Tuesday, 15 January 2019

(10.04 am)

PROF AU TAT KWONG, FRANCIS (on former oath)

Cross-examination by MR BOULDING

MR BOULDING: Good morning, sir. Good morning, Professor. Good morning, Prof Au.

A. Good morning.

Q. I'm acting for MTR and notwithstanding the fact that many of the matters I wanted to discuss with you have been covered by my learned friends already, there are one or two remains matters that I'd like to have a little discussion with you about.

A. Right.

Q. First of all, I'd like to look at your report, please, and if you would be kind enough to go to ER1, tab 7, at page 3, we see there, do we not, the first page of your opinion?

A. Yes.

Q. And in 1.2, you are dealing with a topic of "Sampling method as stated in holistic proposal"; correct?

A. Yes.

Q. Then you say in 1.2.3:

"When non-compliant cases are discovered during the investigation, it is necessary to further assess the effects on the strength and other properties ..."

And so on, and so forth.
A. Yes.
Q. When you are referring to non-compliant cases, I take it that you are referring to the extent of the rebar engagement into the couplers?
A. Correct.
Q. That's on the basis of the government's pass of 37 millimetres or more; correct?
A. Correct.
Q. That has been referred to me as being a quality requirement or a quality standard. You would go along with that description, would you?
A. I believe so.
Q. And this quality requirement or quality standard is not, is it, a recognised measure of what the strength of the rebar-coupler connection is; that's correct, isn't it?
A. Well, I think when we are talking about the acceptance of the coupler, we should -- well, we should require the coupler assembly to satisfy the requirement prescribed by BOSA, the manufacturer of the coupler.
Q. Yes, and that's what we have been talking about which is the quality, the quality requirement, or the quality standard, the 37 millimetres; correct?
A. Well, the 37 millimetres has taken into account certain tolerance of the test. Yes, I think that has been the

standard that the sampling has been carried out so far.
Q. Exactly. But what I'm suggesting to you is that that quality standard is not, is it, a recognised measure of what the strength or the structural integrity of the coupler connection is; that's right, isn't it?
A. Well, actually the requirement sort of implied by the BOSA specification is even more than that; okay? I think 40 millimetres engagement.
Q. We're talking about 37 millimetres, aren't we? That's what government has imposed upon --
A. Yes. So that is used in the sampling approach. But if you refer to the BOSA requirement, they have been talking about 40, ten threads.
Q. But as you say in your paragraph 1.2.3, when you've got a non-compliant case, it's necessary to further assess the effects on strength; correct?
A. Correct, and other things as well.
Q. Right. Staying with strength, we will see, will we not, that even at 60 per cent engagement of the rebar into the coupler, the full strength of the rebar is established; that's correct, isn't it?
A. Just in respect of strength.
Q. And that means, I suggest, that when that strength is achieved, that means that the assembly, the coupler assembly, is safe; that's correct, isn't it?

A. I think we have to be careful on this point, because so far the test results of the partially engaged couplers is not sufficient, just based on one sample. Then apparently the strength of the reinforcing bars used is a bit unsure, because -- well, I just came to know of that yesterday, that on the test report 460 megapascals hasn't been stated, if I'm correct.
Q. No, but they used 500 megapascals.
A. I'm not sure, but even 500 hasn't been stated over there.
Q. Let me put this to you: if they used 500 megapascals instead of 460, you would get an even better result in terms of strength by using the 460 megapascals, wouldn't you?
A. Yes.
Q. Thank you. Now, let's have a look at the BOSA documentation. I wonder if you could go to H44527.1.
I trust that you've seen this table before, Prof Au?
A. Yes, yes.
Q. We can see that it's produced by BOSA, the manufacturer of the coupler; that's correct, isn't it?
A. Yes, correct.
Q. And headed, "Thread strength calculation table"; do you see that?
A. Sorry?

Q. And if we just look at the "Remarks" first --
A. Yes.
Q. -- "1. The above calculation is based on the assumption that the threads are complete with full integrity. There will be deviation in the calculated number of threads if the actual threads are not complete with full integrity due to the quality of the steel bar quality at the threaded ends.
2. The above design data is based on specified strength of material used.
3. Factor of safety calculated above is based on specified tensile strength and not yield strength of material used.
4. Conclusion: For complete threads with full integrity, the number of threads that is required to achieve the specified tensile strength is six."
Do you see that?
A. I can see that.
Q. That is clear, is it not, BOSA's conclusion, the manufacturer of the coupler, BOSA's conclusion, based upon this table: six threads gives the specified tensile strength; correct?
A. Now, the --
Q. Well, is that correct? Please answer my question before you go off on a frolic of your own.

A. It's too fast to jump to a conclusion. Actually, you can look at -- right, the second-last column, that shows the steel bar specified tensile strength.
Q. You're talking about -- that's the stress, that's the column --
A. That's the stress.
Q. That's the stress?
A. That's the stress. Well, which column are you talking about, please?
Q. I thought that was the column you were talking about.
A. Yes.
Q. But I would like you to look down the number of threads, and if you look down the number of threads, you get six; do you see that?
A. Yes.
Q. Then you've got the pitch 4 millimetres below thread?
A. Yes.
Q. Then we look across and then there's the thread effective diameter, the shear strength, and then you've got the thread strength?
A. Yes.
Q. So at six threads, it's right, is it not, that you get a load of 755.87 kilonewtons?
A. Now, I believe --
Q. Is that right?

A. Now, that column is based on calculation. That is not based on test.
Q. Well, it's based on calculation by the manufacturer.
A. It's based on calculation.
Q. With that figure of 755.87 kilonewtons --
A. Yes.
Q. -- you then divide that, do you not, by the area of the rebar to get the stress; correct?
A. Yes.
Q. And we can see that based on the manufacturer's calculation, that is 601.5 megapascals?
A. Yes.
Q. Giving a factor of safety of 1.14?
A. Yes.
Q. Then we can see what their conclusion is?
A. Now, that is also based on calculation.
Q. Okay. Calculations by the manufacturer?
A. Yes, calculation by the manufacturer, yet to be substantiated by testing.
Q. But at the moment you are not in a position to tell the Commissioners that there's any doubt about these calculations, ie they're wrong or they're misconceived, anything like that, are you?
A. Now, actually it is more complicated than that. As far as I can remember, the column showing the threaded

strength is based on proportion, just multiplying, let's say, the number for one thread by the number of threads. Let's say the first one is 125.98 kilonewtons. The second one is 251.96. Just multiplying the first one by 2, and so on. But the actual behaviour of a threaded bar inside coupler is more complicated than that, because when the bar is loaded, not all of the threads are equally stressed. The threads closer to the outside would be more highly stressed. So this is just based on simplified assumption.

Now, if we do test to verify, I'm sure that there would be a bit deviation, and -- now, just imagine, if you have a very, very long coupler, okay, and your engaged length increases from zero to 100 to let's say 10 metres or whatever, then -- now, I don't think it is correct to assume that the strength provided by the threads of the coupler is proportional to the engagement, because the load carried by the threads is not uniform.

Q. Prof Au, let me ask you this: you haven't done any calculations yourself at the moment, have you, to show that what BOSA are calculating here is incorrect; that's right, isn't it?

A. Now, for this --

Q. You haven't done any calculations yourself to show that

what BOSA have done here is incorrect?
A. I have read a paper recently on the distribution of stress of threaded rods inside a coupler or something like that. I mentioned that yesterday. Then somehow the results, based on a finite element software called Abaqus has shown that actually the distribution of stresses in a threaded rod inside a coupler or a nut is not uniformly distributed. So that should apply to this case.
CHAIRMAN: Sorry, Professor, just so I can understand -- putting it bluntly then, if I'm an ordinary contractor and I accept what's here from the manufacturer at face value, what is there may potentially be misleading?
A. Yes.
CHAIRMAN: So BOSA's own documentation may be potentially misleading and affect safety issues?
A. Yes.
CHAIRMAN: All right.
MR BOULDING: But anyway, you've read a paper --
A. Yes.
Q. -- but you've not carried out any calculations of your own?
A. No.
Q. Thank you. If we look at Dr Glover's report, please, ER1, tab 6, page 7, and if you could go to

paragraph 6 -- and here we're talking about percentage strength utilisation -- and you can see that Dr Glover says:

"Figure 1 describes a typical stress-strain relationship for the rebar used on this project, and is annotated to illustrate the relationship of certain terms used in the design process, as explained below."

A. Yes.
Q. Then across the top, the horizontal axis, it's right, is it not, that we have the plastic range of the steel?
A. Yes.
Q. And that -- ie the ductility?
A. Yes.
Q. And then on the vertical axis we have the elastic range of the steel; correct?
A. Yes.
Q. And what this shows, does it not, is that the bar-breaks at 650MPa when you've got six threads engaged; correct?
A. Sorry, are you referring to the ultimate tensile strength of 650 megapascals?
Q. Yes.
A. So I think that refers to the rebar.
Q. Yes, that's correct, the rebar.
A. Yes.
Q. So you're agreeing with me?

A. Yes.
Q. And if we look at the slight blip in the yellow line just below the 560, that shows, does it not, that the rebar has a yield stress --
A. Yes.
Q. -- of about 500MPa?
A. Yes.
Q. So you're still with me?
A. Oh, yes.
Q. Good. Then if we look on at what Dr Glover says, if you could go to paragraph 6.4, he says:

"It will be noted from figure 1 that the design ultimate strength is substantially less than the 650MPa ultimate tensile strength, the maximum tensile stress that a material can withstand before breaking. The difference between the UTS and the design ultimate strength represents a large margin of reserve strength and robustness."

