the Ls was wide enough. But the principle I think is sound, of an L shape investigation.
Q. An objective fact that we see today is -- you know there are altogether 40 locations; right?
A. (Nodded head).

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Q. 22 of the 40 locations were actually honeycomb areas, and someone took the opportunity to inspect the condition and configuration of the shear links as well. These 22 locations, in terms of size, they are not limited to the L-shaped opening. Some of them -- I can take you to one or two of them -- are of a size of over 1 metre. I think one of them is over 2 metres. And because, at that time, we see that there may be a problem of shear link, and then MTR has decided to carry out further opening-up, so further 80 numbers of locations were opened up, and it was only for that 80 numbers that we are doing 300 by 300 and then 200 strip.
The result, overall picture that we have from these observations of the 40 locations is that out of 40 locations, 16 of them showed no shear link at all.
Would it cause any concern to you as to whether shear link was actually installed at those locations, given the number, given the overall picture?
A. I'll take that in stages. My first reaction would be that's not what I expected. I would then have to ask myself some questions about it. I would have to ask myself the question as to what is the reinforcement that I'm looking at, and without -- with all due respect, obviously these 40 locations, that's quite numerous, but

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the honeycomb areas were areas tending towards the middle of the span, you know, where the shear is nominal, and that's why we got the honeycombing because we got all this reinforcement there which didn't allow the concrete to go through. And in those situations the shear links would be really pretty nominal, even though the drawings might have shown them.
If my memory serves me well, in the mid-span, we were generally talking about four layers of reinforcement in each direction, in other words a stack of eight or nine bars, and it is perfectly acceptable, from an engineering viewpoint, that in those situations where you've got multiple layers of reinforcement, that the shear link can be anchored to one of the upper levels.
I can only tell you that as evidence in terms of that would not be wrong to do that, because the standard designs that are drawn is it's always a beam with just a single row of reinforcement in the bottom, with a link going around it. But when you've got many layers of reinforcement, that diagram engineeringly is still correct if the link goes to one of the upper levels.
So, as you said, I would take it in stages in that conversation, so in the honeycomb areas, I would think to myself that the steel fixers, because I know from the

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photographs that I've seen that the link was attached at the top, and it was a fishing trip, and that's really why we don't get the alignment as well, and that's also the reason why it's not tied with the wire, because it was fixed after. It was -- the bar, 3 metres long, I can only imagine, was actually put down (demonstrating) and he fiddled between the steel, and that's why the tab isn't 100 long because you couldn't get them in. They made them 70 long so they could just fish it in and pull it around (demonstrating).
So he would have anchored into whatever one he could get to. That's my supposition. But I agree with you that if you don't see something and you expected something, then it's right that an alarm bell should go on. I don't dispute that.
Q. I believe Dr Lau actually agrees with you on this point, that the shear link doesn't have to be hooked to the lowest layer of the reinforcing bar. It can be hooked in some inner layer, so long as it passes the --
A. That's right. Absolutely. It's a strut-and-tie action, yes.
Q. Given that in the method statement there is a specific requirement that if no shear link is found, then the contractor should expose or continue to dig into the inner layer. Are you aware of any reason why, during

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the investigation, notwithstanding the failure to observe any shear link, but MTRC's contractor failed to continue to expose further layer of the reinforcement, just to ensure that that is shear link?

A. It's very difficult. The instruction is very simple: dig deeper. You've got so many layers of reinforcement, and the clear distance between each of the bars is probably no more than 80 or 90 millimetres, and it's a square, and you're being asked to go through layer after layer, and it becomes -- well, it is impractical, because what you are looking for is something that is spaced at 300 centres, 1 foot, in each direction. Well, how do you know that you're mining in the right location?

I'm not saying there's any right or wrong in it, but the simple instruction of "dig deeper" reaches a practical impossibility, so you reach an impasse, I accept that, you reach an impasse where there is no evidence on the one hand, it's impractical to keep boring in to try and find it, so you reach a situation where there's uncertainty, and I accept that uncertainty.

But that's where we rely on photographic evidence, and whatever evidence we can pull to hand. Does that help you? I'm not sure if it does.

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Q. Yes, it does, except that I have some concern. Please help me.

Now, if we can't find in shear link upon opening-up, and then we assume that there must be shear link which is hooked at the upper layer, then a layman will think what's the point of doing opening-up investigation? If we don't find it, then we automatically assume that they are there but hooked at the inter-layer, then what's the point of doing the opening-up?

A. I agree. You are just demolishing more and more structure, because --

Q. Right.

A. I would then immediately fall back on whatever records can I bring to bear?

Q. Okay.

A. Because I would say there's a limit to what you can do in terms of opening-up without actually destroying the structure.

Q. All right. Can I then refer you to a few photographs of the honeycomb, because I notice that some of the honeycomb goes quite deeply into the slab. Some of them go as much as almost 300 millimetres inside the slab.

Can the photo in bundle ER2, tab 17.10 -- tab 17.10 has actually three files. The first file, page 27. This is one of the honeycomb areas that is designated as

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DS7.

A. I'm sorry, I haven't got the right one. DS7?

Q. It's on the screen.

A. I see, it's not the photograph, it's a diagram.

Q. It's a report. From this report, if we go down from the top, we see -- a little bit up, please -- there's a box with a tick which says honeycomb was observed and the approximate size is 2.3 metres by 1.8 metres, and the depth is 285 millimetres, it's almost 1 foot deep into the slab.

Then if we scroll down -- further down, please -- down to the shear links, you see there's a box; yes? The box which records the condition of the shear links. Now, the design requirement is T20 bars at 150mm spacing, in both directions, and for these areas of 2.3 metres by 1.8 metres, only one T12 shear link was found.

Now, given that the depth of these honeycombs, which is about a foot deep, would it cause -- well, perhaps look at another honeycomb. Bundle ER2, tab 17.10, the same tab, but the second file, page 72. This is the record of another honeycomb area designated as DS19. Now, we see the size of the honeycomb is 2.5 metres by 2 metres by 260mm, which is about 10 inches, slightly more than 10 inches, deep into the slab.

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If we then scroll down to look at the records regarding the shear links observed, again the requirement is T16 shear links at 150mm spacing, and for this area of 2.5 times 2 metres, only one T16 shear link was found.

This shows two things: first of all, our belief that if shear link does not show when we remove the concrete cover, we assume that they must have been hooked at inner layer. But these two honeycomb locations suggest that perhaps it is otherwise.

The other thing is when we talk about lack of tie, because of the lack of tie after concreting, we can't control the spacing, it may have been moved by the concreter, but if you are looking at an area of 2.5 metres wide by 2 metres wide and there is only one shear link found, would this phenomenon be attributable to the fact that shear link was not tied at the bottom and it was moved accidentally?

A. Well, the first thing is they clearly weren't tied at the bottom, because of the way in which they were installed. That's not fatal and I think you and I accept that that's not fatal. It's a little bit misleading to say the depth is whatever it is, 260 millimetres, because I think people would imagine this area, being the size you have described, 2 metres

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square and a foot deep -- well, that's not the case, is it? We had these chasms, as it were.

So unfortunately -- I can see why you are saying what it is, but I couldn't take that chasm as being a reasonable judgment of what was happening elsewhere.

Q. Right.

A. Because if it had been linked at a higher level and you're correct there's this chasm or this hole which is locally a foot deep and you don't see one, that's not to say that there isn't one quite close by.

So I can understand where you're coming from and I can understand the alarm bell going off in your head, but it's not conclusive. It is not conclusive.

Q. Fair enough.

A. But I sympathise with the concern.

I think all these areas are in the mid-span, aren't they, generally, or -- it's not relevant. It was an idle question.

Q. I can't tell for sure. What I can tell you is we have a plan showing the location of the honeycombing --

A. Oh, yes, I appreciate that.

Q. -- it's spread all over the place.

A. But I think you'll find they were mostly actually at the great concentrations of reinforcement at mid-span where, to be quite candid, I wasn't concerned. I'm much more

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concerned where there is a real shear demand, and there aren't that many, are there?

Q. Can I quickly finish off another topic, about arching effect.

A. Arching, yes.

Q. Do you recall that Dr Lau showed a few slides with openings?

A. Yes.

Q. As a matter of principle, if a slab is supported on two ends, if there's an opening in between, we can't expect any arching effect; do you agree with that statement?

A. If you looked at it in two dimensions, you're absolutely correct. But these structures aren't two-dimensional. You have a hole and then either side of that you've got a rib. If you like, the architecture of cathedrals demonstrates this, that you have thrust lines and in between you have voids and then you have cross-arches.

So the fact you have a hole doesn't change the arching principle, because the arching principle occurs where you don't have a hole, and then in between you get counter-arches onto those main ones.

Barrel vault is a classic. You can put a hole in a barrel vault and it won't fall down, as long as you've got that primary. So yes.

Q. So what you are saying is it all depends on the size of

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the hole?

A. Correct. In fact, it depends on geometry. But arching effect, I don't think there's any challenge that it happens, particularly when you've got aspect ratios like this, a 3 metre deep slab.

I'm thinking of flying buttresses, for example, and they don't suffer from shear. They are direct axial forces.

Q. If I may, I will move on to trough walls. In your second report, where you talk about trough walls, paragraph 5.9, you list out a number of factors. You list out the difference between the configuration of the reinforcement, the condition, the location of the trough wall, how it is different from the other station box structures, and on that basis you believe -- you think that one should have a greater confidence on the quality of the coupler connections in the trough wall.

Yesterday, you also mentioned about smaller bar diameter, visible, touchable, inspectable.

A. Absolutely. You've got it right. Yes, that's exactly what I said.

CHAIRMAN: Sorry, can I just ask, a trough wall -- I mean, to me, a trough conjures up images of farm animals, and so putting a head into a trough. So that's a long wall so it seems to me like it's a wall with one side, left,

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and then another side?

A. It is. It's a U shape. So you've got the walls of the U shape.

CHAIRMAN: Why do you have trough walls?

A. That's a good question.

CHAIRMAN: As opposed to block walls, you know.

A. Oh, right. Generally -- this is a siding and I don't know why they have any walls, to be honest. There must be some operational requirement.

CHAIRMAN: Okay.

A. Because normally in a siding you would just have the trains themselves. The way that derailments are normally dealt with, if you are concerned about them, is you have what's called a derailment kerb, which is a second rail, so that if the train does come off -- because of the geometry of wheels on trains, they do have a habit of rolling up under certain conditions, but the check rail is to get the derailment. So that's the way you normally deal with derailment issues.

I've never really understood why they have the trough walls. I thought it might be because they wanted a walkway to be able to get access, because obviously if you are at track level and you are maintaining a train, you might want to have access to it, like a walkway on the side. I don't know why it's there. It could be

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that there was concern about protecting the columns.
MR CHOW: The columns, yes.
A. But then you would focus on protecting the column rather than building a trough wall. But, sir, I don't know where it came from.
CHAIRMAN: Thank you.
A. It's not a function of a siding. You don't have trough walls in every siding. So the concern must have been protection of the columns, which I think you have alluded to in the past.
MR CHOW: Right.
A. And I'm saying you would have dealt with that in a different way.
Q. Sure. We will come to that. One of the issues considered by the Commission in this second round of Inquiry relates to the problem with the stitch joint and the shunt neck joint. I understand it's not part of your brief but are you aware of the conditions of -- the steel fixing conditions of the original stitch joint or the shunt neck joint?
A. I'm not aware first-hand. I have received third-party statements about the workmanship at the stitch joints, yes.
Q. Can I just quickly show you the picture showing the condition of the coupler connections in the original

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stitch joint. Bundle DD2, pages 725 to 729.
Page 725 shows the location. You see, at the right side, do you see the line separating the two contracts, contract 1111 and contract 1112 --
A. Yes, I do.
Q. -- on the lower part, on the right side?
A. Yes.
Q. The shunt neck joint, do you see the arrow pointing at the dotted line? This is the shunt neck joint at the interface between the two contracts; do you see that?
A. Yes.
Q. The shunt neck joint here is on the EWL level; do you see that?
A. Yes.
Q. First of all, the first point I would like to make is: the working condition of the shunt neck joint here is quite similar to the trough wall. It's an open area, with good daylight, visible, touchable, inspectable; right? The bar diameter is only 20mm; small diameter.
If we look at page 731, 731 shows the reinforcing details. Can I borrow the gadget where I can point, to help out? Thank you very much.
You see this (indicating), you see a T20 here --
A. Yes.
Q. -- where my hand is? T20 shows the horizontal