And as an engineering statement that is correct, is it not?
A. Now, we have to be careful with this. There is a certain margin of safety which is expected. This is required by the code. And on the term "robustness" I do have some comment. You may refer to my report.

I have made the comment based on the meaning of

robustness used in the design code rather than used in common language. Well, we have to be very careful with this statement.
Q. We seem to have to be careful with everything, Prof Au.
A. Yes.
Q. But so far as that last sentence is concerned, what I suggest to you -- and Dr Glover is coming along to give evidence in a day or so -- is that he's absolutely right, "The difference between the UTS and the design ultimate strength represents a large margin of reserve strength and robustness" -- that's right as a statement?
A. No. Strength there is a reserve. Robustness is a different issue.
Q. So reserve of strength but you say robustness has to be considered in a different way?
A. Oh, yes. Refer to the design code.
Q. Let's have a look at 6.6:
"Most elements in a structure are not operating at 100 per cent of their capacity under their full operational loadings."
Again, as an engineering statement, I suggest to you that that's correct, is it not?
A. Yes, correct. That is expected, yes.
Q. Thank you. And:
"This can be a result of prudent design,

standardisation or the fact that the critical loading conditions had now passed, for example because they occurred during construction and were not to be realised in the future."
A. Yes.
Q. Agreed? Thank you. And:
"The measure of this over-provision is commonly referred to as the percentage strength utilisation of an element; the SLS stress will be proportionately lower."
A. Yes.
Q. Thank you. Then we see what the percentage strength utilisation equation is in 6.7; presumably that's something you'd go along with?
A. Yes.
Q. Thank you very much.
6.10:
"For this structure, these low levels of utilisation arise in great part from the phased nature of the construction. During construction, the EWL slab was free spanning between the diaphragm walls and subjected to severe construction loads; the slab was designed for these extreme conditions."
And again presumably you would agree with what Dr Glover says there?

A. Now, yes, of course --
Q. Thank you.
A. -- the construction loads must be taken into account, but whether these are the extreme conditions, I'm not sure. There may be other extreme conditions.
Q. Okay. So subject to that perhaps reservation you agree with what Dr Glover says?
A. Yes.
Q. Thank you very much.
Then going on to 6.13, he says, his opinion is:
"These levels of utilisation confirm the structure has a comfortable level of robustness and redundancy."
Again I suggest to you that that is the proper conclusion to draw, Professor.
A. I don't agree. I don't agree.
Now, there are two terms here. Robustness, as I have pointed out earlier, has another meaning; okay? Strictly speaking -- actually, you can refer to the Concrete Code, and if you refer to British Standard or whatever, there is certain explanation for robustness, and I think in my report I have referred to the case of the Ronan Point incident and that was the beginning of the design for robustness. So that is for robustness, as used in structural engineering.
The other term, "redundancy", of course the

technical meaning of that in structural engineering may be different from common usage. So the redundancy of a structure actually is something which is quite basic. So the course that I'm going to lecture in the afternoon actually covers that, so it can be covered in very elementary structural engineering books, textbooks.

Q. It sounds as though Dr Glover ought to go to the lecture to be properly tutored.

A. I would welcome, yes.

Q. There we are. That's the difference between you; Dr Glover has got it completely wrong.

Let's have a look at another document, H44520.

CHAIRMAN: Could I just ask here -- I understand what "robust" means in layman's language. What does it mean in engineering terminology?

A. Robustness or redundancy?

CHAIRMAN: Robustness. Redundancy we've heard about and I understand that.

A. Okay. Perhaps I can refer to the Ronan Point case, incident. I think that happened in the 1960s, if I'm correct.

MR BOULDING: 1968.

A. Yes, I think in the UK. Actually in a tall building explosion happened in one of the flats, and somehow it blew out the wall, and because of that the upper storeys

fell down; okay?
CHAIRMAN: Yes.
A. And the weight of the debris was so heavy that all the storeys below it somehow collapsed. So that was the beginning of the study of robustness. Later on people realised that we cannot just provide -- well, just consider the normal loading cases. We have to refer -- we have to provide something more, for example lateral ties, because things can go wrong. So if we provide all this, then there will be more robustness.
The other case which I can remember vividly was a video which I received from Prof Paul Pang, I think some of you may know him -- he used to work in the Buildings Department -- he sent us a video showing the shelves inside a warehouse, and the shelves are all carrying certain heavy loads, and then somehow a lightweight truck touches one of the shelves and then the shelf began to fall down and then all of the shelves fell down.
Now, that is related to robustness. So if we provide some additional bracing or horizontal ties or whatever, that would help a lot. So that is why there are certain design rules in the design code specifying that we need to provide certain percentage somewhere to ensure that things won't go wrong. Now, that is the

meaning of robustness. And yesterday I mentioned a paper which won me an award in 2016 and that was also about robustness of precast segmental bridges.
MR BOULDING: Good. Well done.
CHAIRMAN: Could I just -- I'm sorry, I didn't mean to cut across you, Mr Boulding.
MR BOULDING: No, sir, it's important that you --
CHAIRMAN: Could I just ask you, it's clearly in engineering terms now a very well-accepted principle that needs to be adhered to.
A. Yes.
CHAIRMAN: And no doubt Atkins would have built that into their design, would they not, a robustness element?
A. So if they have provided all the necessary reinforcement specified by the code, even though it is not required based on design calculations, I believe it will be robust enough.
    So when we try to assess whether a structure is robust or not, we should not just look at the stress level. Stress level is one thing, but then how are various components tied together --
CHAIRMAN: I appreciate that, and I'm not saying that Atkins have got it right or wrong, because I'm taking tentative steps on this, but my question was: surely Atkins would have, in its overall design, sought to integrate into

that design the principle of robustness?
A. Yes, I believe so. So following the code would --
CHAIRMAN: So that answers that.
A. Yes.
CHAIRMAN: So the next question -- you have to forgive, lawyers like to take things directly and get a direct answer; build the integrity of the structure on the basis of direct answers -- if they have done that, are you then saying that they may not have been fully successful in integrating the concept of robustness into the design?
A. Now, what I'm saying is that if -- okay, my point is that just looking at the stress level cannot lead us to a conclusion that it is robust. We have to look at the other things. We have to look at whether or not certain provision of reinforcement, ties or whatever, have been completed. So the stress level is one thing but it is not sufficient to conclude that it is robust.
COMMISSIONER HANSFORD: Prof Au, I think what you're telling us -- or tell me if I've got this right -- is if it's complied with the code, then it will be sufficiently robust?
A. Correct, including those rules for detailing.
COMMISSIONER HANSFORD: Which is compliance with the code?
A. Yes.

COMMISSIONER HANSFORD: So compliance with the code provides robustness?
A. Yes.
COMMISSIONER HANSFORD: Is what --
A. Yes.
CHAIRMAN: And has there been compliance with the code in that regard?
A. I believe -- now, I haven't looked at all the design drawings or whatever, but based on my observations so far, I think the original design should be acceptable, the original design. But then after omitting something or changing something, that I'm not sure.
CHAIRMAN: We're talking about the through-bars now?
A. Okay. Well, two different things. When I answer --
MR PENNICOTT: I think that was a question, sir. You were asking the question, whether he was referring to through-bars or something else.
CHAIRMAN: Yes. In this, you are referring to the through-bars, because that's the change you're relating to?
A. Yes, yes.
CHAIRMAN: All right. Good. Thank you. That helps me to understand the concept and the difficulties. Thank you very much.
MR BOULDING: Thank you very much, sir.

But again, so far as you are concerned, you haven't carried out any calculations to date to establish that the through-bar is not robust?
A. No, not myself.
Q. Thank you. And if we could go back to the document that I was inviting your attention to -- it's H44520. I think we need to get it on the screen for the Commissioners. That's not H44520. That's it. Splendid.
Here we've got, have we not, the lab tests which were carried out by BOSA with the Buildings Department witnessing what was carried out; correct?
A. I believe so.
Q. And the tests were carried out in the CASTCO Testing Centre in Fanling; do you see that? It's on the top, third line.
A. Yes.
Q. I understand that's a reputable testing centre.
A. Yes.
Q. And in circumstances where BD witnessed the test, you would expect them to point out, would you not, if they considered that the tests were invalid in any way?
A. One concern is the strength of the --
Q. Can you answer the question first, please?
A. I beg your pardon?

MR SHIEH: What was the question?
MR BOULDING: In circumstances where BD were witnessing tests, you would expect them to point out, wouldn't you, if they thought that the testing procedure was invalid in any way?
A. I believe so.
Q. Thank you.
Then if we look at the document, we've got some legends down at the bottom, have we not?
A. Yes.
Q. B, S and C, explaining what the letters in the "Mode of failure" column mean?
A. Yes.
Q. We can see, can we not, the left-hand column, "60 per cent threads engaged"; do you see that?
A. Yes.
Q. And we get a tensile strength of 705MPa in the rebar; correct?
A. Yes, correct.
Q. And that tells us, does it not, that the connection between the rebar and the coupler remains intact; that's right, isn't it?
A. The results are not conclusive, because the trend of the results is very strange.
Q. I'll put the question again.

A. I don't believe it. Now, I don't think we can base on this for our design. So just one sample for each case, and then the results are so strange.
If the coupler is fully engaged, 100 per cent engaged, we would have expected that the strength is even higher, but now, presently, the higher strength is when the coupler is partially engaged. Now, apparently that shows the variability of the assembly. I won't trust that. Just one sample for each case.
Q. Prof Au, again, have you done any calculations of your own?
A. No.
Q. And proceeding on the basis here -- I'll put the question again -- the figure of 705MPa tells us, does it not, that the connection between the rebar and the coupler remained intact in the test; that's right, isn't it?
A. That's right, yes.
Q. And thus served its intended purpose; correct?
A. No. So, in addition to strength, we have to look at the performance of the coupler assembly in elongation, ability to survive, cyclic loading and so on.
Q. I'll come to that. And the "B" in the "Mode of failure" column tells us, does it not, that at 60 per cent engagement it's the parent bar --

A. Yes, correct.
Q. -- and not the coupler which breaks?
A. Correct.
Q. What I suggest to you is that on the basis of these test results -- and you've told us you've got nothing of your own to contradict it -- this means, does it not, that at 60 per cent engagement the rebar-coupler connection has adequate structural integrity?
A. No.
Now, looking at strength, it appears to comply, but then -- now, it doesn't mean that it can be comply with other things.
Now, the other thing that we can sort of -- another question we can put forward is -- now, if the results are really that trustworthy, should we only partially engage all of the couplers by 60 per cent? Because the strength appears to be highest. Is it realistic? Is it reasonable?
Q. Well, that's what the test is showing, I suggest to you, Prof Au.
A. Well, now, if you have done any testing, if you do testing of a number of samples, which are identical, the results won't be exactly the same. There would be variations.
Now, what I'm suggesting is looking at these

results, I don't trust them, because it suggests that there is large variation, because it doesn't make sense. The trend is very strange.
CHAIRMAN: Can I ask this -- sorry, Mr Boulding.
MR BOULDING: Please go ahead, sir.
CHAIRMAN: Mr Boulding perhaps will be able to assist me. Were these tests carried out in respect of one sample each?
MR BOULDING: I couldn't tell you that, sir, without asking for instructions.
CHAIRMAN: I take it each one has to be separate, because you're talking about a breakage point.
MR BOULDING: Yes.
CHAIRMAN: So the 30 per cent of threads engaged, that would have to be one sample, and then there would have to be another one, for each one to reach a destruction point. I'm just wondering if this was done with just one sample going through or a number of samples.
MR PENNICOTT: Our understanding, and it is only an understanding, is that it was just one sample for each percentage.
CHAIRMAN: All right. So what you're saying, Professor, is one sample, strange result, can't trust it?
A. Correct.
MR BOULDING: Well, I suggest to you that the peak we see at

60 per cent, which you say is the strange result, is the consequence of the natural variation of the coupler connections and the material properties.

A. No, that's wrong. I can tell you, if you are familiar with the testing of samples inside laboratory -- let's say if we have a number of samples which are supposed to be identical, we test them, we would expect some natural variations; okay? But there may be some outliers which are very far away from the mean.

Now, I guess probably some of the results could be outliers.

Q. So you are guessing?

A. Well, the results show that they are strange, strange result.

COMMISSIONER HANSFORD: Prof Au, I'm puzzled by this because if BOSA and CASTCO and BD who witnessed these tests had considered this to be an unusual result, they would have called for further tests.

A. Yes, I believe so.

COMMISSIONER HANSFORD: But they didn't.

A. I don't know why.

COMMISSIONER HANSFORD: So presumably they accepted this?

A. I'm not sure if they accepted. I'm not sure.

May I --

COMMISSIONER HANSFORD: I'm puzzled by it.

A. May I add some further information?
MR BOULDING: Are you guessing or are you giving an opinion?
A. No, no, no. Well, it's an established practice. On site, when we need to monitor the concrete strength, normally we need to cast many cubes, and then after 28 days we crush them and then find out the strength. We don't just test one cube because there could be variations. That is the standard practice. So I'm very surprised to find that just one sample is tested for each case.
COMMISSIONER HANSFORD: But BD would be familiar with that practice as well?
A. I believe so, but the problem is I don't think this is the standard testing procedure, because this is something which is unusual. BOSA -- well, the testing of the coupler assembly should be tightened up entirely and then tested. So this is something which is unusual. I think this is not standard practice.
CHAIRMAN: All right. So this is a series of tests --
A. Yes.
CHAIRMAN: -- conducted for the purposes, essentially, of the Commission of Inquiry?
A. I believe so.
CHAIRMAN: And perhaps not thought out as deeply as it should have been?

A. I believe so, because they should have addressed not only strength but also elongation, and so on. They should have tested sufficient number of samples.
I think that is well accepted in the industry.
CHAIRMAN: Yes.
MR BOULDING: Anyway, I've got to suggest to you -- and it's certainly Dr Glover's opinion of the matter -- that the test that we've been discussing over the course of the last ten minutes or so establishes that the rebar-coupler connection has adequate structural integrity and is safe.
A. No. No. So the results are unreliable. Just one sample, it is not enough.
Q. Okay. We look forward to your calculation, Professor.
A. Well, actually, this is not based on calculation. This is based on testing.
Q. Exactly. You've never done any tests, have you?
A. I have. I have been authorised signatory of our HOKLAS accredited lab until some years ago we gave up that lab.
Q. So have you carried out this test yourself?
A. Yes.
Q. What are your results then?
A. The results normally -- the results fluctuate. So that's why, when I look at the results, I don't trust the results.

CHAIRMAN: All right. That's it. You are saying, "I have not carried out tests myself that I have the results for. I am concerned that only one sample is used for each of the six tests; there should have been more"?
A. Yes.
CHAIRMAN: "Especially as the results are not entirely predictable"?
A. Correct.
CHAIRMAN: So if there had been, shall we say, three samples in each case, and then you had been able to look at the results of the destructive issues there, then they would be more persuasive?
A. Yes, I believe so. I think three samples would be the absolute minimum to give us any confidence. The more the better.
CHAIRMAN: All right. Thank you very much.
MR BOULDING: Thank you.
Then proceeding on the basis that I have put to you, if we look at OU314 -- and here we've got the 67 testing results, have we not, as of 12 January?
A. Yes.
Q. You will recall discussing these with Mr Pennicott yesterday, won't you?
A. Yes.
Q. Proceeding on my basis and the opinion of Dr Glover,

that 60 per cent engagement represents safety, it would be right, would it not --
A. No.
Q. You haven't heard the question yet!
A. I should pause a bit.
Q. It would be right, would it not, that there are only two results on this sheet that would be regarded as "failures"; that's right, isn't it? Number 6, number 5 and number 22?
A. Are you referring to the 60 per cent engagement as the criterion?
Q. Yes.
A. Well, I don't accept 60 per cent, sorry.
Q. And of course the slab on which these results -- the results comes from slabs which have now been completed, what, for something like two years?
A. I'm not sure. Around that.
Q. And during that period, the trains -- we've had trains running; correct?
A. Yes, correct.
Q. And it would be right, would it not, that after two years the slab is approaching its full loading?
A. Well, it's hard to say so because --
Q. Sorry, I've stopped you.
A. Well, it is not normal running yet and it depends also

on the design loading adopted.

Now, we are just considering the working condition or what we call serviceability limit state, just that, the working condition.

Q. It would be right, would it not, that two years after it's been completed we've seen no indications of fatigue in the sense of cracking, spalling of concrete; we've not seen anything like that, have we?

A. Except for honeycombing, that type of thing, yes.

Q. Well, that's something different.

A. Yes, I haven't seen any. But I have just visited the site a few times. I'm not a site staff now.

Q. I think if we had any evidence of fatigue, cracking or anything like that, it would have been put before the Commission of Inquiry, wouldn't it?

A. I'm afraid you don't understand the term "fatigue cracking".

Q. I'll come to your lecture this afternoon.

A. Sorry, I should be away this afternoon for teaching.

Q. If we look on in your report, page 4, paragraph 2.1, "Acceptance criteria and performance of reinforcing bar couplers":

"Reinforcing bar couplers are proprietary products designed and manufactured to comply with the relevant design code or an alternative standard accepted by the

Building Authority. Apart from satisfying certain strength requirements, the coupled bar assembly should also comply with certain requirements in respect of deformation characteristics."

We have already discussed the 60 per cent engagement point, but so far as your point on deformation characteristics is concerned, it's right, is it not, that because of the increased stress in the assembly, there would be what is referred to as elongation in the coupler bar assembly?
A. I beg your pardon. Can you repeat the last sentence?
Q. Yes. Because of the increased stress -- you're referring to deformation characteristics; correct? That's what you're referring to here?
A. Sorry, which sentence?
Q. The first two sentences.
A. Okay.
Q. Do you want me to read it again?

"Apart from satisfying certain strength requirements, the coupled bar assembly should also comply with certain requirements in respect of deformation characteristics."
A. Yes.
Q. It's right, is it not, that because of the stress in the assembly, there will be what is referred to as

elongation in the coupler bar assembly; correct?
A. Yes.
Q. And, as I understand and Dr Glover understands your concern, this could be a crack propagator, correct; that's one of your concerns?
A. Are you talking about the concrete structure or just the coupler assembly?
Q. The coupler assembly.
A. I think that has been caused by slipping inside the coupler.
Q. Well, whatever the cause, what I do suggest to you is that these deformation characteristics, as you refer to them, would be very small, wouldn't they -- very small?
A. We are talking about --
Q. Will you answer my question.
A. I am answering your question.
Q. Good.
A. We are talking about elongation in the range of 0.1 millimetre, of course small, but then, even though it is small, it may cause cracking.
Q. We know, do we not, that these very small deformation characteristics, 0.1 millimetre, will be studied in the test programme that MTR is just about to embark upon? Is that something you know?
A. That I don't know.