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reinforcement. So the reinforcement used in the shunt neck joint is a small -- compared with the platform slab is a very small diameter, T20.
If we go back a few pages and look at the condition of the connection. Perhaps 728. Now, what we see from 728 is actually the bar was not properly screwed into the couplers. Now, if it is not as clear as I describe, perhaps we can look at the description set out in the report at page 721, paragraph 3.5, where it describes what was observed at that time:
"At the chipped off locations at both East Wall and West Wall surfaces, rebars were found connected to the reserved couplers at the shunt neck structure built under [contract] 1111. However, as seen from the photos taken at the East Wall, rebars fixed by [contract] 1112 appeared smaller in size than the rebars fixed by [contract] 1111. The connection appeared to be a slot-in connection rather than a threaded-in connection. The connection location could not be considered as a proper construction ..."
If we then take a look at another situation, the VRV room -- are you aware of the problem with workmanship, coupler connection workmanship --
A. I'm aware of stated problems, yes.
Q. As far as layman is concerned, the working condition for

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the slab in the VRV room again is under open area, smaller diameter of reinforcement, inspectable, touchable, all that, as in the trough wall. But nevertheless Leighton managed, in the case of the shunt neck wall, to pour concrete. So apparently they managed to pass the hold-point inspection. I appreciate there is some issue between MTR and Leighton as to whether hold-point inspection was actually taking place, but what we see as a layman is when Leighton, with such a good working condition, handled with smaller diameter bars, managed to produce couplers of that level of quality.
In view of all this, can you explain why we can, in the case of a trough wall, have higher confidence that the quality of connection in the trough wall is of a better quality? Can you clarify?
A. Sorry, that's the question? Well, each -- you've only shown me the one photograph which is the interface between two contracts, and clearly there has been malpractice, really, in that position. You can't say otherwise. They are two separate contracts, which means one contractor came along, I guess Leightons came -- well, contract 1112, looking at this drawing, must have come after contract 1111, so 1111 had put the coupler in, had used what looks like perhaps a 25 millimetre bar

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or something, I don't know. But the coupler looks to be a bit larger than was required.
Q. Yes, what's described in the paragraph, a smaller diameter --
A. Yes, and visually it looks like that, and somebody just pushed the bar in.
Q. Right.
A. Is that correct --
Q. Yes, yes.
A. -- or did they try and screw it? The photograph is not that good.
COMMISSIONER HANSFORD: Sorry, I think we are missing a piece of the information here, aren't we? Because we now know that the coupler in contract 1111 --
MR CHOW: Is different.
COMMISSIONER HANSFORD: -- was in fact a tapered coupler, a Lenton coupler, and this appears to be an attempt to connect a cylindrical threaded bar to a tapered coupler.
MR CHOW: Yes, I appreciate that. But the point I'm trying to make is working conditions are similar to the trough wall, touchable in open area, small-bar diameters, inspectable, yet what we obtained is a defective product.
A. Yes, but I think for different reasons, aren't they?
Q. Yes.

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A. It's becoming clearer now, this is -- there is no way I would defend such a situation, but somebody should have said, "This isn't right." But that doesn't affect the steel fixer who's going to connect this coupler. Your VRV room is probably more appropriate.
Q. Absolutely.
A. Oh, good. So can we leave this one aside because it doesn't seem to be appropriate?
Q. Yes. Let's have a look at BB8, page 5793, please.
Here we see that -- ultimately concrete was poured. We see we have, luckily, photo records that shows that the threaded bars were not properly screwed in. For your information, in this particular instance, it was spotted by MTRC's inspector. The steel fixing works were condemned by MTRC, but nevertheless Leighton proceeds to pour concrete.
But the point is the working condition for the steel fixers on that occasion is similar -- I would say, using your description, touchable, under open area, visible -- but it does not assist at all. We still have these defective coupler connection that was done by the steel fixers.
The two examples I have just shown you would have an inspection process. The trough walls that we are talking about, as far as I understand, there were no

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formal inspection process. My question is, in view of what Leighton is capable of doing, even with an inspection process, what is the basis for us to have confidence of the quality of the coupler connection for the trough walls?
A. That's the question?
Q. Yes.
A. Clearly, these two couplers are not correct. I don't know to what extent that's true of the volume of the effects, but clearly that isn't, and the fact that Leightons -- it was brought to Leighton's attention and they continued I think is clearly not correct.
And I can understand that's why you have the concern that you do, but that doesn't explain to me why -- for the trough walls, for example, because they seem to be the key issue, why some even nominal opening-up couldn't have taken place to give some assurance, because what you've shown me and what I understand is, those are the sorts of things which would set the alarm bells ringing, and for that reason I do understand the actions that might have been taken. But some of the question marks I would have pursued would have tried to answer some of those questions.
Q. Yes.
A. And that wasn't done, to the best of my knowledge.

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So superficially, you know, there's something here. I would have gone a bit further.
Q. Right. In fact I intended to ask you whether you are aware of any reason why MTRC failed to do any opening-up exercise to ascertain the quality of the coupling connection for the trough wall.
A. I don't know. I didn't give advice on that. I wasn't asked for advice.
Q. Okay. Can I then now move on to the last topic I would like to discuss with you, about the design of the trough wall, the yield line analysis done by Mr Southward. You have given some comment in your report about whether the yield line assumed was the proper one, but we don't have an answer for the time being and we are not going to have an answer --
A. No.
Q. -- even with our discussion.
A. No.
Q. What I'm trying to get your agreement on is that, as a matter of principle, if one uses a yield line analysis, we are getting into an area of plastic design; right?
A. Yes, absolutely.
Q. And the yield line that we assume, we assume that at the time of failure, coming back to our case, we assume that

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when the train hits onto the trough wall, a yield line will be formed; right?
A. Yes.
Q. Assuming the pattern suggested by Mr Southward is a correct, basically, if one wants to visualise what actually happened, it's that the section of the concrete along the yield line will undergo plastic deformation?
A. It hinges.
Q. It hinges, like a door hinge?
A. That's right.
Q. Mr Southward was asked whether he has checked the deformation. I would take it -- perhaps a more proper description would be the lateral displacement of the wall when this so-called hinge of yield line is formed, and Mr Southward said he was checking ultimate limit state and therefore, as a matter of rules, he doesn't have to check the lateral displacement; right?
But do you agree with me that if there's a column at close distance from the trough wall, then it is not the normal situation -- if one uses a plastic design, one has to ensure that when the hinge is formed, the piece of triangular wall above the hinge line will not flip sideways so as to touch or damage the column; you agree with this? Someone has to look into this?
A. I would agree with you that with any impact -- and my

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experience is much more to do with ships smashing into things, but it's the same principle. Indeed, when I was much younger, we were involved in train impacts related to nuclear flasks, so I do have a bit of experience.
These are dynamic problems and you have to -- it's based on energy absorption. I would rather move my answer to the fact that the model is wrong anyway.
Q. Right.
A. I believe in the use of a yield line for a situation like this. I wouldn't call it a yield line. An energy absorption, a plastic thing. And because of the other aspects which haven't been taken into account into the model, you would not get the sort of movements, the idealised idea of this door hinge, because I understand -- your description is good, understanding, but the parameters of the model are not correct.
Q. Right. You are talking about all the soil behind and --
A. It's the soil and because of the restraint from the slab above, you wouldn't get that particular yield line. That's why I raised the question. And because of the oversite concrete at the top and because of the soil, a lot of that impulse, because that's what it is, it's an impulse, is actually dissipated into the soil and also into the adjoining wall.
Q. Yes.

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A. And unfortunately, because the oversite concrete was cast against the column, the column also shares; all right?
Q. Yes.
A. But the yield line that you then get has got a lot more energy absorption than the hinge, because the amount of energy is related to the length of the yield line and its rotation.
Q. Rotation, yes.
A. But once you start to bring in other constraints, the actual amount of energy which is absorbed through the yield line increases enormously.
Q. Dr Glover --
A. So all I'm saying is, I'm not making a judgment as to whether the calculation is correct or wrong, other than to say it is not the appropriate model to consider, and nor is this elastic cantilever. The fact that the elastic cantilever passes the test to me demonstrates there isn't an issue, because once you start to look at the plastic design, you will find it will pass. That's all I'm saying.
Q. Right. Dr Glover, let's see if we can better assist the Commission by separating these two. I appreciate that you have a further point, saying that we have soil behind the wall and we have a slab --

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A. Yes.
Q. -- right, which shares a loading. But let's, for the time being, assuming -- I'm trying to assist the Commission to have a clearer picture -- assuming we don't have any soil, we don't have a concrete slab behind it -- because this is not what Mr Southward assumed, I'm just at the moment trying to focus on the theory --
A. Yes. I'm trying to give evidence, I'm sorry, Mr Chow --
Q. Of course.
A. -- and I'm giving my evidence not on the basis of whether the calculation is valid or otherwise, except to say I don't agree with the calculation.
So to ask me whether it has repercussions, I don't think you are going to get the answer you want, because all I'm going to say is: I wouldn't have done it that way and I don't have a judgment of what was done in terms of its consequences.
All I would say is the fact that the designer, AECOM, was happy with a cantilever design which satisfied all of the criteria, and then he applied a 35 per cent reduction factor and it didn't work, I am saying there are other devices at work that would have compensated for that loss of strength of the 35 per cent, which are not taken account of in the

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calculation.
That's my -- it is a superficial judgment, I agree, but I think I'll be found to be correct. But if you took into account, even the cantilever model, with the 35 per cent reduction factor, but you recognise there was soil and there was a slab -- because it seems a bit sort of other-worldly, isn't it, to say, "Ah, yes, let's assume the soil isn't there and the slab isn't there"? Well, it is. So don't you analyse what's there? And then have arguments like, "Oh, someone might come along and dig it out" -- well, I mean, that doesn't -- the other thing, if you really want me to get into detail here, the impact load which is being given is just too large.
Q. Dr Glover, I can assure you, I will get into a discussion with you regarding the soil later on.
A. I'm sorry. In the fullness of time.
Q. Let me finish this part first.
A. Yes.
Q. For the sake of our discussion, if there is no soil and concrete slab behind, on the basis of our discussion earlier, you would accept that for someone who carries out design with yield line, in these particular circumstances where we have a column very close to the trough wall, you would have to check the lateral

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displacement where the train hits the trough wall? Are we in agreement on that?
A. I think, if that was the design objective, you would have to ask yourself those questions. You would have to, yes. But the fact that there is soil there, the fact that unfortunately, the oversite concrete is in concrete with the wall, so why was that done? And interestingly enough, I don't want to sort of -- no, I won't. I won't go there.
Q. Let's discuss about the soil and the concrete slab. This is really the last area that I would like to explore with you.
A. Great. Good. We can continue the conversation afterwards, can't we?
Q. Of course.
A. I enjoy the conversation.
Q. You mentioned about there is soil behind the trough wall, and the soil would help to resist the impact load, and also there's a concrete slab which would share the impact load between the two trough walls.
A. Mmm.
Q. Now, the government can only act on the information that we have on the drawings. On the drawings, there is no concrete slab.
Can I show you the relevant drawings.

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A. I didn't call -- I might have called it a slab, but on other occasions I've called it oversite concrete, because what it is, you see. I think I've said that on other occasions, and in a dynamic problem you would include that in it. But I agree it doesn't have an official title called "slab".
Q. All right. Perhaps it's helpful for us to call up the relevant drawings. Bundle DD8, page 11248.
Now, on the right side of this drawing, it shows -- actually, you see there's a vertical dotted line. I believe it shows the centre line of the column.
A. Yes.
Q. And we see the two trough walls on each side of the column; right?
A. Mmm.
Q. This is the information given to the government, and this is what is shown in the accepted design.
In the accepted design, we only see there are -- it looks like we have soil backfill --
A. Mmm.
Q. -- between the trough walls, but we don't see any concrete slab.
In addition to the concrete slab that you said, which might help to transfer the loading, my understanding is that during a site visit amongst the

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experts, someone observed there is a concrete surface on top, in between the trough walls, and I believe that one of the MTR staff informed the expert that these are lean concrete.
A. Yes.
Q. In other words --
A. It's oversite concrete, yes.
Q. -- there's no reinforcement and the concrete strength is not guaranteed, it's not supposed to be structural, and there is no information as to the thickness of the so-called lean concrete layer.
A. Yes.
Q. To make use of this so-called concrete slab -- assuming it is really lean concrete, because it's the only information we have today, as it's not on the drawing, it's not part of the design -- do you agree it would be rather risky to rely on the lean concrete, without knowing the thickness, and knowing that there is no reinforcement in that layer of lean concrete, to transfer the impact load to the other side of the wall; would you agree with this statement?
A. I agree entirely, but when you are looking at a particular problem area and you are trying to appraise it, then you take into account all of the parameters. I think you and I would agree that this is a compressive

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situation and if the concrete could be shown to be there and of a certain thickness, then in a dynamic situation you would include it.

I would observe, though, if that concrete goes in front of the column, then it should have been dug out, shouldn't it, if you are worried about contact between -- I mean, you've got to take your solutions through, haven't you, as we discussed yesterday? If you are worried about the column, then really it should have been isolated.

Q. All right.

A. And it's not, so you haven't really helped the situation -- sorry, not you; whoever.

Q. Now, about the soil backfill behind the wall. Obviously, the main concern is the risk of damaging the columns, and the most critical locations is where the distance between the existing column and the trough wall is very close.

According to Dr Lau, the columns that we -- perhaps let's go to look at drawings -- bundle DD19, page 19058. This is a drawing that I think Mr Southward has been shown. We see on the right-hand side a vertical solid line (indicating). Do you see that?

A. Yes.

Q. You see the marking "MJ"; do you see that?

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A. Yes, I can. Thank you.

Q. The MJ, in engineering drawings or construction drawings, usually signifies -- stands for movement joint?

A. Mm-hmm.

Q. And this vertical line, the solid dark vertical line, shows the alignment of the movement joint; do you see that?

A. Yes, I do.

Q. Okay. If we then go down a little bit in the drawing, and then if we come to here (indicating) -- so we have a situation where a movement joint is very close to -- you know the dark circle shows the existing column; correct?

A. Yes. Sorry, yes, I do.

Q. And the sort of rectangular -- these two lines here (indicating), where my hand is, is the trough wall, just next to the column; do you see that?

A. Yes, I do.

Q. So because of this movement joint here, so we have a situation similar to by Mr Southward at page 13 of his report, do you see that, with the triangular thing?

A. Yes, I do.

Q. So we have a similar situation here.

According to Dr Lau, the gap between the existing

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column in that location, between the column and the trough wall, is about 60 millimetres.

Do you agree with me that under normal circumstances, even if there is soil between the column and the trough wall, with a gap of only 60 millimetres or even less, the soil would have been placed loosely, could not have been properly compacted; right?

A. Mmm.

Q. And would you agree with me that for someone who needs to rely on the resistance of soil to help to resist any impact load, the soil first of all has to be properly engineered, has to be properly placed, properly compacted, so as to provide the necessary resistance?

A. Its resistance comes from another of aspects. I don't want to get into the dynamics now. But one of them is -- it's just its mass. You know because it's an inertia problem, dynamics is all about something coming in and mobilising the motion of other things. So it's not just compaction, it's also the mass.

Q. All right.

A. The other thing I was saying -- you see, the trouble is, it's an artificial loadcase, isn't it? I mean, when I look at this diagram showing this line loading acting at the top of a wall, it's very idealised. And what you are doing, quite correctly, is you are applying

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an idealised situation to a specific one.

I'm saying you are extrapolating it too far, because if you were to take that line load, for example, and apply it to this particular situation, you would see that that line load extends well beyond the column. You know, it's -- so beyond the column all the aspects that I was just describing would be in play, wouldn't they?

Q. Right.

A. All I can answer -- I'm sorry, I know you are trying, and you are being very helpful, actually -- but I can only come back to you and say I can see mathematical models which would take your idealised loading and would demonstrate that it would be all right.

Q. Okay. My last question is -- now, I appreciate that there is a suggestion that in real life the train that gets into the HHS area would be empty, it would travel in low speed, and also the impact load would not be acting perpendicular to the wall. These are some considerations that perhaps one can give a little bit of leeway in terms of checking the capacity.

A. Yes.

Q. In that situation, am I right in saying that if one uses a lower impact load, because of these considerations, and find that the trough wall is of an adequate capacity, one can only say that this trough wall is safe

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if the impact load that is going to be experienced in the case of a derailed train is less than what is set out in the design code?
A. Yes. It goes back to the core of the whole investigation. A forensic looks at not what the stated requirements were from the client, in terms of safety, that is.
Q. Mm-hmm.
A. One looks at what the realism is.
I did some rough calculations, actually, and for the life of me I'm trying to remember them, but this force here would still represent something like half the line speed of the train, and it just wouldn't be travelling at that speed. It just can't physically do that.
So to answer your question, satisfying the stated requirements is an issue, but if you are asking me for my opinion as to whether there is a safe -- I won't even use "fitness for purpose" here -- but if it's safe, then I say it's safe.
But I take on board your point that from a compliance point of view, it probably doesn't, or is to be tested. But from a "safe" point of view --
CHAIRMAN: I apologise for interrupting -- "safe" in what respect? Because we've got a column, but we've also got the trough walls and we've got trains.

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A. I think I'm being invited to put forward what I would consider to be the proper design problem, rather than address -- you know, saying, "If I apply this load, I would get that result."
CHAIRMAN: Yes.
A. So I'm saying that I recognise that protection of the column is fundamental. How would I have approached it as a designer to avoid that problem? I wouldn't have applied this force, for example, and I wouldn't have used that analogy, and I would have used all the parts that make up the structure to do that, and on that basis I believe it is safe.
I'm not going -- and I hope I've never even strayed into saying it complies with the client's requirements, et cetera. I'm not saying that.
MR CHOW: Right. There's one last question --
CHAIRMAN: Sorry to interrupt.
Do I understand -- so what we're talking about here, really, is a design problem? We are not talking about a poor workmanship problem?
A. No. It's the repercussions of application of a reduction factor. If the structure had been -- no, nobody is questioning the construction. People are saying, other than the couplers have to have a reduction factor applied to them, that weakens the structure as

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designed and, as a consequence, you have this issue. But if you didn't apply that 35 per cent reduction, it satisfies all the requirements.
CHAIRMAN: Would that be right?
MR CHOW: Yes, because under the original design, Atkins is based on the elastic design and, if there's no problem with the coupler connection, the trough wall is able to resist the design loading, the impact load.
CHAIRMAN: All right. So you are saying essentially, therefore, it is a workmanship problem at the end?
MR CHOW: With the couplers.
CHAIRMAN: With the couplers.
MR CHOW: Yes.
CHAIRMAN: So then they are the cause of what we are now debating?
MR CHOW: That's correct, Mr Chairman, yes.
May I ask one last question, about this train getting into HHS will be running at low speed and that it would not hit at a right angle to the trough wall. If a layperson comes to you and asks, "Under normal circumstances, the train will never hit the trough wall, it will only in a situation where there is an accident. How about, when there's an accident, the train running in normal speed got into the wrong track and accidentally get into HHS? Can we rule out this

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possibility when it comes to accident?" What is your answer to that?
A. Well, I would have to look at the signalling system, because switch and crossings are controlled by the signalling system.
Q. Yes.
A. So somehow -- the train driver can't suddenly press a button to change the switch and crossings to go into the siding. It's dictated by the signalling system, not by him. And the possibility of that is extremely remote, because the signalling system is tied to so many other points.
Q. So, in other words, what you are suggesting --
A. And that is a bit of -- you do come up with some nice questions!
Q. Are you suggesting that --
A. In railway terms, it's called wrong-roading. You know, you shouldn't be doing it, in other words.
Q. So are you suggesting that actually the design load specified in MTRC's design manual is too high; it should be lower than --
A. No. But this a special -- no, in the manual, it doesn't cover this situation. This was, I understand, I stand to be corrected, that this was a factored-down loading from what the loading would be on the line. On the

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line, they consider this as impact loads, but the load is higher than this, because the train is travelling higher. I understand that operations said, "Okay, it would be silly to apply the full line load, we will apply a different one", and they reduced it. But what I'm arguing, on the basis of just shear dynamics, it's still higher than I would have arrived at.

Q. I stand to be corrected as well. My understanding is this impact load is actually specified by MTR in MTR's New Works Design Manual. So if there is any suggestion that this specified load is too high, unrealistic, should have been revised --

A. No. I think you will find the line -- as I say, I stand to be corrected on this, but I'm trying to help the Commission here -- I think within the manual there is an impact loading, and it's for line speed and it's higher than this, and this is a special situation, and I think this was a special advice. I don't believe this is in the manual. This was a project, a contract-specific loading. But as I say, I stand to be corrected.

You certainly wouldn't be applying line speed here.

Q. All right.

A. Can I also say the energy of a train is related to the square of the velocity. So, if you are doing a dynamics

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problem -- so that's really why, if you say the train is travelling at half the speed, then actually that would be a quarter less force. The trains here will be travelling at a fraction of the line speed.

So I'll leave it to you to do the mathematics. So you can see the forces are pretty incidental.

MR CHOW: Thank you very much, Dr Glover. Thank you for your patience. I have no more questions for you.

WITNESS: Thank you very much.

MR CHOW: It's a pleasure.

CHAIRMAN: Excellent. Thank you very much. Thank you, Mr Chow.

So, Mr Khaw, I'm not suggesting you do it now, but will there be questions for -- I'm sorry, Mr Boulding, yes, of course.

MR SHIEH: I have one question.

MR BOULDING: It's Mr Shieh next.

MR SHIEH: I can save it until after lunch.

CHAIRMAN: Okay.

WITNESS: Does that mean --

CHAIRMAN: I'm sorry, I had overlooked you. My apologies. Thank you very much indeed. So we will just have some more questions after lunch.

WITNESS: Okay.

CHAIRMAN: But it shouldn't be too long. We'll make it 2.40

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so we've got the normal time. Is that all right?

MR PENNICOTT: Let's make it 2.30.

CHAIRMAN: We'll keep it at 2.30 because we are going to finish at 4.30 this afternoon.

MR PENNICOTT: Yes, sir.

CHAIRMAN: Thank you.

(1.10 pm)

(The luncheon adjournment)

(2.32 pm)

CHAIRMAN: Mr Shieh, my apologies for getting the orders a little muddled up just before lunch.

MR SHIEH: It's perfectly fine, Mr Chairman. I intend to stay a bit low-key during this part of the hearing.

Cross-examination by MR SHIEH

Q. Dr Glover, good afternoon.

A. Good afternoon.

Q. Could I refer you first of all to your own report just to lay the groundwork, paragraph 7.11.

Here you said you are in agreement with Mr Southward on various things, and specifically at (ii) you say:

"I acknowledge and support the point that Mr Southward has made in sections 6.7 and 6.8, [which is] that 28 millimetre engagement has been shown to satisfy the strength requirements of CoP ..."

But you then went on in the next sentence to say:

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"However, my selection of 32 millimetre engagement is based on the fact that it has been shown beyond reasonable doubt to pass any test that can be sensibly applied, albeit I acknowledge that neither 28 ... nor 32 ... meet the contract requirements."

Let's leave contract requirement to one side because we are not concerned with "code compliance"; right? 28 millimetres, although we have been working on the basis of one thread equals 4 millimetres --

A. Mm-hmm.

Q. -- because of the 2 millimetre chamfer and the 2 millimetre half-thread allowance, 28 millimetres translates to six threads; correct?

A. Correct.

Q. I don't think I need to look up Mr Southward's actual report, but Mr Southward actually says whether you use 28 millimetres, that is six threads, or 32 millimetres, ie seven threads, they pass the requisite strength requirement, static tension test; right?

A. That's correct.

Q. Now, you have somehow preferred -- if it's not the correct word, tell me -- you prefer 32 millimetres because it has been shown to -- shown beyond reasonable doubt?