Q. You don't know? Okay.

Staying with your paragraph 2.1, in the last sentence, you tell us:

"It is often expected that the structural performance of a concrete member with coupled bar assemblies is not inferior to that with the equivalent continuous bars in all aspects."

Now, what I've got to suggest to you is that's an incorrect statement, because it's never expected that a coupler connection will behaviour in the same way as a continuous bar.

A. What I am saying in this statement is that it is not inferior to. If we replace continuous bar by coupler assembly and if the performance is inferior, I don't think that is acceptable.

Q. What I'm suggesting to you is that a coupler connection will never behave in the same way as a continuous bar, will it?

A. They may not be the same.

Q. Thank you. That's because, I suggest, there's always some give in a coupler connection; that's right? That's always some give?

A. Yes.

Q. Albeit that we'd be talking, I suggest, about fractions of a millimetre?

A. Well, are you talking about elongation?
Q. Yes.
A. Well, I think that is the problem. Now, the code specifies certain requirement on elongation and the coupler assembly must comply with that.
Q. Then 2.3, you say here:

"In appendix 1 of Paulino Lim's witness statement regarding BOSA's requirement, the equivalent strengths of coupled bar assemblies of the BOSA Seisplice system for 40 millimetre reinforcing bars having different engaged lengths are calculated and presented in Chinese. Based on BOSA's calculations, a splicing assembly having 6 threads engaged (... as opposed to 40 millimetres ...) will be sufficient to develop the axial strength of reinforcement. It is however noted that strength is just one of the aspects of structural performance."

Now, it would be right, would it not, that you would need to take account of utilisation levels when one is considering structural performance?
A. Are you talking about strength utilisation factor of the structure or just this coupler assembly? Which are you referring to?
Q. What are referred to as utilisation levels.
A. Well, if you are referring to the utilisation level of the structure, sorry, well, still we have to make sure

that the coupler assembly complies with the requirements.
Q. But what I'm talking about is the fact that it's right, is it not, that elements of a structure are tested to 100 per cent of their utilisation; that's right, isn't it? Elements of a structure are tested to 100 per cent of their utilisation?
A. What do you mean by "element of structure"?
Q. Well, all the elements of a structure.
A. Are you talking about beams, slabs, columns or the coupler assembly?
Q. The structure as a whole.
A. Could you repeat your question, please?
Q. Yes. Elements of structure are tested to 100 per cent of their utilisation; that's correct as a proposition, isn't it?
A. I don't understand your question.
Q. And the structures that we're talking about here do not perform generally above a utilisation of 50 per cent or less?
A. Well, I've seen figures provided by different experts and there are variations. Well, most of them appear low, but then there are some figures which are quite high. I think there are some figures even above 100 per cent.

Q. Well, Dr Glover tells me, and I suggest to you, that the structures under consideration do not perform generally above a utilisation of 50 per cent, and sometimes less.
A. Well, it may be true, yes.
Q. Thank you. And if it be true, it would be right, would it not, that we would be doubly assured that the structure has the requisite structural integrity because it would never have to meet 100 per cent utilisation; that's right, isn't it?
A. No, it is not right. You are just talking about strength. There are other aspects, the elongation, and so on, that would be related to the cracking, possible cracking of the structure, and then possible increase of deformation, deflection, and so on. We cannot just focus on strength. There are other aspects: deformation and ability to sustain cyclic loading.
Q. I hear what you say about that.
At paragraph 2.5 -- we have touched upon this table already. Table 2.4.1 replicates the table we looked at earlier, does it not, Prof Au?
A. Oh, yes.
Q. Just to pick up a point I don't think we quite bottomed out before, you say in 2.5:
"However, the unusual trend observed (eg the maximum value occurring at 60 per cent engagement) suggests that

variations of results can be quite large."

Again, I would have to suggest that it's not in fact a large variation, being just the sort of natural variation you would expect to get in materials and the testing thereof?

A. Sorry, no. Actually, I would have expected that the larger the engaged length, the larger the strength -- if that's the case, I would tend to believe, despite the fact that there is just one sample for each case. Now that the maximum strength appears at 60 per cent engagement, it is very strange. Now, how can we tell people that "we should not tighten all the couplers, just tighten to 60 per cent of the length"? It doesn't make sense.

Q. We talked about that earlier. There's a difference between the quality requirement and the structural integrity requirement. We can check the transcript for that. That's where we started.

A. Okay.

Q. What I would suggest is that, for example, the difference between the 705MPa and the 693MPa is just a natural variation in the strength of the rebar.

A. That's true. But then if you go further to 100 per cent, it looks very strange. So if you fully tighten that up, it drops. So it looks very strange.

How can we explain?
Q. What I'd also suggest is that these variations are of no significance in the overall scheme of things.
A. Well, they are significant. We cannot accept that kind of test results. So the test results do not display a reasonable trend. I think, as an engineer -- now, there must be more tests to come up with something more trustworthy.
Q. All right. Just going back on our discussion about utilisation and my suggestion of low utilisation, it would be right, would it not, that when one talks about low utilisation, you would get a low or small amount of cracking? If you've got low utilisation of a structure, you get a small amount of cracking; correct?
A. Well, I think your question is very strange. Well, allow me to say so. Of course the larger the loading or the utilisation, the larger the deformation, the larger the internal forces or whatever.
Q. So I think you're agreeing with me.
A. Yes. But whether cracking occurs depends on the design of the structure.
Q. But generally, with low utilisation, you get low cracking and you would also get low deflection; that's right, isn't it?
A. Yes, assuming that the structure has been constructed as

designed, there is no defect or whatever.
Q. And, on the other hand, with cyclic loading, which I think is a term that you introduced into our discussions, you would only get cyclic loading in circumstances where you get, for example, earthquakes? Earthquakes would give rise to cyclic loading, wouldn't they?
A. Earthquakes certainly would give rise to cyclic loading, but remember we are talking about a railway station. It is true that when we design railway bridges, we need to check fatigue, we need to check cyclic loading, exactly. That is one of the additional reasons to support the need to look at the performance under cyclic loading.
Q. And I think we could agree, couldn't we, that fortunately we don't have very strong earthquakes in Hong Kong, if indeed we have them at all? It's an area of low seismic activity, isn't it?
A. I agree.
Q. Just a small point. You were asked -- at paragraph 3.1, you're talking here, are you not, about the Code of Practice for Structural Use of Concrete 2004; correct?
A. Yes.
Q. You were asked about that by my learned friend Mr Shieh yesterday. I just wonder whether we could have a quick look at that: H2821.

Do you remember being taken to this document by Mr Shieh yesterday?
A. Yes.
Q. And the discussion is recorded in the transcript that it was suggested to you that it wasn't a mandatory requirement?
A. Yes.
Q. Do you remember that suggestion to you?
A. Yes.
Q. What I just point out to you -- do you see the foreword, first paragraph:

"This Code of Practice provides guidelines for the professionals and practitioners on design, analysis and construction of concrete structures. It was prepared by the consultant under the direction of the Buildings Department's steering committee for the consultancy study on structural use of concrete using limit state approach."

What I suggest to you is that in addition to the various points put to you yesterday by my learned friend Mr Shieh to the effect that it was not mandatory, that's another indication, is it not, guidelines provides guidelines for professionals, but the contents of this code are not mandatory; that's right, isn't it?
A. Actually, yesterday, I explained that this code lays

down all the requirements. If you comply with the code, fine, that would be accepted by the Building Authority. Of course you can come up with your alternative design which does not comply, but you have to demonstrate, by whatever means, calculations, testing or whatever, to demonstrate that it is not inferior to the performance prescribed over here. And that requires a lot of work to demonstrate.

So I explained yesterday already two cases are possible. The first one is you can come up with exactly the same performance, and then the next one is to come up with a performance higher than the level prescribed over here. But in most cases it would be higher; okay? So in this sense we can regard that as mandatory. Of course that sets the standard. It's up to you. If you follow, fine. If you don't follow, you demonstrate that it's not inferior to that.

Q. Guidelines only?

A. But I have mentioned that it must not be inferior to what is laid down over here.

Q. Just to pick up a point so far as cyclic loading is concerned -- you referred to the fact, did you not, that we're talking about a railway station, and thus you have to take account of the effect of trains?

A. Well, I am just telling you that a railway station is subject to cyclic loading, but as far as I understand there is no need to do the load combination to account for the cyclic effect.

Now, of course we can do that, but normally that is not critical, but if we design steel bridges, now, that is a different issue, steel railway bridges; that would be very critical.

Q. What I've got to suggest to you is because of the fact that the trains sit on the D-wall and not the slab, that would mean that the slab doesn't vibrate and thus the cyclic loading would be very low. That's what I suggest to you.

A. So are you referring to the vibration caused by earthquakes?

Q. No, trains.

A. It would be very low because it's quite bulky, yes.

Q. Thank you. Staying with paragraph 3.1.1 of your report, you refer to the Code of 2004, and we talked about that, and then you say:

"Therefore, the proper connection of the bottom reinforcement of the EWL slab to the diaphragm wall by way of mechanical couplers was required and would also serve useful purposes."