A. Correct. It is beyond reasonable doubt.

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Q. My question is this. The relevant test strength is 529 megapascals.
A. Yes.
Q. Which, and again I hope you remember, is passed by six threads and seven threads in the tests.
A. Correct.
Q. So does it not suggest that in terms of strength, six threads is good enough, because six threads, the minimum stress in order to break a six-thread threaded rebar, has been shown to be, I think, 565; do you remember?
A. Yes.
Q. That is higher than 529, so that would pass the strength test. So does it not suffice in terms of strength that six threads could withstand the requisite test to be applied?
A. Yes, if one is just considering the straight tension and, as far as I'm concerned, that would be adequate in practically every situation that one finds on the structure. The one caveat I would put on that is that in area A, when we looked at the requirement to redistribute -- in other words, there is an implied requirement for some degree of plasticity in the reinforcement, and as I showed on the steel stress-strain relationship, the 529 figure comes about from a straight linear approach. In other words, it's

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the limit of the elastic range. If you go beyond that, that's when you start to move into allowing a little bit of plasticity, but it's a very moot point. I took the 32 because, as I said, I wanted to be able to demonstrate beyond any shadow of a doubt that 32 would pass any criteria, and I'm sure you will get people, potentially, saying, "Ah, you've assumed some redistribution, so therefore you must be assuming there's some plasticity, and so therefore I want a connection which has that degree of plasticity in it", and it is a moot point and I'm being very defensive on it.
Why I didn't sort of defend the 28 more strongly or attempt to was quite simply: was there any requirement to do that? Because when I look at the statistics, if you look at the distribution of failure rates, you find the difference between 28 and 32 is really just a few percentage points, and rather than get myself bogged down in an argument as to whether there was plasticity or not, quite honestly the difference between 9 per cent and 12 per cent failure rate is just not worth the fight.
So it's a combination of reasons as to why I took 32, but I understand your point about 28.
COMMISSIONER HANSFORD: So I can understand that, when you

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say the difference between 28 and 32 was just a few percentage points, are you saying that there are very few bars where the engagement is somewhere between 28 and 32?
A. Correct. In fact, if I could refer you to -- I don't want to do this unnecessarily, but annex 1 to my report.
COMMISSIONER HANSFORD: Yes.
A. I give all the numbers there for different engagement lengths.
COMMISSIONER HANSFORD: Yes, you do.
A. And you see the difference between 32 and 28 is really very small. Once you are getting down to 12 per cent failure rates, I think from memory, for 32, and you're down perhaps at 9 and 10 per cent for 28, quite honestly, it's just not worth having the argument as to whether there's plasticity or not. Just go for 32 and you've used a ductile coupler, you've passed all the tests, so you are allowed to do as much redistribution as you like.
COMMISSIONER HANSFORD: That answers my question.
MR SHIEH: So I can put it quite bluntly: 32/seven threads is a "nice" point for the purist, whereas for the practical-minded pragmatist, six threads/28 millimetres would be good enough?
A. As long as you can deal with the issue of, "Ah, but you

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assume some redistribution, therefore you are assuming some plasticity going on." That's all I'm saying to you.
MR SHIEH: Thank you very much.
Re-examination by MR BOULDING
MR BOULDING: Good afternoon, Dr Glover. It's a long time since I threw you to the wolves, but to be fair to them they have given you a full opportunity to explain yourself. In the light of that I only have one topic upon which you might be able to give the learned Commissioners further assistance.
Do you remember giving evidence yesterday about cube tests and core tests to establish the strength of concrete?
A. Yes, I do.
Q. I wonder if we can just remind ourselves of the transcript for yesterday, Day 10, and if we could go to page 74. I'd like to pick it up at line 22, just to remind you the evidence you gave. You return to the witness box and say:
"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.
Dr Lau referred to he would have great confidence or greater confidence in our hypothesis of increased

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strength if there had been cylinders taken and tested."
Then Prof Hansford said, "Cores", and you answered:
"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 ..."
Then you go on.
I wonder whether we can look at some of the evidence that was put before the learned Commissioners in the first hearing. For that purpose, I'd like to look at a statement from Mr Michael Fu. We can pick that up at B13679. I think that's B16/13679.
There we see a reply statement, and if we can scroll down a bit we see that he's the construction manager, do we not, for MTR? Paragraph 1; do you see that?

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A. Sorry, I do apologise. Yes, I can.
Q. Then if you would be kind enough to go on to paragraph 16. That's on B13682. He tells the learned Commissioners:
"As explained in paragraphs 14 to 17 of the second witness statement of Mr Louis Kwan (ConE 2) (which I have reviewed), three random concrete core samples were taken from the top of the EWL slab track slab in October 2017, and another three core samples were taken from the top of the NSL track slabs in December 2017, and those core samples were then tested at MTR's HOKLAS accredited project laboratory in Tseung Kwan O."
Then 17:
"The core test reports dated 2 November 2017 (EWL track slab) and 18 January 2018 (NSL track slab) were formally submitted by Leighton to MTR under CSF no. 1112 [and then it gives a reference number] on 26 January 2018. These core test reports demonstrate that the compressive strength of the concrete tested is satisfactory and, more importantly, that there was no honeycomb concrete at the core locations. These test reports were submitted to the BD's Mr Lok Pui Fai by MTR's letter [of] 17 July ..."
I wonder whether we can have a look at that letter together. That's, I trust, at B17/14210. There we've

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got a letter of 17 July and I wonder if we can scroll down. I hope that at B17/14220, there do we see a concrete core test report, Dr Glover?
A. Yes.
Q. Is this something you have seen before?
A. No.
Q. Could you take the opportunity to scroll down and look at the results we see there, in particular at the bottom of the page, do you see the little box entitled, "Load at failure (kN)"?
A. Yes, I do.
Q. Would you look at the results therein and tell me what, if anything, they tell you about the strength of the concrete?
A. They show a very consistent pattern -- or individually they fit very well within the distribution we discussed yesterday, which is a mean of around about 80 and an absolute minimum of 60. I could see that that would fit in that distribution extremely well.
Q. And in terms of design strength, is that a strength that you would regard as being satisfactory?
A. I would regard -- in isolation, you would say that's going to be a high strength, but I would need to look at the data more closely. I would say a safe design load there would be -- stress would be 60, agree on that

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basis. On the basis that I'm just imagining the shape of the bell curve, with a mean at about 80, then I would think the 95 per cent pass rate would really be around about 60. It looks consistent with that.
Q. Then looking back at Mr Fu's statement, if I may, paragraph 18, this is B13683, and he tells the learned Commissioners:
"More recently, in July 2018, MTR instructed Leighton to carry out concrete core tests on the EWL track slab, for the purposes of the load test proposed by CM Wong & Associates. These tests were carried out by Fugro Technical Services Ltd, and the preliminary test results of the six core samples were circulated by MTR's Mr Raymond Chow and copied to me on 31 July 2018. Once again, the test results showed that the compressive strength of the core samples was satisfactory, and there was no indication of honeycomb concrete at the core locations."
I wonder if we can just look at that. I hope we find the document dated 31 July at B17 and page 14236. There do we see the email which is referred to by Mr Fu, Dr Glover?
A. Yes.
Q. If you would be kind enough to scroll down, I hope we find the results to which Mr Fu refers at 14238. There

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do you see the results starting at 14238, and take the opportunity --
A. 71.
Q. 71. And I think they go down through to 14243, just for the sake of completeness.
A. Sorry, just one second.
Q. Yes.
A. Yes. Sorry.
Q. Scroll down. Do we there see another result?
A. Yes, and they are very consistent, not just in the strength but looking at the other parameters.
Q. Right. Go down to 243.
A. Yes.
Q. There do we see further results?
A. Yes. Yes. Yes. Yes. Yes.
Q. Now, what, if anything, do they tell you, Dr Glover, about the strength of that concrete that was tested?
A. They would tell me that they are slightly less than the ones I saw earlier. I'd have to look at it more closely. I mean, not dramatically so, but these are certainly towards the -- they're shifted, as it were, to the left, but not hugely. I mean, 59, you know, would always stand out, but actually, if you took that with the family, you would expect that.
Because remember, we expect to have one failure in

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every 20 anyway, and they don't get anywhere near the sort of limits that I would be expecting. But no.
In summary what I'm saying is these results appear to be slightly less than the first three you showed me, but they are very consistent with the strength in the works being substantially larger than the design strength of 40, substantially. I mean, this is not sort of measuring a few percentage points. These are significantly larger. And the fact they have been taken at different locations, different times, is -- and you take that with a family of 6,000 other cubes being taken, it's very good evidence. In fact, I would think it really reaches the point where it's almost beyond doubt that you've got a situation where the concrete in the works is indeed substantially stronger than the design strength that was achieved, or was set out in the design of 40.
MR BOULDING: Thank you very much, Dr Glover.
Mr Chairman, Prof Hansford, I don't know whether you've got any questions.
COMMISSIONER HANSFORD: I have one question. I think you can predict what it might be, Dr Glover.
Questioning by THE COMMISSIONERS
I'm interested in your views for the future. How do you think -- what could be put in place to ensure the

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proper connection of couplers by --
A. Of this particular coupler?
COMMISSIONER HANSFORD: Of this particular coupler.
A. Yes, because I think you've got -- because of patents and all sorts of other things, they've all got their little touches.
COMMISSIONER HANSFORD: Yes.
A. But this is about as simple a coupler as you will get. It hasn't got the sophistication of a Lenton coupler.
This issue of butt-to-butt, I'm not convinced of it. There has been no evidence -- can I be outrageous? Yes, I can, because I'm normally slightly outrageous. I don't believe the general workmanship on this site in terms of the operatives, whatever, in terms of forming the connections, was substantially substandard. I don't think there was anything where the workers were of a lower quality. There is no doubt that sometimes people didn't fix it as well as they possibly could, but I think, if you take it as an average across Hong Kong, it would probably be reasonably representative, perhaps at the lower end.
Now, what does that mean? That means that in all of the diaphragm walls or the slabs or whatever which are throughout the construction in Hong Kong, you would have a very high proportion of partially engaged couplers.

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Particularly from a strength point of view, I'm not at all concerned about that, because of the evidence of the tests that we have.
So the issue really is this one of permanent elongation, and again, if you look at as-constructed buildings, particularly large infrastructure, I've never heard reports of this cracking, this large cracking. So I believe it's one of those things that the laboratory seems to throw up a question mark, but in practice there's all sorts of reasons why it's not happening.
That doesn't mean to say that there shouldn't be some studies on this. I think there should be because it puts people's minds at rest. But if butt-to-butt or something approaching butt-to-butt is important and is essential, then I think there are two things that have to happen.
One is you have to be able to establish what the free -- I'm thinking now of the situation where it's cast into a capping beam.
COMMISSIONER HANSFORD: Yes.
A. In other words, you can't see the other side.
COMMISSIONER HANSFORD: Yes.
A. So a simple device is to establish what the depth is, and it doesn't have to be to the nearest micron. It's just I have three threads, four threads, five threads --

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what is it? If it is small, then the hand should go up and the inspector should come and say, "Yes, I agree, this is a defective coupler", and a decision has to come out of it.

So I think measurement is important in that situation. Once you know what that distance is, then you know how much it's got to go in.

The other thing is, I think this thing about taking it so it's tight, that's not an engineering term. I think the idea of this -- and I think Dr Lau mentioned it -- a torque wrench. In other words, there should be a minimum specified exertion, and -- you know what a torque wrench is. It actually is a wrench but it has a dial which you've got to get to that level.

So I think those two things: measurement, and a scientifically arrived-at effort of screwing it in. That's for the capping beam situation. For the free-air coupler, I think it's pretty self-evident -- the operator is in control of the dimensions, but I still think the torquing by a controlled effort is certainly something which should be done, and I believe Lenton couplers require that. That's the vehicle ones.

Is that --

COMMISSIONER HANSFORD: That's very helpful. Thank you very much.

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CHAIRMAN: Good. Doctor, thank you very much indeed. Thank you.

WITNESS: Thank you very much.

(The witness was released)

MR PENNICOTT: Sir, we now come to the Commission's structural engineering expert, Prof McQuillan. I see the time and I was going to propose that obviously we will call Prof McQuillan. As you will appreciate, he has a presentation to do. I suspect that that's going to last the best part of an hour, possibly a little bit longer, and so what I suggest we do is we have this presentation and then we don't start the cross-examination, particularly as we've got to leave at 4.30 in any event, if that's satisfactory to the Commission.

CHAIRMAN: Yes.

MR PENNICOTT: Can I call Prof McQuillan?

PROF DON MCQUILLAN (on former oath)

Examination by MR PENNICOTT

MR PENNICOTT: Prof McQuillan, as with Dr Glover, we won't, as it were, take the oath again, as you did last time, and we will regard you as being on the oath that you took on the last occasion you were in the witness chair.

Can I then just run through the position on your reports, if I may. You've prepared two reports for this

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part of the Commission, part of the Inquiry. Could we first of all look at the first one. It's at ER2, tab 15.1. Is that the front sheet of your COI 1 report?

A. It is, yes.

Q. Dated 6 December.

If we go to page 10, please, is that your signature?

A. Yes, it is.

Q. Confirming the date of 6 December.

Prof McQuillan, I understand that there's one erratum to this report which we'll find at tab 15.2, I hope. Is that the erratum that you wish to make to the report we've just looked at?

A. It is, yes.

Q. Then the second report, could we go to ER1 in the COI 2 file, and tab 11, please. Prof McQuillan, is that the front sheet to the COI 2 report?

A. Yes.

Q. If we go again to page 10, that's your signature?

A. It is.

Q. Then, in addition to those two reports, Prof McQuillan, there is a joint statement and a supplementary joint statement. Could we again just look at those, pick up the references, stay in the same file, ER1(COI2), tab 14.3, and is this the joint statement of the experts made on 20 December 2019?

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A. It is, yes.

Q. Yourself and Dr Glover in London, and Dr Lau and Mr Southward in Hong Kong?

A. That's correct.

Q. This is the typed-up version, and I think it's all accepted that that was signed up by all four of you?

A. It was.

Q. Then at 15.2 in the same file, this is the supplemental memorandum of agreement, signed more recently. If we can scroll down, please. That's signed by all four of you?

A. That's right.

Q. Prof McQuillan, so far as the reports and joint statements that we've just looked at contain expressions of your views and opinions, are they opinions honestly held by you?

A. I confirm that.

Q. So far as the reports and statements contain matters of fact, are they true to the best of your knowledge and belief?

A. Yes.

MR PENNICOTT: Prof McQuillan, at that point, I will sit down and allow you to make your presentation.

Presentation by PROF MCQUILLAN

A. Thank you. Thank you for the opportunity.

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As I proceed, the Commission will note a lot of commonality and repetition with the evidence given by Dr Glover in particular. That commonality is not in any way contrived, and in fact my summary was drafted before I heard the evidence, obviously, of the other experts. I just make that a point.

I make no apology for the fact that Dr Glover, myself and also Mr Southward, we all appear to be singing off the same hymn sheet.

The severity and impact of some of the issues that have been raised I feel have been overstated in previous evidence and I hope to put these into meaningful perspective, if I can.

Could I say that those of us who visit universities and teach the students, we are expected to explain in simple language, if we can, concepts which are sometimes difficult to understand. There has been a lot of heavy technical stuff in the previous evidence. Hopefully I've tried my best to explain that in simple terms.

The directions of the Commission to the engineering experts, quite simple: are the structures safe, first of all, and secondly, are they fit for purpose? I know, as somebody has already said, that means different things to different people.

I always look at it in the sense: are the structures

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safe, that means failure has to be the key consideration, in the ultimate limit state sense. Are they fit for purpose? Different words have been used to describe this, but for me function is the key consideration. Is it good enough to do the job it was designed to do, without compromising durability and longevity?

As I've already said, item 1 is really a ULS issue, whereas fit for purpose is really a serviceability limit state issue.

This isn't really an attempt to teach Granny to suck eggs but just to illustrate my understanding of the two terms, if you consider a simple beam, it could be concrete, it could be made of steel, it could be timber or it could be another material, but it has a defined size and it has a defined span.

The beam has a strength capacity which is a function of its span and its size, the member properties, the materials and the loading that it carries. So it's quite simple. If the load applied to that beam is less than or equal to its capacity, the beam is deemed to be safe. Conversely, if the load applied exceeds the capacity, the beam is bound to fail.

However, although the beam could be safe, it could deflect or it could sag, and this is just one example.

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It could do that excessively under the applied loading. So finishes could be damaged, if that happens; glass partitions attached to the underside of the beam could break, in which case the beam would not be deemed fit for purpose.

The Hung Hom Station structures, however, are of necessity very robust. They can carry the full range of loading and, as has already been discussed, they have already carried 95 per cent of the total load. The figure of 90 per cent that we remember is dead load, but you must remember that there has been commissioning of the empty train sets, and so I'm saying that that probably accounts for probably 5 per cent of the residual 10 per cent dead loading. So that's how I arrive at the 95 per cent.

In addition, the slab deflections are minimal, and last time I demonstrated that by standing up and giving you an indication of how thick the slab was. Dr Glover has done that today already. So the slab deflections are minimal because of the depths involved. Cracking is non-existent.

CHAIRMAN: Sorry, what do you mean in layman's terms, Professor, as to deflection of something like a slab? Because I see a slab as being -- it's joined to the diaphragm walls. I can't see where it would deflect.

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A. At mid-span -- so if you can imagine that slab being much, much thinner, imagine it was only, say for example, half a metre deep, and supposing that half-metre depth was able to sustain the loading, its deflection, its sag -- let's call it a sag at mid-span -- would be excessive.

CHAIRMAN: I have it. Thank you very much.

A. So the cracking is non-existent, and that proves that the partially engaged couplers have functioned as intended. I will come back to this topic later, but any movement in the coupler assemblies under load take-up would already have manifested as cracking at the top of the EWL slab at the D-wall connections, and you've heard already from the other experts, we have visited the site; we've never seen any such cracking.

The internal environment is dry, and it's not humid either, and so no rebar corrosion is possible in any event.

So the HUH structures are therefore, in my opinion, both safe and fit for purpose, based on the understanding that I have described to you.

I make the point in my reports that a structure can be safe and fit for purpose and yet be only partially or totally code compliant -- sorry, non-code compliant, whichever way you want to look at it. I also make the

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point in my report that the trend nowadays is on performance-based design instead of prescriptive code requirements. This avoids building in unnecessary reserve capacity, such as is generated by the Hong Kong Code of Practice and indeed the British and European codes and other limit state design codes.

Performance-based design is already a well-known technique, for example in fire safety engineering where bespoke solutions are produced from first principles.

COMMISSIONER HANSFORD: Prof McQuillan, for the purposes of the Commission, could you explain performance-based design?

A. So the British -- if you take the Code of Practice and if you take its associated code on recommended live load, for example, and take, in conjunction with that, the client's performance requirements which may go in tandem, what you're normally doing in terms of live load, and the same happens for the rail loads on this job, you read them off a table, essentially. There's no attempt made to actually look at the specific loading and what that implies.

So performance-based design is really putting instrumentation on a structure like a bridge, monitoring its performance over a long period of time to see how you can translate that performance into the actual

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loading that it's actually sustaining as opposed to theoretically designed for, and inevitably we're going to find that there's a gap between the two, between the design load under the codes as is now and what the actual load is that the structure is subjected to.

Does that explain it adequately?

COMMISSIONER HANSFORD: Thank you.

A. I also highlight again, as mentioned by others -- I'm doing it in a much simpler way, I hope -- that the codes already contain these factors of safety, partial safety factors, and these cater for uncertainties in materials and loading.

To put it quite bluntly, even if a structure was designed to be 100 per cent efficient to the code, if you could do that so that there's no excess fat, Dr Glover has admirably illustrated that the structure still has a capacity of strength, a reserve capacity.

That is why, when a structure is retro-analysed or retro-assessed, it is permissible, as he has demonstrated, to use actual material properties. For example, the actual concrete strengths that we have been talking about, in some cases the actual rebar strengths, if you go to that length to prove them, instead of using the specified mark-ups that the code gives you.

I then wanted to just set the scene, if you like,

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because the Original Inquiry focused on areas B and C, comprising hundreds of metres in length of heavy civil engineering work. Those have now, in essence, been given a complete clean bill of health in spite of all the investigation, the testing and the assessment which has been carried out, thus corroborating the Commission's interim report, if you like. Despite this, the station extension and the railway lines are still not open to the public, and we are sitting here today discussing the safety and fitness for purpose of a small outstanding section of the works, a very small section of the work by comparison; that is, namely, areas A, SAT and HHS.

So the two principal factors that this Inquiry is hinging on is the fact that MTR's assessments -- and when I say "MTR" I'm obviously including their design engineers -- have totally disregarded any structural contribution from, number 1, the partially engaged coupler assemblies and, number 2, the shear link rebar. So those two components may as well not be there, and that's how the assessments have been carried out. In my opinion, hugely conservative and extremely brutal.

In addition, no opening-up at all has been carried out in the areas in question to verify and substantiate these allegations of defects. The EWL slab soffit in

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area A in particular was, despite the fact we were told it has been upfilled with mass concrete, it is still accessible if needs be. This is also despite the fact that sections of the works in question were constructed, as we've heard, at different times compared with areas B and C. We must also bear in mind that the type of construction in areas A, SAT and HHS is easier and much less complicated.

So inevitably I have to spend longer on this issue than the other four, as have the other experts. I have seen the threaded bar ends both with a chamfer and more recently without a chamfer, and the diagram shows the chamfers but it really doesn't matter much. Every threaded bar -- and I think this is an important point to make -- that I have measured, including those in BOSA's factory when I visited them, and the recent batch shown to us downstairs, measure at the very most 44 millimetres. Despite BOSA's published information, I have never yet seen a type A bar with 48 millimetres of thread. In fact, when you look at appendix B3 of the holistic report, it indicates that with a few exceptions, for example test 11 of the NSL slab might be an exception, the maximum threaded length is 44 millimetres. Otherwise, the enhanced PAUT results lack credibility and would be deemed to be inadequate

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and misleading.
In respect of test 11 of the NSL slab, it shows -- you can take my word for it, if you wish -- a length of 47.7 millimetres, but it should be remembered from the Original Inquiry that there was evidence of type B bars occasionally being cut and used in lieu of a type A bar.
I also suggest that if one looks closely at appendix B3 of the holistic report, the following assemblies appear to show the use of cut type B bars, and I'm referring to the EWL test numbers 27, 30, 33, 34, 37, 38, 42, 49, 51, 63, 64, 74, 75, 84 and 85. I may have missed a few in the reckoning.
It is highly improbable, in my opinion, therefore, that any type A/48 millimetre threaded rebar was used. Can I repeat that? It's improbable because I haven't seen any evidence that there is a type A rebar with 48 millimetres used in this particular project. I don't think it's even made.
The important point, however, to note -- and this came up at the end of Dr Glover's evidence or cross-examination -- the definition of "engagement". The testing labs are all using a consistent terminology in the context of the enhanced PAUT testing. So ten threads, as we've heard, equates to 44 millimetres engagement, and that is made up of the multiplication of

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ten threads, each at 4 millimetres, and it doesn't matter whether you take a 2 millimetre half-thread plus a 2 millimetre end chamfer or simply 4 millimetres at the end, the last 4 millimetres closest to the inside of the coupler, that is reckoned as being non-effective thread. So equally, if ten threads equates to 44 millimetres, nine threads equates to 40 millimetres, and so forth.
A lot has been said already about the tensile tests on partially engaged couplers. Six threads, as you've heard, will pass the basic strength requirement but not that of a ductile coupler. It's a moot issue I'll come back to later, but some of the tests show that six-thread engagements will not satisfy the higher 575 Newton per square millimetre test. It's not an issue. I have played safe and I have simply, by way of using almost like a sensitivity analysis, decided to use 7.5 millimetres as in the Original Inquiry, and these all fail in bar-break mode which is a requirement of the ductile coupler.
I then want to say a little bit about ductile grade couplers. They are only -- apart from the small issue of plasticity, they are only required in structures which experience cyclical load reversal. So, for example, a tower block, like you have here in Hong Kong,

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with variable wind load and direction, so one minute one side of the building is in tension; the next minute, that side of the building is in compression.
However, that is not the situation in these HUM structures, except of course for the D-walls.
In this context, I would like you to -- if you could pull up the reference on the screen, OU6/4139, just to give you a flavour for what Atkins are saying.
It should come up as section 16 on page 102.
MR PENNICOTT: That's right.
A. I'll just paraphrase this, if I can.
At 16.9.2, Atkins are explaining that the ductility requirements of the Hong Kong Code of Practice were derived for aboveground building structures.
Then if you drop down to 16.9.3, please, from there, and including 16.9.5, if you just scroll up a little bit -- okay -- I'm quoting here the behaviour of an underground structure such as HUH Station, subjected to seismic excitation, is different from that of an aboveground structure. The design of the slabs/diaphragm wall joints is governed by static load combinations and not due to seismic demand. The issue as to whether type 2 ductile couplers have been installed at the slab joint connection to the diaphragm walls is not of significant engineering concern, since

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the detailing rules applied for the station design were written for a different type of structure.
Thank you for that. If we could go back to the presentation.
Based on my 7.5 threads as almost like a sensitivity analysis, my acceptable criteria, my overall actual combined failure rate is only 6.9 per cent. But to allow for an approximate 95 per cent confidence level -- and I will frankly acknowledge I have absolutely no interest in statistics, I don't know anything about statistics, I try to stay away from them -- but I have operated my 6.9 per cent failure rate by approximately 35 per cent, which is consistent with the mark-up values of 31.6 per cent for the EWL and 34.4 per cent for the NSL slabs respectively, which MTR have used. If I do that, if I mark up my 6.9 by 35 per cent, I'm arriving at approximately a 10 per cent strength reduction factor, which to put in context compares with approximately 35 per cent applied by MTR.
A lot has been said about the PET, as we will refer to it, the permanent elongation test, and we've all acknowledged that to allow any sort of machine screw or a bolt or a stud to be screwed into a female coupler, there has to be tolerance, otherwise it just wouldn't fit.