Now, as to your statement that connection by way of mechanical couplers was required, it would be right,

would it not, that you do not in fact need type II ductility couplers? You do not need type II ductility couplers?
A. I understand that --
Q. Is that right or wrong?
A. Let me explain. Now, Concrete Code 2004 has certain restrictions on the use of couplers, especially at certain locations. But as far as I understand, if you put couplers at such connections, that would be a requirement, to use ductility coupler. Now, that is as far as I understand -- that is a requirement by the Buildings Department.
Q. Anyway, we know that but you nipped down for your sandwich on 18 December, all of the experts agreed that there was no requirement for ductility couplers. You have seen that in the joint statement?
A. I saw that.
Q. I wonder if we can discuss why that is a correct statement. If you would be kind enough to go to Dr Glover's report, and when you are there go to page 4, paragraph 4.5.
A. Thank you.
Q. Here Dr Glover says, 4.5:
"A type II coupler has been designed for more extreme loading conditions where the connection is

subjected to stress reversal (ie tension to compression) through a number of cycles of such stress reversals, as would be the case in very strong ground motions caused by large earthquakes. However, the Hung Hom Station box would not be subjected to such very strong ground motions under the low to moderate earthquake seismicity classification which it is predicted that Hong Kong might be subjected to."

Presumably, that's a paragraph you would agree to, is it not, Prof Au?

A. Now, I think the last part of the paragraph, yes, I do agree. In Hong Kong, the seismicity should not be too high. But then the use of the ductility coupler is a different issue. It cannot be just related to seismic design. So it is now a requirement.

Q. Sorry, you say it's a requirement. Why is it a requirement?

A. That is required by the Buildings Department. So if you use couplers at that location -- now, that has to be ductility coupler. That's what I have understood.

Q. Why is it required by the Buildings Department? What's the authority for that?

A. Well, now, as far as I can understand, Concrete Code 2004 has certain rules for the location of the laps and the couplers.

Q. We've been there. That's guidelines only and it's not mandatory.
A. You have to demonstrate that it is not inferior to that, not only in terms of strength but also ductility and other things.
Q. And if you were to point out to the Building Authority that Hong Kong is low to moderate earthquake seismicity classification, you would say, "Buildings Department, that is a jolly good reason, is it not, why we do not need type II couplers"; that's correct, isn't it?
A. No. No. Well, actually, we are moving towards seismic design. We are moving in that direction.
Q. And as Dr Glover says in paragraph 4.6, there are other jolly good reasons for that. First of all, as we've discussed already, the Geotechnical Engineering Office of Hong Kong states in its recent note of 2015 that the seismicity of Hong Kong is low to moderate; that's correct, isn't it?
A. Correct.
Q. And, secondly, he's right to point out, is he not, as he does in his second bullet point:

"Underground box structures have performed exceedingly well in very strong earthquakes which is reflected in the way these structures are designed internationally."

Again, he is correct to make that statement, isn't he?
A. I have no comment because I haven't read that particular document.
Q. So you can't contradict him then, can you?
A. No.
Q. Then the third bullet point:
"Hong Kong reference documents also reflect the low seismic risk associated with such structures. Information note [again 2015] ... states in its key messages '(c) The possibility of significant earthquake damage to manmade slopes, retaining walls and reclamations in Hong Kong is low'."
Again, it's correct, is it not?
A. Yes.
Q. That's another reason, I suggest, why you do not need type II couplers; correct?
A. We still need type II couplers. Why not? It is very important, because we have to ensure that our structure is strong and ductile and safe.
Q. Then the last reason -- and again I suggest it's a good reason:
"Due to the disproportionately stiffer and stronger EWL slab (3,000 millimetres deep) relative to the diaphragm walls (1,200 millimetres thick), it would be

impossible to develop ductile behaviour in the slab or its connection to the walls since the wall would have failed structurally under ultimate load conditions long before the rebar in the slab would have reached its yield stress ..."

Again, that is a correct statement, is it not?

A. Now, we have to be careful. What you are referring to is the possible failure mode. It may occur at the wall, but it is not the reason or excuse not to use ductility coupler at the bottom of the EWL slab.

Q. There we are. For all those reasons, I suggest that type II couplers were not required, as in fact all of the experts agreed during the course of the meeting held on 18 December.

A. I don't agree.

Q. You don't agree.

Then if we look at your paragraph 3.2.2, here you are dealing, are you not, with the assessment of the NSL slab?

A. Oh, yes.

Q. You say in 3.2.2:

"Moreover, the top reinforcement in NSL slab near the east and west diaphragm walls may also be required to take tension in the rare case of future dewatering in the vicinity."

Now, first of all, it's right, is it not, that the NSL slab is ground-bearing; correct?
A. Sorry?
Q. Ground-bearing?
A. Yes.
Q. So it rests on the ground?
A. But then very often we have to -- in this case the ground would also deform, and it will be prudent to also consider the load case that the slab is carrying its own weight.
Q. But here we know, do we not, that the land upon which the Hung Hom Station was constructed was reclaimed back in the 1960s; that's right, isn't it?
A. Right.
Q. And when it was reclaimed, the ground was surcharged?
A. Yes.
Q. So we've had surcharging going on, what, for at least 50 years?
A. Yes.
CHAIRMAN: Sorry, Mr Boulding, just a quick lesson -- surcharging? The only surcharging I know is financial.
WITNESS: Let me explain. Sir, may I explain?
COMMISSIONER HANSFORD: (Unclear words).
CHAIRMAN: Ah. Okay. Thank you very much.
COMMISSIONER HANSFORD: Sorry, will that do, Prof Au?

WITNESS: No problem.
COURT REPORTER: Sorry, I didn't hear what you said.
COMMISSIONER HANSFORD: I said it's had a load applied to it for 50 years.
MR BOULDING: What I'm instructed is it's been surcharged for 50 years with a depth of soil of some 15 metres?
A. Yes.
Q. And in those circumstances it would be right, would it not, that the ground has been very well compacted?
A. Yes.
Q. And in those circumstances, what I suggest to you is that the risk of settlement would indeed be very remote?
A. Now, it's very complicated. If we consider the part of the station below the NSL slab, so the NSL slab is supported by the soil underneath, but there are also certain diaphragm walls on the two sides and -- well, some other piles or whatever.
    Now, in comparison, the stiffness of the diaphragm wall and the piles would be a lot bigger than the soil.
Q. I thought you'd say that. And what I suggest to you is that because of the long spans of the slab, the slab would, to a very large extent, be unaffected by the D-walls which support the end of the slab. Because of the long span of the slab, the slab would be, to a very large extent, unaffected by the D-walls?

A. Sorry, I don't understand your question.
Q. Well, because of the fact you've got a long span of the slab, and then it's joined up to the D-walls --
A. Yes.
Q. -- and what I suggest to you is that if there was any effect at all, it would only be in terms of minimal flexing where the slab joined the D-wall.
A. Well, now, that is precisely the concern. The slab will deflect downwards, while reversing the force carried by the top reinforcement. That is the concern.
Q. It would be minimal, wouldn't it?
A. Well, we have to do calculations.
Q. Again, but you haven't got any calculations?
A. I haven't.
Q. Well ...
A. But then it doesn't mean we can ignore it.
Q. And so far as dewatering is concerned, we know, do we not, that the Buildings Ordinances do not allow dewatering to occur, do they?
A. Now -- well, there are certain restrictions. So that's why -- well, where are we? I think I mentioned "rare".
Q. Yes.
A. So it's rare. But then, if it's rare, does it mean that we can ignore it?
Q. There we are. I think I've taken that as far as I need

to take that one.

Paragraph 6.1.1. Here you deal with possible safety concerns. 6.1, "Ductility in structural design". You say:

"In general, ductility is a desirable quality of all structures, irrespective of whether a structure is designed for seismic resistance or not."

Just pausing there, it would be right, would it not, that albeit that it might be a desirable quality, you would get it naturally in reinforced concrete structure? Reinforced concrete structure would have inherent ductility in it, wouldn't it?

A. Okay. If you follow the code and all the rules for reinforcement detailing, you probably will get the ductility required.

Q. Thank you.

A. But if you don't follow that, just like one of the first few slides I showed you, the plain concrete beam, if you don't put in any reinforcement, it would be very brittle; no ductility at all. So it depends how you provide the reinforcement.

Q. And whilst ductility is desirable, presumably you would agree with me that strength is absolutely essential?

A. Yes, of course.

MR BOULDING: Thank you, Professor.

WITNESS: Thank you.
CHAIRMAN: Anybody else have any questions? Any matters arising?

Questioning by THE COMMISSIONERS

COMMISSIONER HANSFORD: I have one question, perhaps if I could ask Prof Au.
Prof Au, in your witness statement, in paragraphs 4.2 and 4.3 --
A. Yes.
COMMISSIONER HANSFORD: -- you address the matter that's been suggested by some of unscrewing threaded reinforcement bars that are already in situ --
A. Right.
COMMISSIONER HANSFORD: -- unscrewing them for testing, and you conclude in your final sentences of both of those paragraphs, firstly, it's "considered unnecessary and therefore not recommended", and you go on to say it "would mean that the structure will be damaged further".
A. Yes.
COMMISSIONER HANSFORD: And you still hold that view?
A. Yes, but then in case there is a need to address public demand, then perhaps a small sample can be done, but I don't think that is necessary.
COMMISSIONER HANSFORD: You consider that unnecessary --
A. Correct.

COMMISSIONER HANSFORD: And you also think it would damage the structure further?
A. Yes.
COMMISSIONER HANSFORD: Thank you. That's all.
CHAIRMAN: Anything arising? Yes, sorry.
MR CHOW: Mr Chairman, we have a few questions in re-examination, but I see it is 11.20. I wonder if it's a convenient moment to take the morning break so we can come back and I can start.
CHAIRMAN: If you would prefer that. I'm quite happy to finish and then we can have the break and start with the next witness.
COMMISSIONER HANSFORD: That way, Prof Au can get to his teaching appointment.
MR CHOW: But I'm afraid my re-examination will take longer than 15 minutes, so that's the reason --
CHAIRMAN: All right. If you would like to have the break now, we will oblige. Thank you very much. 15 minutes.
(11.19 am)
(A short adjournment)
(11.41 am)
Re-examination by MR CHOW
MR CHOW: Good morning, Prof Au.
A. Good morning.
Q. I have a few questions for you arising from your

discussion with various counsel yesterday and this morning with Mr Philip Boulding.