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In the laboratory, it has been demonstrated by CEEK that when the tensile load is initially applied, so at very low load take-up, the slack is taken out or, as Dr Glover said, the threads bed in. Therefore, for a partially engaged coupler assembly in the laboratory, PET is not measuring irrecoverable elongation or stretch as the test is designed to do. It is instead primarily measuring the take-up of the slack, as I call it, in the threads.

In the test conducted -- and, Commissioner, you have stolen my thunder, I'm afraid.

COMMISSIONER HANSFORD: I do apologise.

A. -- the failures were equivalent in thickness to three or four sheets of normal writing paper.

COMMISSIONER HANSFORD: I must confess I had read that before I said that. So, Prof McQuillan, you can keep the credit for that.

A. Thank you, sir. I know that's jovial, but to put things in perspective, if a crack in the concrete at the top of the EWL slab was to actually form, it would only be a fraction of 1 millimetre in width. And I noticed when Dr Glover was talking you did take out your scale rule and you showed the chairman what that meant.

It would in any event be hidden by the track form concrete above it, so in my reckoning you have been told

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all structures contain cracks, this one is out of sight, out of mind, and it should be highlighted in any event that the filling and sealing of cracks, if deemed necessary, it isn't rocket science. It's straightforward and there are plenty of products that can be applied.

It has been agreed by all, however, that only fully engaged butt-to-butt couplers will pass the PET in laboratory conditions, and I stress "laboratory conditions". That is what Highways Department's acceptance criteria intend. So, in other words, to be code compliant, bars need to pass PET. There's no simple way around that.

It is highly significant, however, that CEEK have proved, with their series of tests, that if a coupler contains grit and the bar is only partially engaged but rotated to refusal, it will pass PET. That is because, as we've already described, the slack in the threads has already been taken out.

COMMISSIONER HANSFORD: Sorry, just to understand that point, can we go back a slide, please.

A. Yes.

COMMISSIONER HANSFORD: Just to understand that point, you are saying that PET can be passed, the PET test can be passed without it being butt-to-butt if there is grit in

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the thread?

A. Exactly. So all it takes is for something inside the coupler to cause the threads to lock or to bed in, and then providing it has the necessary tensile strength in the assembly, in other words a minimum of six threads, it will pass the test.

COMMISSIONER HANSFORD: So, therefore, the permanent elongation, ie the bedding-in, the permanent elongation is not prevented by it being butt-to-butt; it's prevented by there being something that locks it?

A. Yes. To put that in a slightly different way and rephrasing what I said earlier, the PET has -- the test results that have been given for partially engaged couplers are indicating that the measured movement is not so much the stretch or elongation. It's really the taking up of the slack.

COMMISSIONER HANSFORD: Yes.

A. I think that's a very important point to get across.

COMMISSIONER HANSFORD: Yes.

A. We'll come back to it in a moment as well.

So, bearing in mind that I've never seen a type A bar with 48 millimetre threads, I've produced this what I call a coupler engagement calculator. It just allows you to read off, very simply. This is repeated in paragraph 58 in my report.

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It assumes, first of all, two bars, each with 11 threads, and that is ten threads plus the non-effective end 4 millimetres. And I'm assuming first of all that these two bars are centred inside the 88 millimetre long coupler.

Highways Department's two-thread -- let's call it HyD for short -- HyD's two-thread allowance exposure, if you look down the fourth column from the left to where you see the "2" -- and I don't know, can I use this thing? -- there (indicating), so I'm pointing at the "2", you will see that it has a gap -- if you focus your eye on that fourth column from the left, on the 2 millimetre exposed threads, you will see, as you read across, that there is a gap in the middle of that coupler between the two bars of 12 millimetres. So just to make sure you're focused, if you go over to the left-hand column and read down to a length engaged of 38; scroll across to where it says two exposed threads; go further across and you will see there's a 12 millimetre gap in the centre of that coupler.

So it will fail the PET, but it has satisfied HyD's criteria.

Then equally, if you look at HyD's allowable minimum 37 millimetres engagement, so that is actually an effective thread of 8.25 millimetres, which is the

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top row, reading from the bottom, in red.
So, if you read across, it actually fails on the criteria because 2.25 threads are exposed, and so you have to move up a row, and the minimum engagement has to actually be 38 millimetres, which is 8.5 threads, to satisfy the other criterion of a maximum of two threads exposed.
However, as shown, this will also fail, in other words the 37/38 millimetres, will also fail the PET, because of the gap in the middle.
COMMISSIONER HANSFORD: And not only that. Presumably or clearly it's not butt-to-butt.
A. That's why there's a big gap in the middle, yes, 12 millimetres.
Sorry, let me go back. What about the scenario where one bar is fully screwed into the coupler which more or less simulates what we're talking about in this Inquiry, because of the D-walls? This is just one of the couplers retained by the Commission, and I think the original copy is in the room here if anybody wants to check. The photograph -- one end of the bar, at the left, is fully screwed in, hand-tight. And the measurement shows that there is only an available internal dimension of 44 millimetres.
So what about the hypothetical scenario of having

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only 40 millimetres inside the coupler which is represented by this mythical 48 millimetre bar if it was screwed in? I produced that little table to show the result. Even though I don't think it's possible for that to be done.
So the 38 millimetre -- that's the third row down from the top -- or 8.5 threads, then the one above that, the 39 millimetre with 8.75 thread engaged lengths, they also both fail PET because there's still a gap in the middle.
Mr Southward's evidence included photographs of this situation that I've just described. In other words, what I'm saying confirms the photographs that he had shown were one bar fully engaged to refusal and a continuation bar was screwed into a coupler so that two threads were showing. The coupler was then cut open and his measurements concur with my central gap; QED.
So we're asking ourselves the question and I think Dr Glover had a slide with the same title on it: is PET relevant? I put it to you that coupler assemblies could comply with HyD criteria and yet fail PET.
The HyD acceptance criteria therefore -- and I want to underline this; I've put it in bold -- sanction the use of partially engaged couplers, whether they like it or not. It therefore follows logically that partially

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engaged coupler assemblies which do not pass PET should not therefore be structurally disregarded, provided they achieve the full tensile capacity. That's what we're here discussing today, the fact that perfectly good partially engaged couplers have been discredited and discounted from the structural assessments.
PET is therefore not a relevant test, principally because by way of repetition and summary. In the lab, the PET is carried out in free air and unrestrained, whereas in situ, the coupler assembly will behave completely differently because it's encased in concrete, and any thread slack is taken up post-installation because of the self-weight of the bar. I'll show you a diagram of that later.
To repeat, there has been no evidence of any cracking even though the slabs have experienced most of the loading. The upper surface of the EWL is in an internal controlled environment so there is no risk of water ingress. This point has been rehearsed over and over again today. Rebar corrosion, even if -- there's no risk of water ingress and corrosion even if cracking did occur, but of course it won't.
And if cracking was to occur, for example at the bottom of the NSL slab which hasn't been mentioned because it can behave in the same way, upside down,

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corrosion cannot occur because, as you've already heard today, you need the three elements: you need oxygen, you need water and you need steel. Therefore, I put it to you that in these structures, durability and longevity are not compromised.
That's the little diagram I referred to. So when these guys are screwing in a 6 metre long starter bar to refusal and it's not fully engaged, if it's perfectly aligned and the weight is supported, you could actually feel the wiggle in the threads or the slack. Once these guys insert it and let it go, the end of that bar is going to try to sag, dip at the end, and in so doing, it's trying to pull out of a coupler, and in so doing, if you think about it, it's actually locking the threads.
So to me there is absolutely no issue with slackness of threads once these starter bars are engaged. The problem just dissipates.
Then we come to the question -- this is really the reason why of -- the entire station areas B and C have passed. The only area that has failed is the EWL slab in area A, and that is because of this issue of the doubly defective coupler assemblies.
I use the term to describe the couplers adjacent to the D-wall which have partially engaged bars on one or

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both sides, and I would just like to draw your attention to the fact that again, as with the Original Inquiry, it's simply the couplers in the top of this slab that are the ones in question, because they are the only ones in tension. The bottom rebar assembly is in compression.

In my opinion, what the holistic assessment has done is to double-count or should I say double-discount these double couplers, and this of course has the effect of approximately doubling the strength reduction factor, SRF. So instead of the normal average that they have computed of 35 per cent, say -- I know it differs slightly from the EWL to NSL -- instead of that average of 35, it's effectively been doubled to 68.3.

I think it's quite significant that Dr Glover's strength reduction factor -- and he is a statistician as well -- he has arrived at 23 per cent, just to put it into perspective, compared with his average of 12 per cent.

In such a situation -- I'm coming at this purely from an engineering perspective. I don't trust statistics. The quotation was made in the statistical enquiry, Mark Twain's famous quotation: "Lies, damned lies, and statistics". I'm a bit sceptical like that. So I'm just looking at this almost from an engineer's

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perspective sanity check, let's call it a sanity check, and I'm saying: how can this possibly be? I recognise that the probability of finding double couplers with partial engagement on both sides is high, but bear in mind that partially engaged couplers we are postulating are perfectly safe.

I cannot understand how the failure rate doubles because you are only ever going to achieve a failure, even if it was to occur, on one side of a coupler. So it's the concept of the weakest link in the chain. If you put a coupler like this into a lab test and you had -- in fact, the tests, I've already done it in the lab -- only one side of the assembly is ever going to fail. Two of them cannot fail exactly at the same moment in time.

So I'm putting it to you, and there's nothing hinging on this because I'm quite happy to take Dr Glover's 23 per cent failure rate because it shows the situation to be safe. I'm saying that based on my analysis, I would apply the same structural reduction rate, strength reduction factor, to area A as I would to the rest of the structure.

I'm moving on now to do a quick summary or synopsis of my review of the other reports. I think it's very important for the Commission just to get a flavour for

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what the other eminent consultants are saying. We're talking about the leading companies in Hong Kong here. We're talking about the best brains in the industry and their views on the matter.

So looking first at Atkins stage 3, partially engaged coupler assemblies are ignored. This is unrealistic, and -- I've used the word "hugely" previously -- it's hugely conservative, because the contribution of partially engaged couplers has been completely ignored. Yet Atkins might say apologetically, to their credit, make the point that partially engaged couplers do contribute to structural capacity. They say that a minimum of six threads is okay for ULS condition. That's failure. They say that a minimum of seven threads will satisfy the serviceability limit state criteria.

COMMISSIONER HANSFORD: It's the other way around on the slide.

A. It's the other way around. Yes. I've got it wrong on the slide, actually. It should be six threads for ULS; it should be seven threads for SLS. My apologies for that. They agree the non-compliant PET results are because slack has been taken up in the threads.

CHAIRMAN: Sorry, just to assist me, do Atkins say why they have ignored partially engaged coupler assemblies?

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A. Because they were simply acting on the recommendations of the holistic report in their stage 3 assessment.

CHAIRMAN: Okay. Thank you. Yes, that's right.

A. So AECOM, they used the original design parameters, and they looked initially at the effect of partially engaged couplers with a minimum of seven-thread engagement. Their strength reduction factor was 5.1 per cent. They then also did a kind of sensitivity analysis and played safe, and this time they used a minimum engagement length of 37 millimetres. Their conclusions were that the structures were still safe.

They then carried out a sensitivity analysis themselves, and this time they used not the original design parameters but the updated ones, and they used the strength reduction factors that MTR had imposed on them via the holistic report, and they found the structures were still safe.

However, then came this issue of the double coupler in area A, and what I'm thinking is that when they said the structures were safe, that they hadn't got wind of that at that particular stage.

So my conclusion to this whole issue of the coupler issue is that the structures are safe and fit for purpose.