The first topic I would like to discuss with you is you will recall that yesterday Mr Pennicott had taken you to or referred you to some test reports produced by BOSA --

A. Yes.

Q. -- regarding pulling-out test or tensile test on couplers with different engaged lengths?

A. Yes.

Q. And some of the experts refer to those test reports and suggest that new acceptance criteria should be considered which is six threads being engaged would be sufficient to develop the tensile strength or the design strength of the reinforcing bar. Do you still recall that?

A. Yes.

Q. And Mr Pennicott asked you to confirm whether you have taken this into account when you formed your view as to the percentage of failure of the various results under the opening-up exercise.

A. Right.

Q. Your answer, you basically confirm that you have not taken the test into account.

A. Right.

Q. I am going to ask you why, but before you answer this question, for the benefit of the Commission and the public as well, I think it is useful for us to set the scene and go back a little bit in time to show what actually happened leading up to the test and the test result and the subsequent correspondence.
MR BOULDING: Sir, are we going to allow this sort of leading question in the Commission of Inquiry?
CHAIRMAN: I don't know what the question is going to be at the moment.
MR CHOW: I have not started to ask my question. Of course I will not ask leading questions.
First of all, the first thing to happen is a table setting out BOSA's calculation as to the corresponding strength when different engaged lengths was being investigated.
A. Right.
Q. Do you recall that?
A. Yes.
Q. That table we have looked at this morning.
A. Yes.
Q. It can be found at bundle H25, page 44527.
A. Right.
Q. If we can just briefly look at the table first.
A. Yes.

Q. This is in Chinese. 44527.1 is in English. So this is the table on the basis of calculation; right?
A. Yes.
Q. Then we have the test results which can be found in the same bundle, H25, page 44520. This one we have also looked at.
A. Yes.
Q. In this table, the test was carried out on 21 November last year, 2018.
A. Yes.
Q. In your report, paragraphs 2.1 to 2.5, you have given your comment, your query, as to the reliability of these test results; do you recall that?
A. Yes.
Q. You said the number of samples tested is not enough?
A. Correct.
Q. The result appears to be strange?
A. Yes.
Q. Which does not make sense?
A. Correct.
Q. And therefore, in your report, you form a view that it is not reliable; do you recall that?
A. Correct.
Q. Then we have a press release given by BOSA in the evening on 23 December last year.

A. Yes.
Q. Mentioning about the six threads being engaged, and because of that press release, the Buildings Department raised queries directly with BOSA, and the letter can be found at bundle H26, page 45479.
Now, this is a letter dated 28 December from the Buildings Department to BOSA Technology Holdings Ltd for the attention of Mr Paulino Lim.
In this letter, the Buildings Department refers to the press release issued by BOSA and raised a number of queries and expects BOSA to address them; right?
A. Right.
Q. The first query in paragraph 2:
"We would like to seek your clarification on the following issues mentioned in your press release".
Under paragraph 4 of the press release BOSA said:
"For BOSA's type 2 couplers for a 40mm diameter coupler, the bars are designed to have a threaded length of 44mm, or 10 threads full engagement. In other words, the correct installation is to have 10 threads fully engaged into the coupler."
Now, this is part of the press release by BOSA.
A. Yes.
Q. The question raised by the Buildings Department was:
"We noted the information in the statement is

different from the dimensions table attached in the quality assurance scheme submitted for the captioned project."

And the Buildings Department seek clarification on the minute dimension and how to measure the various lengths, the width of one thread, that sort of thing, but this is not very important for our present purposes.

It is the second part of the letter where the Buildings Department refer to paragraphs 6 and 7 of the press release, in particular the part where BOSA said:

"For a 40mm coupler, 10 full threads will provide a design strength of 1,003 megapascals ... and accordance with BOSA's design approach outlined above, an engagement of 6 threads for example, may provide a design strength of around 600 megapascals, exceeding the specified yield strength of the bar, subject to verification in accordance with structural engineering principles."

Do you see that?

A. Yes.

Q. The query raised by the Buildings Department was:

"What is the correlation between threads of the coupler/rebar and design strength with reference to the performance requirements as stipulated in clause 3.2.8.4 of the Code of Practice for Structural Use of Concrete

2013 (ie the performance [requirement] for permanent elongation, static compression and tension test and cyclic tension-and-expression test, et cetera)?"

Can you confirm that clause 3.2.8.4 of the Concrete Code actually refers to requirements as to elongation?

A. Well, I need to check.

Q. You can take it from me that this is what clause 3.2.8.4 is about.

A. Yes.

Q. We can always verify it later on.

A. Yes, okay.

MR PENNICOTT: No doubt it will be explained why the question was asked by reference to the 2013 Code of Practice rather than 2004.

MR CHOW: Perhaps we can take a look at the version of the Concrete Code for --

CHAIRMAN: Tell me, where are we going on this question? I appreciate some questions require a very large preamble. I'm just wondering what we're going to deal with.

MR CHOW: Mr Chairman, the reason why I need to take Prof Au to this letter is because the following letter, the letter in response from BOSA, I need to set the scene for people to understand under what circumstances BOSA made its response subsequently.

CHAIRMAN: All right.
MR CHOW: Perhaps at this point I don't need to go to the
Concrete Code as such and I can simply move on to the
following letter.
MR PENNICOTT: Sir, I'm not trying to stop Mr Chow going to
the letters if he wishes to, provided we are going to
get some precise questions and some answers that are
relevant to the Inquiry, but could I just put down
a marker that BOSA have been very cooperative to the
Commission and it would appear to government departments
as well. They are not an involved party at this
Commission of Inquiry. Nobody has ever suggested any
criticism should be directed at BOSA, and that's
of course a very important point in the context of this
Commission. They have been helpful, nobody has ever
pointed a finger at them, nobody has ever criticised
them, and they are not a party that can be subjected to
any criticism by the report of this Commission at the
end of the day, because they are not an involved party.
CHAIRMAN: Thank you.
MR CHOW: Chairman, I have no intention whatsoever to
criticise or point fingers at BOSA, but given the fact
that at the moment various experts refer to the test
result as something which entitled the change to the
acceptance criteria, and that is important to have

a look and understanding as to the position of BOSA in relation to the testing they performed.

CHAIRMAN: All right. Fine.

MR SO: Mr Chairman, I do apologise. We wish to put down a marker that those instructing me have confirmed that in the beginning of this Commission of Inquiry, China Technology did make an application that BOSA be made an involved party but that request was not acceded to, but we just wish to make a marker here and we are not insisting that BOSA be made an involved party.

CHAIRMAN: Yes.

MR CHOW: Prof Au, then we have a formal response from BOSA made to the Buildings Department on 7 January 2019. In fact, this is a letter that you have actually referred to in one of your slides as well.

A. Yes.

Q. Can I ask you to go and have a look at H26/45640.

A. Thank you.

Q. Do you recognise this letter?

A. Oh, yes.

Q. In your slide number 17 -- you still recall your presentation?

A. Right.

Q. -- you put down "Couplers with only six threads engaged may not be acceptable"; do you recall that?

A. Yes.
Q. Then in your slide number 8 you also refer to BOSA's letter, that's the letter we are now looking at.
A. Yes.
Q. And you emphasised the requirement of "butt-to-butt"?
A. Yes.
Q. And also ten-thread engagement as what BOSA's position was --
A. Right.
Q. -- as per what BOSA put down in its letter on 7 January.
A. Yes.
Q. Can I ask you to take a look at the letter and tell us which particular part of its letter would give you such an understanding as to BOSA's position?
A. Actually, the last one, if it is not tightened to be butt-to-butt, then the assembly will be loose. That would be one important thing that we need to address.
Q. How about turn over the page to page 2.
A. Okay. So the first paragraph does say something, that we do not have any test data on correlating partial thread engagement of a coupler to its structural performance. I think somehow we are moving to a certain area that is unsure. So even the supplier isn't sure of the performance of the coupler.
Q. Right. In line 6 of the same paragraph, BOSA said:

"... we have provided such samples and conducted tensile strength tests on them and representatives from BD were invited to attend and witnessed such tests. We also understand MTR has conducted various similar tests."

Pausing here, can I ask you whether you have seen any test report in relation to the tests carried out by MTR?

A. No.

Q. Then we can move on:

"So far as we are aware this is the single type of test that has been conducted on couplers with partial engagement and the test results are shown in the photo enclosed. Regarding these results, we could offer no further comment other than that these test results are consistent with our design strength as quoted in paragraph 2(b) of your letter."

A. Right.

Q. And the following paragraph:

"Regarding your question on how a partially engaged coupler would perform in permanent elongation test, static compression and tension tests and cyclic tension-and-compression tests, it is our opinion as explained in paragraph 4 above, that it is unlikely that such couplers, without being spliced butt-to-butt and are therefore loose, will survive permanent elongation, and cyclic tension-and-compression tests.

However, with sufficient partial engagement of threads, such couplers should survive static compression and tension tests in accordance with our design, subject to sufficient tests to be conducted for verification."

Now, if I can then refer you to the second-last paragraph on the same page, starting with, "However":

"However, in the event that full compliance cannot be achieved such as these partially engaged couplers due to various reasons, engineers will need to go back to first principles of laws of mechanics to find out the various objectives of each individual test stipulated in the Code and determine if such objectives can still be achieved without full compliance with these deemed-to-satisfy requirements for a specific structure."

A. Yes.

Q. "It is our opinion that permanent elongation test is for crack control for achieving the required durability performance in the Code."

Then if we can move on to the last page of the letter, on the top of the page:

"Likewise, cyclic tension-compression-compression test is to ensure structures will not fail under reversible extreme loading. If deemed-to-satisfy requirements of the Code cannot be complied with, the structure under study should be analysed under actual loading to determine if deviation from such compliance can be justified, subject again of course to the scrutiny of the Building Authority."

Now, we have looked at part of the details of this letter.

A. Yes.

Q. Earlier, I indicated that I am going to ask you why you have not taken into account the test result in determining what acceptance criteria should be adopted.

A. Right.

Q. You have looked at this letter. Would this letter contribute to your opinion?

A. Well, I tend to agree with this. So I believe there is a need to do more tests and then to come back to the principles of mechanics and to understand, well, how it is going to perform under different types of loading.

So I think, in general, I tend to agree, and there should be a lot more to do instead of just testing one sample, in particular that the results look very strange.

COMMISSIONER HANSFORD: Sorry, Prof Au, I've understood the point about your view that further coupling tests are

needed because we only have one test here and that's an extremely low sample.
A. Yes.
COMMISSIONER HANSFORD: And also that the results look strange to you.
A. Yes.
COMMISSIONER HANSFORD: Which perhaps would be another reason for having another sample.
A. Right.
COMMISSIONER HANSFORD: But are you suggesting different tests should be carried out?
A. No.
COMMISSIONER HANSFORD: You are not?
A. I'm still referring to the standard test.
COMMISSIONER HANSFORD: Thank you.
MR CHOW: Having received this letter, the Buildings Department wrote to MTR on 10 January. Are you aware of that?
A. No.
Q. I will now move on to another topic.
A. Right.
Q. You remember there's a joint expert memo signed --
A. Yes.
Q. -- among the experts, and paragraph 3 of the joint expert memo has been thoroughly discussed between you

and both Mr Pennicott and Mr Connor yesterday.
A. Right.
Q. Yesterday, we have also looked at the additional comments that you have made subsequent to that expert meeting.
A. Right.
Q. Paragraph 3 has also been addressed by you in the additional comments.
A. Right.
Q. Your additional comments can be found at bundle G20, page 15046, paragraph 3. Then turn over the page. You have two or three bullet points --
A. Right.
Q. -- dealing with paragraph 3 of the joint expert memo.
A. Right.
Q. What you have put in these additional comments -- basically, what you are saying is the internal stresses generated inside the joint have to be checked --
A. Right.
Q. -- numerically --
A. Yes.
Q. -- and it is premature to jump to any conclusion, in particular the adequacy of the joint?
A. Right.
Q. That is what you are trying to say in your additional

comments?
A. Yes.
Q. If I can now refer you back to the signed joint expert memo.
A. Right.
Q. Paragraph 3. I believe it's at the end of Mr McQuillan's report. Unfortunately I don't have a page number.
COMMISSIONER HANSFORD: Page 118.
MR CHOW: Thank you, Prof Hansford.
In paragraph 3, the last statement of paragraph 3, the last sentence, where it is put down:
"Notwithstanding, all agreed the outcome would not show the construction joint to be problematic."
A. Right.
Q. What you have put down in your additional comment, how would this reconcile with this last sentence?
A. Actually, I didn't agree with that, and I raised concern during the meeting. So I believe there is a need to check numerically.
Q. You mean you don't agree with the last sentence of paragraph 3, of the summary --
A. That is based on guesswork. I didn't hold a view that there shouldn't be any problem. We still need to wait and look at the outcome of checking.

Q. All right. Then I will move on to another matter, the diagram which you have drawn yesterday which is still on the whiteboard.
A. Yes.
Q. Yesterday, you tried to explain to us how you looked at the internal stresses by way of free body diagrams.
A. Right.
Q. When you explained what you drew on the whiteboard, at one point you mentioned about lack of lapping.
A. Yes.
Q. Can I ask you to clarify what lapping were you referring to at that time? Which thing is lapping?
A. I was referring to the lapping of some additional L-bar or U-bars with the vertical reinforcing bar, because with additional lapping, then the change in the force inside the vertical bars would be less abrupt.
    So right now, in this arrangement, at the top, here (indicating), of the vertical reinforcement, the stresses will be effectively zero. So there is a rapid drop in the stress, which means that there is very large bond stress, this kind of stress (demonstrating with fingers), and actually there is a certain possible critical shear pane, over here (indicating whiteboard). So we have to check. I think that is a concern.
Q. The U-bar you mention, I believe that we can all imagine

what you are talking about is an inverted U-bar?
A. Something like that (indicating whiteboard).
Q. Right. But the L-bar you have just mentioned, it may not be very clear what L-bar is referred to.
A. It may be something like that (indicating whiteboard).
Q. Okay. Thank you. Another topic. Paragraph 99 of Mr McQuillan's expert report, about the clamping action; do you recall that?
A. Yes.
Q. Where Prof McQuillan said because of the clamping action no shear can be generated at the new construction joint; do you recall that?
A. Yes.
Q. In response you said, in order to mobilise the clamping action, one has to do a post-tensioning; do you recall that?
A. Yes.
Q. What you said is, and I quite:
"So if we provide some tendons and do post-tensioning, then there will be clamping action, but if we just cast it in situ, there won't be any clamping action that is useful."
A. Correct.
Q. What I want to ask you is about the word "useful". Are you suggesting there may still be clamping action but

for some reason not useful?
A. Well, actually, if we don't do post-tensioning, there won't be any clamping action. There won't be any.
COMMISSIONER HANSFORD: Prof Au, my understanding from yesterday was you were suggesting post-tensioning as a possible remedial measure if the numerical calculations demonstrated that there was a problem.
A. I was referring to something differently. Now, when I talk about possible clamping action, I was talking about some horizontal tendons, like that (indicating whiteboard); okay?
COMMISSIONER HANSFORD: Yes.
A. So when I was talking about possible remedial works, I was talking about something vertical, something like that (indicating whiteboard), some bar anchors. So that would help to strengthen the joint.
COMMISSIONER HANSFORD: Sorry, but my question -- my understanding from yesterday was that post-tensioning would only be required if the numerical analysis demonstrated there was a problem.
A. Yes.
COMMISSIONER HANSFORD: Are you now saying that post-tensioning is definitely required?
A. No, no, no.
COMMISSIONER HANSFORD: You're not?

A. No.
COMMISSIONER HANSFORD: So my understanding is correct?
A. Yes.
COMMISSIONER HANSFORD: Thank you.
MR CHOW: Still on the question of clamping action.
A. Right.
Q. In Prof McQuillan's report, the professor has prepared a diagram at page 42 of his report, to explain the clamping action.
A. Right.
Q. I would like you to briefly refer to the diagram that you have drawn in your report.
A. Right.
Q. At page 12, please. Yes.
The figure 6.4.3.5.1 on page 12 also indicates the reinforcing details inside the joint.
A. Right.
Q. And also in the OTE down-stand structure as well.
A. Right.
Q. By reference to the reinforcing detail, I would now like to go to Mr McQuillan's diagram at page 42.
A. Right.
Q. Regarding the clamping action, can you tell us that the blue part of the structure which forms a cap above the diaphragm wall --

A. Right.
Q. -- is the blue part of the structure reinforced in such a way to enable it to perform or act as a clamp?
A. Well, I doubt if it can do this. Well, unless -- so, first of all, there may not be any natural clamping action. Now, actually, we are most concerned about the horizontal shear force in the additional construction joint. I think considering the so-called clamping action is unnecessary and it will just complicate the matter. We should focus on that.
    So if one would like to consider that and prove that it can serve the purpose, then please, do a calculation. So just looking at that, I don't think it can serve the purpose of reinforcing the shear resistance at the additional construction joint.
Q. Thank you. Yet another topic. Yesterday, Mr Connor asked you questions in relation to paragraph 6.4.3.6 of your report, in which you refer to section 3.8 of the Concrete Code.
A. Yes.
Q. Perhaps it's easier for you to look at paragraph 6.4.3.6 on page 13 of your report.
A. Right.
Q. Here you mentioned "the principles underpinning the design of ... beam-column joints as described in

section 6.8 of the Concrete Code".
A. Right.
Q. You said those principles should also apply for analysing the stress inside the connection.
A. Right.
Q. Do you recall that?
A. Yes.
Q. You mentioned about the book Park and Paulay; do you recall that?
A. Yes.
Q. In front of you, we have prepared two copies of the Park and Paulay. One we have already handed up to the Commission, and you can take a look -- because yesterday you have not mentioned the name of the book, you just mentioned the author.
A. Correct, yes.
Q. I just want you to take a look to see if this is the book you are referring to.
A. Yes.
Q. You also mentioned that the way we should calculate or analyse the stress inside the connection is explained in this book.
A. Yes.
Q. And you could even identify the chapters.
A. Yes.