Anticipating your question at the end,

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Mr Commissioner, stealing your thunder in reserve.
COMMISSIONER HANSFORD: Touche!
A. It's interesting that in BOSA's technical literature they have this recommendation. It's really a product quality control check. I don't think they are intending this to be used every day in life by workers on the site. But based on this, a possible recommendation might be -- and this is now almost in addition to what Dr Lau and Dr Glover have proposed -- I'm suggesting that before every starter bar is installed -- and I think Dr Glover has covered the first one -- the inside of a coupler dimension is measured. I'm suggesting that it's quite easy to record that with a photograph and that can be whizzed back to base.
I made the point in the Original Inquiry that even when you're working in car engines with a torque wrench, it's recommended practice to lubricate the threads, and I'm suggesting the inside of the coupler is lightly sprayed with WD40 if you have that in Hong Kong or some similar light oil, and I'm suggesting that the QSP and other relevant documentation is amended accordingly.
However, that comes at a cost, and before anybody would rush off and make this a firm recommendation, it's obviously important to consult with all the relevant stakeholders, but it kind of tightens up the whole

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procedure. I'm not even saying this will ensure you get butt-to-butt. I haven't even considered that as a necessity or a requirement. I'm just saying on site, with these heavy bars, the best way to try to get it in as tight as you can is to take these precautions.
We come to the next issue which is the shear link reinforcement. Again, I don't wish to appear patronising here but there may be someone in the room from a non-technical background who's dying to ask the question: what is a shear failure? Probably everybody.
CHAIRMAN: (Raising hand) I realise I have been given instruction on this, but as a non-technical person the instruction dissipates very quickly, and so something that you can give now, Professor, which I can then write down and goes into the record, would be of assistance.
A. Thank you, Chairman. What I've done in that diagram is show you a simple picture. It could be a concrete slab or it could be a concrete beam, supported, as we have in this situation, by a wall at each side. If that beam or slab has no reinforcement in it, what tends to happen is that there are two potential shear plane failures, and I'm pointing to that line (indicating) and I'm pointing to that one (indicating), and it tends to be at an angle of 45 degrees. If the load applied exceeds the shear capacity of that non-reinforced section, the centre of

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the slab simply drops, and it happens suddenly and it happens as a brittle failure.
So how do you prevent that from happening, and you've heard all this talk about shear links, maybe someone wants to ask: why do you need shear links as well? This is the answer: to prevent the shear failure occurring at any one of those potential failure planes, you stitch across it, is probably the simplest word, using these vertical bars. It's a bit like mending a tear on a piece of cloth; the thread is sort of stitching the two sides together. And when longitudinal reinforcement is present, to provide the bending strength, it partially also contributes to the shear capacity. That's maybe a point to come back to at the SAT.
So let's look at the alleged -- in terms of the alleged defects, we're talking now about area A. No opening-up or minimal opening-up was done in area A to substantiate these alleged defects. The thinner slab in area A -- and we are talking about a 1 metre depth this time -- is more conducive to placing the shear links, so a much easier task. Dr Glover was telling you and illustrating, as he stood, how difficult it would be to retro-install those 3 metre long shear links from the top of the EWL slab. It becomes quite complicated.

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COMMISSIONER HANSFORD: Why do you say retro-install?
A. Because the reinforcement was built from bottom up and it's more likely, he explained, that they were dropping them down through at the end.
COMMISSIONER HANSFORD: Yes.
A. In spite of all of that, in spite of no investigation and in spite of the more conducive conditions in area A, the strength reduction factor is, to put it in stark, blunt terms, is 100 per cent. In other words, zero contribution from the as-placed shear links.
Again, I'm just letting the Commissioner and the Chairman have a flavour for what the other experts or non-experts in this case have had to say about the matter. Atkins disregarded the shear links but they admit it's very conservative. And although they use -- they go ahead and they base their assessment on the actual design concrete strength. Notwithstanding they admit that the actual concrete strength as placed and as hardened could be used to reduce shear enhancement work, and they also admit that partial contribution of -- I use "non-compliant" in the sense that they don't exactly meet the requirements of the code in terms of the detailing -- the partial contribution of those shear links could be included in the shear capacity assessment.

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Then I look at the AECOM --
CHAIRMAN: Sorry, did they explain -- I've been through the report but I can't remember whether they explain why they disregarded all shear links.
A. For the same reason as I gave in my previous answer: the outcome of the holistic report. So they were simply implementing the recommendations of the holistic report, which said, "Disregard all shear link rebar."
COMMISSIONER HANSFORD: That was their brief?
A. That was their brief. I'm not criticising Atkins for what they did, by the way. I'm just saying they were acting on instructions.
CHAIRMAN: Would this be valid, in other words, to say, "Well, we've got photographs of certain shear links, we can't believe that everybody forgot and they all had a bad day at the office and even the inspectors forgot over a period of time, but because it's uncertain, let's just work on the basis they are not there at all and see what comes out of it?"
A. That's exactly what has happened, Chairman, yes. That's why I describe it as hugely conservative.
CHAIRMAN: Yes.
A. So AECOM, in their assessment report, although they use the design concrete strength, they also advocate using actual strength. So I'm afraid Dr Lau is very much out

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on a limb here when he says you cannot use the actual strength in the assessment.
Then Atkins carried out what's called a sensitivity study. There were these 18 openings in the -- let me just go back to my notes here. 18 locations on the EWL slab soffit but none in area A, although I wrote that when I only had the discovery to work on. I think there is evidence that there may be one or two openings in area A now.
Of those 18 openings, only four of them required shear links, and even then it was found that the original design provision of shear links, had you assumed it was valid, had you assumed that the original shear was contributing, then they said that there was no problem.
Then we come to Arup and I'm just really -- Dr Glover has given you most of it but I'm just summarising it, to put it in context. Arup carried out an FEA without incorporating mitigating effects such as arch action, and they used the specified design concrete strengths. They found that only a few locations in area A and at one location at an air duct in area B required any shear links at all, and then only nominal -- by nominal we mean the code minimum requirements.

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So non-compliant links are only a potential issue when it comes to these very minimal locations. Then Arup went on to suggest some mitigation features or measures to compensate for the shear link non-compliance, including using the actual concrete strengths, and Dr Glover has explained that very adequately today, and in light of the new core test evidence in the EWL slab, that becomes a very valid consideration.
Arup also advocated using arch action, but they didn't.
They then carried out a sensitivity analysis on concrete strengths with the shear links disregarded. They found that, for example, a modest increase in strength from 40 megapascals to 45 megapascals resulted in a 10 per cent increase in shear capacity, and so it probably increases linearly up to the 60 megapascals that we've been hearing about.
Still, with shear links disregarded, Arup then carried out a safety check, and we've heard about this yesterday, using, as is allowed, by the way, using lower and more realistic load factors. I have written that wrongly. Arup then concluded that the structures had more than adequate shear capacity. QED.
Then we come to an Australian consultant that were

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hired by Leighton, EIC. Theirs was probably -- perhaps the most extensive and sophisticated approach to looking at this, the most thorough in terms of their review of the shear capacities, and they used what's called a hierarchical mitigation approach to consider factors such as shear enhancement, axial compression, actual concrete strength, reduced partial safety factors, reduced anchorage length, partial engagement of shear links because of the little non-compliances, and also what we call modified compression field theory, and I don't even pretend to have read what that is.
I don't need to elaborate on all of those, but what they are basically saying is that they start from the top down and they look at the effect of one of these mitigating factors, and it will partially if not totally eliminate the shear under-capacity. Okay? If it doesn't work at that stage, they go to the next one on the list and it will have some contribution. So they work their way down through and they find that all of these taken together will more than compensate for any perceived lack of shear deficiency in the structure.
So a lot has been talked about the strength of concrete, should it be as designed, should it be as actual?
Even if you don't take it to its full extent, you

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only have to take a small bit of that as a contributor to reducing the shear under-capacity.

So I'm not going to elaborate on all of these but just if you take, for example, axial compression -- and they don't even mention arching action -- I've used this little illustration in my report. I would show this in a university class, to let the students get a feel for it. If you have ten standard bricks sitting on a plank and you try to casually lift them, you can't lift them because in that situation you have nine vertical shear planes. There's a zone of weakness at the interface of each brick. So how do you lift them? You simply clamp your hands together on the ten bricks, squeeze as tight as you can and you will find you can actually lift them off the plank.

So that illustrates the concept of axial compression, and that is also necessary to a certain extent in arching action, because you need a compressive force at either side.

So then I'm giving you what EIC say about -- their conclusions. They use the original design parameters, but the actual concrete strengths, and they identified only two locations, at SP37 and SP47, which in theory required shear capacity enhancement. That's based on the fact that shear links have been disregarded.

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They also took partial shear link engagement into account, so in other words if the shear link doesn't comply fully with the code, they are saying it still has a contribution to make. It mightn't be 100 per cent, it might be like 70 per cent, but they consider it valid based on work that a Prof Foster, who is a world-renowned expert on shear, has carried out at one of the Australian universities, they are quite happy to take the partial contribution of the shear links into account. When they did that, they identified only one location, at SP37, and then when they took Dr Foster's modified compression field theory into account, they reckoned the structures were safe, no remedial work, no enhancement at all, was necessary.

They do make the point that if they then apply the updated design parameters, the situation will become even safer.

So my conclusion on issue 2 is there is more than adequate shear capacity, as demonstrated by all the brains who have looked at this, and the structures are safe and fit for purpose. QED.

We come on to the thorny issue of the horizontal construction joint in the D-wall. Prof Au, in his evidence to the Original Inquiry, recommended the insertion of these retro-installed vertical dowel bars,

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and it was very much to prevent slippage across what he claimed was a highly stressed construction joint. In other words, he was suggesting the dowel bars as a structural strengthening mechanism.

No one is challenging the fact now that the stresses in the CJ are extremely low. There is no evidence of slippage. That has been confirmed by the core tests. Failure of the D-wall, if it ever happened, and it won't, at the sort of load levels we are operating at, Dr Glover has explained that it would occur in the D-wall, at the underside of the EWL slab connection.

The connections are therefore safe, and the issue with the CJ is purely one, as you've heard, of defective workmanship, identified in just a few locations. The retro-installation of the vertical dowel bars is therefore not premised on structural integrity considerations or structural safety.

The experts' memorandum of agreement December last, states that Dr Glover, Mr Southward and myself, we've agreed that although there is no impact on structural performance or safety, that no rectification is therefore required. It might be prudent, in order to allay public safety concerns, to remediate the construction joint in those few locations where substandard workmanship was found defective, but only in

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those locations.

The way to remediate the CJ is to pressure-inject grout. If you want to fill any perceived voids, if you want to be seen to be doing something, just fill the crack with grout, as you would with any other crack. So the retro-fitting of the dowel bars, in my opinion, is completely unjustified for the reasons they have explained it necessary. I fail to understand why they have been agreed.

The retro-installation of dowel bars requires, as we have seen, the coring of deep holes, down into the D-wall, with a possibility of cutting shear reinforcement, and by comparison pressure-grouting requires much smaller diameter holes, so less chance of hitting steel.

I agree with Dr Lau, one of the few agreements, that a concrete drill bit and a hammer action, a percussive drill, you will know when you've hit the steel, because we've all done it at home.

COMMISSIONER HANSFORD: Sorry, you said there's a risk of cutting shear reinforcement?

A. Yes.

COMMISSIONER HANSFORD: Is that reinforcement only there to --

A. It's the lengths in the D-wall we're talking about.

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COMMISSIONER HANSFORD: Right.
A. Because the main reinforcement is in the form of vertical bars, so you would be drilling parallel to those. The only horizontal steel you're going to encounter is the actual shear links in the D-wall.
COMMISSIONER HANSFORD: Is there a risk of hitting the vertical bars?
A. No, because you are drilling parallel to them.
COMMISSIONER HANSFORD: How do you know?
A. Because, if I can use my hands, if that's the D-wall (demonstrating), the main bars are inside the extremity slightly.
COMMISSIONER HANSFORD: Yes, I understand.
A. So you are drilling down parallel.
COMMISSIONER HANSFORD: Yes, I understand.
A. But what I do not agree with Dr Lau on is a that coring machine will bounce off steel and you will know the difference; okay?
That photograph is one of a number of a series that I personally watched being extracted recently from a concrete wall and I was doing it --
COMMISSIONER HANSFORD: This is not Hung Hom?
A. This could be anything.
COMMISSIONER HANSFORD: And this wasn't Hung Hom?
A. No, this is back in the UK.

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COMMISSIONER HANSFORD: Okay.
A. So I was witnessing these series of cores being extracted on behalf of the group of experts who were involved in the case. I was there to witness it. It's typical of any project. What I've pointed out there, with the two little Xs, are two layers of rebar, one longitudinal, one transverse, and the operator of the core machine has no idea at all whether the diamond-tipped coring tool was cutting through hard granite aggregate or the rebar. The torque exerted by the machine was such that it never even slowed down when cutting through the two layers of steel. So I perceive no actual difference. I think that illustrates the point that some of us are making that there is the risk, with coring as opposed to drilling, of actually cutting steel reinforcement.
So my conclusion for issue number 3, and you can see I'm kind of speeding up as we get through this, the structures are safe and fit for purpose, the issue is very much one of workmanship, as has been stressed. No intervention is required, and I have in my report stressed that you're best to let sleeping dogs lie, nothing needs to be done and it's risky doing anything.
That brings us on to the issue in COI 2 of the HHS coupler connections. Again, pardon the typo on the

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screen. First of all, the function of the stabling sidings. You've already heard some of this. It's a non-public area. The trains that are being stabled are travelling empty and at very low speed as they approach. The HHS walls are intended to contain a train in the event of derailment collision and, where relevant, to protect the adjacent podium columns.
Again, there are much more amenable working conditions, lighter rebar, greater visibility for inspection. That means that there was much less chance of getting it wrong. And yet without any opening-up to prove the defects, a global 35 per cent SRF was applied. There are numerous record photographs, when these walls were being constructed, of good-quality coupler connections at the HHS wall kicker level.
I've shown just one of a number of examples, and even though it lacks a little bit of definition on the screen, you can see that those couplers appear perfectly sound, easily inspected. The weight of the rebar, because it's a smaller rebar, is a lot easier to handle The bars are being dropped in vertically so there are no alignment problems as with horizontal bars. This had a much better chance of getting it right, and yet the contribution of those have been reduced by 35 per cent.
Again -- and I'm going through this process for

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every issue -- I'm looking at what the other experts and other consultants have to say about these issues. So AECOM, as instructed by MTR, have applied the 35 per cent SRF. They've used a very basic, conservative analysis method that you've heard about, which is the cantilever. Everything passed except for the panels adjacent to the vertical movement joints where they were found to be under-capacity.
You've already heard from Dr Glover how they carried out an FEA. What they did, instead of considering the load spreading down at the conventional 45-degree angle through the walls, they played tunes with that. They looked at 30-degree angles, et cetera. However, they found that the maximum utilisation, even with the joint incorporated, was 92 per cent, and they found the trough wall satisfactory, notwithstanding, as you've heard from Dr Glover, Arup suggest a lot of mitigation factors that could be taken into account which would significantly enhance the reserve capacity.
He's mentioned the reduction in live load, because there's no passengers running on these trains. The MTR criteria ask you to take the collision angle of the train at right angles to the wall, which doesn't make a lot of sense when the train is actually moving. It's going to be more of an inclined impact angle. He has

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described the energy absorbed by the train itself, and like me he considers that the earth fill, even though it may not be compacted, still has a very significant energy-absorbing contribution to make.

Can I just say, as another incidental point, the soil -- and I've shown the little concrete paving bit at the top as non-structural. It really is just to cap the soil and provide a platform for personnel to walk on. If the soil was going to be dug out, as Dr Lau has told us, I'm afraid they have to close down the operation in those sidings because it would be unsafe for men to walk in that area. There would be no ability to maintain the trains or anything.

COMMISSIONER HANSFORD: And also to remove the soil presumably you would have to take the capping concrete off?

A. You would, yes.

Let's just go on. You have heard from Mr Southward already. He used what I consider, assuming the soil was removed, a more relevant ultimate limit state analysis, the yield line analysis, and as he has described, it is the most applicable method in terms of a cantilever upstand because it's taken from the analogous bridge parapet work that has been done in America.

COMMISSIONER HANSFORD: Just so we can understand that -- so

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that is being used for demonstrating that bridge parapets, when struck by a vehicle, are safe?

A. It's actually the code that is used, as I understand. I think Mr Southward gave that evidence. So that's how you design a bridge parapet for vehicle impact loading.

COMMISSIONER HANSFORD: That's really what I meant.

A. Yes. So if the soil is removed -- I agree with Dr Glover if the soil is there, the analysis is inappropriate, it wouldn't fail like that -- but if the soil isn't there, in that hypothetical situation, then I agree that Mr Southward's analysis is perfectly valid.

I think the point that has been lost with all of this, and I will repeat it -- he can probably do it much better -- he hasn't in any way tried to argue against the 35 per cent strength reduction factor. He's accepted that, if you like, as a hypothetical situation.

He has also taken into account something that hasn't yet been mentioned and that is the markdown in tensile strength of the rebar, and this is all linked with the perceived fact that not enough rebar had been tested on arrival at site.

COMMISSIONER HANSFORD: Yes.

A. So that was another imposition by the verification report. Mr Southward has taken both of those into account and said, "Let's just accept them and see what

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happens", and he has proved that the trough walls have more than adequate capacity. He then kind of does a sensitivity analysis and his conclusion is in fact they could cope with a 58 per cent strength reduction factor, if I'm quoting him correctly.

So in conclusion, these structures are safe and fit for purpose. A lot has been said about the need to protect the podium columns. I'm quite sure that a designer like AECOM would have fully taken this into account when they were designing these trough walls, the accidental impact on the columns. So my premise is that, on the basis of everything I've seen, the walls are safe and therefore there isn't any danger to these podium columns whatsoever.

Which takes me nicely on to the final theme, and that is the shear link reinforcement in the NSL slab in the South Approach Tunnel. Again, the shear links have been completely disregarded in the assessment of shear capacity, so a 100 per cent strength reduction factor applies. We make the point again, this is despite the lighter type of construction, the more amenable working conditions and the ease of installing the shear links.

Again I'm letting you see -- I know it's getting a bit boring but these are the other non-expert and expert reports. It's useful to see how many people are

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on the same wavelength when it comes to these issues.

So Atkins, in their revised structural assessment, they admit to a conservative method of analysis, and even with a 35 per cent strength reduction factor in the couplers they concluded that both strength and shear capacities were adequate. But I'm thinking, I'm quite sure in my own mind, that was premised on them taking into account the full contribution of the shear links at that stage, when they carried out their assessment. They concluded that the SAT NSL was okay for shear before that markdown was applied.

COMMISSIONER HANSFORD: What leads you to that conclusion, that they -- sorry, if we can go back -- assume premised on the full contribution of shear link.

A. So what they were doing at the top, because they were looking at the shear contribution of the main bending steel, if I could call it that --

COMMISSIONER HANSFORD: Yes.

A. -- they applied the 35 per cent reduction as for the rest of the structures, but at that stage they were still assuming that the shear links were fully contributing, and this is basically just setting the scene, if you like, to show that the thing was perfectly safe, on that premise.

They obviously then got an instruction to say,

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"Disregard the shear links", which brings me on to the next slide.
COMMISSIONER HANSFORD: Okay.
A. So they then have to submit to the Buildings Department for approval, and in this case they apply the tensile strength markdown for the rebar -- that's a fairly minor issue -- but the main point is they disregard all the shear reinforcement. They use the actual concrete strength, and they find that there is one potential shear failure zone, and this is in the base slab which happens to be over 2 metres thick.
I contend that this mode of failure cannot occur, and Dr Glover showed you the diagram showing the difference between a basement at low depth in Hong Kong and the box tunnel sunk a way down into the completely decomposed granite layer, that soil is already over-consolidated. Don't forget that, as he explained and I concur with him, to build these structures in the first place, the groundwater table was reduced to a level of minus whatever it was, 16-18 metres. So the ground has already been subjected to that.
As both Mr Southward and Dr Glover have explained, you've got to look at this in three-dimensional mode. So what you have is a wedge of already consolidated, compressed soil, overlying bedrock, contained by D-walls

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on both sides. There isn't a pup's chance that if you were able even to excavate down to that level, you would ever find a gap below it.
What I'm saying is that the punching shear mode that has been predicted by Dr Lau -- sorry, that he concurs with but which was predicted by Atkins, it simply cannot occur because that layer of decomposed granite acts almost like concrete, if you like. It prevents the punching shear failure.
EIC again used their hierarchical approach to shear reassessment, to look at the shear capacities, and unequivocally they concluded that the structures have adequate capacity.
I'm not quite sure how SYW came into the equation; there were another set of consultants.
COMMISSIONER HANSFORD: Who are they?
A. The full name is -- somebody help me.
MR KHAW: Siu Yin Wai.
A. I have abbreviated as "SYW". They were also engaged by MTR to carry out a slightly different function. I think their main function was to look at all the records and piece together all the information, but they were asked to do some other work.
COMMISSIONER HANSFORD: Yes, I recall.
A. So their report, of course, on the basis of what I've

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said, contains a lot of record photographs taken during construction, and shear links are clearly seen in the completed rebar mats. I haven't replicated them here. You can find them in my COI 2 report, if you wish to look at them. There are shear links seen in the completed rebar maps in the NSL roof slab and also in the mezzanine slab. I wasn't able to find any for the base slab, but that doesn't mean that those photographs don't exist; okay?
And on the basis of that, SYW concluded the structures are also safe.
Then we have Mr Southward's report. He correctly makes point that in localised areas where shear reinforcement is required, and it is only a few areas, it only needs to be nominal. That means it only needs to be the minimum specified by the Hong Kong Code of Practice.
He makes the point very sensibly that because the links actually provided have a greater cross-sectional area than the minimum required, then the shear capacity is provided and it actually is code compliant, even though that's an issue we shouldn't be considering.
So my conclusions -- and we are near the end, you will be pleased to know -- on issue 5: Atkins' shear assessment -- I use the word again -- was hugely

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conservative because no investigation was carried out to confirm the shear link defects; the shear link contribution has been totally disregarded; the design concrete strength has been used; the modelling and analysis was too simplistic and did not properly represent the SAT structural behaviour.
Just for the record, again, I want to say I'm not angling at Atkins, I'm not criticising Atkins. They were obviously under instruction.
There is adequate shear capacity and the structures are both safe and fit for purpose.
So my summary opinion on COI 1 and COI 2. First of all, my summary opinion expressed in my first report in respect of areas B and C remains unchanged. Those particular structures are both safe and fit for purpose as-constructed. That includes, of course, the issue of the CJ.
Having reviewed areas A, HHS and SAT -- and bear in mind that even though NAT was assessed it was found satisfactory -- I am satisfied, without any doubt, that the structures overall are safe and fit for purpose as-is; there is no reason why the station should not be open to the public; I am aware the public have been told that remedial works were necessary, they will be expecting such, so to allay public concern I recommend,

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as I did in the original hearing, that long-term monitoring of structural performance be carried out.

I have kept that suitably vague. I am on the same wavelength as Dr Glover, as you've heard, visual inspection probably is satisfactory, so I have kept it loose.

COMMISSIONER HANSFORD: So you are not suggesting here the form of monitoring; you are saying long-term monitoring?

A. Correct. So that should be carried out except for the trough walls, because there's no need to monitor them. They should never have to come into operation unless there's a train derailment.

And we are suggesting that long-term monitoring, even though no significant results are to be expected, and we stressed that at the original hearing.

I've gone one step further because we know that enhancement works are already being carried out, suitable measures are already being implemented. I know it's an area we are not asked to stray into, if the structures are safe, but I'm suggesting that if that work is being implemented, there's no need for even monitoring, apart from maybe the odd casual visual inspection, because the structures are then going to be much safer than we are saying they are at present.

Thank you for your patience. That concludes my

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rather lengthy but necessary summary evidence.

COMMISSIONER HANSFORD: Just on your final point there, I recall in oral evidence way back, about a year ago, the reference was made to belt and braces.

A. Yes.

COMMISSIONER HANSFORD: What I think you are saying in your final bullet there is now with not only the belt and braces but also the piece of string added, you see no need for any monitoring?

A. And probably adhesive tape as well.

COMMISSIONER HANSFORD: Thank you.

CHAIRMAN: Good. Excellent. That's actually fairly good timing, it's heading for 4.30 and you have finished, so we will adjourn until tomorrow.

MR PENNICOTT: Yes, sir.

CHAIRMAN: 10 am?

MR PENNICOTT: Yes, sir.

CHAIRMAN: And we are satisfied we will finish tomorrow?

MR PENNICOTT: Very satisfied, yes.

CHAIRMAN: All right. Good. The only reason I mention --

MR PENNICOTT: Subject to Mr Chow.

CHAIRMAN: Ah, yes.

MR BOULDING: Sir, can I just hopefully answer a query? I think there was a query as to quite who SYW were.

CHAIRMAN: Yes.

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MR BOULDING: They are Siu Yin Wai Associates Ltd, apologies for the pronunciation, and they are referred to in the verification report which is BB9960 in the bundle, and on that page, paragraph 1.6(a) tells you exactly what they did. I hope that's helpful.

CHAIRMAN: That's excellent. Thank you very much indeed. Good.

I will ask the Secretariat to inform those who do some of the backup work and who get paid I think on a daily basis that they won't be needed on Friday; all right?

MR PENNICOTT: Yes.

CHAIRMAN: Which will mean that if by 4.55 tomorrow evening we are still in full flow, I'll become agitated; all right? Thank you very much.

MR PENNICOTT: And I'll become liable.

CHAIRMAN: Probably. Thank you very much. Tomorrow, 10 am.

(4.26 pm)

(The hearing adjourned until 10.00 am the following day)

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