Q. Perhaps for the sake of completeness and for the benefit of the Commission and the public at large, can you just, if you are able to, make reference to the relevant parts of the books so that we can make copies and perhaps insert it as one of the appendixes to your expert report?
A. Okay. Now, actually, section 13.8 is on beam column joints.
Q. Do you have a page number?
A. Page 716, starting from that page.
Q. I see. And it goes all the way to ...
A. It's very long, actually.
Q. It doesn't matter. We can make copies. As long as you identify the relevant part for the benefit of the Commission.
A. All the way until almost the end of the book. So it's an advanced topic, actually.
Q. So it's up to page 758?
A. Yes, correct.
Q. Thank you.
Yesterday, you also mentioned --
CHAIRMAN: Sorry, I hope I'm not going to be asked to read this, am I?
MR CHOW: No.
CHAIRMAN: I'd like to be told about it rather than -- I'm

happy if it's an audio book but not one that I actually have to read myself. I'm just wondering what the purpose is. Is it a general reference to support what Prof Au has said?
MR CHOW: Basically, because yesterday --
CHAIRMAN: It identifies his source?
MR CHOW: Yes, to support the way he analysed the problem, he sees how a stress should be determined inside the joint, because all along Prof Au has been using free body diagrams to explain what the proper way should be to look at the problem, and in fact my next question is -- at one point yesterday, Mr Connor cross-examined you on something and then you started talking about checking the internal stress.
A. Yes.
Q. You also mentioned about making use of free body diagrams --
A. Yes.
Q. -- to analyse; it's pretty common, you said.
A. Yes.
Q. The relevant part of the book in Park and Paulay, would it have covered also the use of free body diagrams?
A. Yes.
Q. So is it also included in part of your pages that you have just mentioned?

A. Oh, yes. For example, on page 727, it shows also a typical beam-column joint, and then in figure 13.58(a) -- now, the central rectangle is acted upon by a number of forces, and that is what I am talking about, free body diagram.

This free body diagram represents the entire joint, but to understand what happens inside, we have to look at other smaller free bodies and then try to understand what's going on inside.

Q. I see. So the method of using free body diagram to analyse internal stresses in any continuum material is not something that you invented yourself, it's something in basic engineering textbook; is that right?

A. This book was published in the year 1975, over 40 years ago.

Q. Thank you.

Yesterday, you were also asked by Mr Shieh for Leighton --

A. Yes.

Q. -- regarding the clamping action and also the cap at the top of the diaphragm wall, and you were referred to a diagram at page 28 of Mr Southward's report.

A. Right.

Yes.

Q. Thank you. Mr Shieh actually asked a very fair question

from a layman's point of view. Given that the diaphragm wall was being capped above by a new structure, how can it slide? Yesterday, you tried to explain that cracks will form at the new interface.
A. May form.
Q. May form.
A. Yes.
Q. If it fails, it may form.
A. Yes.
Q. Then, after the formation of the cracks on the construction joint, further cracks on each side of the diaphragm wall along the vertical direction may also develop.
A. Right.
Q. My question is that for a structure to be considered as starting to fail, does it have to slide physically?
A. Well, actually, this is just a postulated failure mechanism. There may be many possible failure mechanisms. Actually, in this case, what we should be careful with would be the internal behaviour of the joint, whether it is going to fail by other means. Because failure of the joint is brittle, it's very dangerous; we can't see it. So this is just one of the possible modes of failure that we need to address. There should be others that we need to check, see

whether there are problems.
CHAIRMAN: All right. And my understanding is that you have your postulation and there are other postulations --
A. Right.
CHAIRMAN: -- but you can satisfied that those postulations needn't worry us, if you conduct certain mathematical calculations as opposed to laboratory tests?
A. Both are possible, but of course laboratory tests would be very time-consuming.
CHAIRMAN: Yes, so mathematical calculations would, you think, satisfy you that these postulations in fact are not realistic?
A. Well, actually following the book by Park and Paulay, I think the last chapter or whatever, I think that should be the initial step. If the results show that the stresses are very low, there is no need to worry, fine. But then if the stresses are fairly high, then there is a need to look at what happens. And regarding the criteria, that would be difficult, because normally people won't check it afterwards, they normally start from something standard, and if they try to satisfy equilibrium at the very beginning, then normally there is no problem.
So, in that book, there are certain standard details. If people follow the details, normally the

connection will be okay, but if someone tries to omit something, wow, that would be a concern. To prove that it still works is very difficult. But then at least the simplified check is the first step.
CHAIRMAN: All right. Good. Thank you.
MR CHOW: Can I further ask this: if cracks that you have described yesterday develop, would it give rise to a safety concern?
A. Now, the problem is -- of course, yes, if a joint fails, it may fail by cracking and crushing of concrete. So the crushing of concrete is even more dangerous because it would be very brittle, so it would fail all of a sudden. So that's why the connection is something that is very important.
Q. Then, lastly, I would like to move on to a few areas that you have been cross-examined on this morning.
A. Right.
Q. This morning, when you were discussing with my learned friend Mr Boulding about the concept of robustness, you were asked by the Chairman that we are only talking about the change on top of the diaphragm wall now, and you said "yes".
A. Yes.
Q. But one of the other areas that we have to investigate in this Inquiry is the proper installation of the

couplers.
A. Yes.
Q. Would the quality of the installation of couplers go to the issue of robustness as well?
A. Let's say the amount of defective couplers is very high, then I think there is a concern, if it is very high. But so far I don't think it has reached that level yet, taking into account the amount of partial engagement, I think still there is a possibility of trying to assess the structural behaviour based on that. I think that is possible. Probably, that is a sensible thing to move ahead.
COMMISSIONER HANSFORD: Is it possible, Prof Au, to quantify what that level is?
A. Level of robustness?
COMMISSIONER HANSFORD: No. You said you don't think it's reached that level yet.
A. Okay. So you are referring to robustness. It's just based on impression. Just based on impression.
COMMISSIONER HANSFORD: I know. But is there anything more -- I mean, that's based on an impression.
A. Yes.
COMMISSIONER HANSFORD: But is there anything more definitive than just an impression?
A. Well, regarding robustness, it is difficult. It is

difficult.

So I referred earlier to a paper in -- well, that won me an award. Even in that paper, we classified the robustness into I think three categories or whatever. We couldn't quantify that. But then looking at the behaviour, we can have an idea.

But I think that so far the structure hasn't reached any serious concern of lack of robustness, so far.

COMMISSIONER HANSFORD: Okay. We'll take that. Thank you.

MR CHOW: Thank you.

Prof Au, do you recall that Mr Boulding also asked you about the NSL slab?

A. Right.

Q. And he suggested to you that the NSL slab sits on ground.

A. Right.

Q. I wonder whether you have had a chance to look at the Buildings Department's Mr Humphrey --

A. Humphrey Ho?

Q. -- Ho's second statement, in which he also talks about the NSL slab. Can I refer you to bundle H, page 40064. I'm sorry, I don't have the more detailed bundle number. I believe it's at the very end, the second statement, page 40064.

H20. Paragraph 32.

A. Yes.
Q. Mr Ho responds to Mr Aidan Rooney's statement, where Mr Rooney said "the NSL track slab is a ground-bearing slab with structural connections to the diaphragm walls at the east and west sides of the NSL track slab", and Mr Ho points out that "according to the accepted plans and the supporting calculations, the NSL track slab is a suspended slab supported on piles and also on the diaphragm walls at east side and west side respectively. Therefore, the NSL track slab is not a 'ground-bearing slab' as asserted by Mr Rooney."
    Can you recall having read that or you have never seen this before?
A. I have read that letter but not the whole thing. I have heard about that. I tend to agree with that, because if we ignore that situation, it would be dangerous. So that is a possibility anyway.
Q. In what way would it be dangerous?
A. Because if there is future dewatering, when the groundwater table drops below the NSL slab, then -- the soil may not be as stiff as the diaphragm walls and certainly there would be some downward loading acting on the slab. It is just prudent to design for this possible load case.
Q. Then you were further asked or suggested that the train

actually sits on the diaphragm wall and not on the slab.
Do you recall that?
A. Yes, I recall that.
CHAIRMAN: That's the EWL, East-West slab.
MR CHOW: Yes, the EWL slab. But from my recollection, it's also suggested the NSL slab is also in a similar situation, where train sits on --
A. No, no. The NSL, of course, if it's not directly on the diaphragm wall. I think even for the EWL slab -- well, there is a certain eccentricity, it's slightly offset. I think we have to take that into account.
Q. That must be my fault. My apologies.
Lastly, do you recall that this morning, Prof Hansford mentioned or indicated that the Buildings Department has witnessed the test carried out by BOSA.
A. Right.
Q. And the Buildings Department, if there is any objection, then should have raised it; right?
A. Yes.
Q. Do you recall that?
A. Right.
Q. As far as you know, what was the Buildings Department's involvement in that test?
A. I'm not aware of the details, but I think in this situation, the Buildings Department may not need to

object. They simply do carry out the test, whatever test, but later on whether they accept, that is another issue. But of course they can sign -- I mean, they can sort of verify that they are present. But then whether it is accepted, I don't know. That may be a different issue.

COMMISSIONER HANSFORD: Presumably, Prof Au, if the Buildings Department were present for the test, if they'd had any concerns about the test, they would have raised them?

A. I think if they were aware of that, I believe they would have raised, but then very often the witness may not be aware of everything. So later on, when they receive the report, they have to check. I think it would be fair for them to check everything, whether they can decide to accept or not.

COMMISSIONER HANSFORD: Exactly, and then at that stage, if they had concerns, to raise them?

A. Oh, yes.

MR CHOW: Thank you, Prof Au. I have no more questions for are.

WITNESS: Thank you.

MR CHOW: Thank you.

CHAIRMAN: Prof Au, thank you very much indeed. You have been of very great help to us. Thank you for preparing

your report and for the earlier work done. Thank you. Your evidence is now completed.
COMMISSIONER HANSFORD: We hope you make your lecture on time.
WITNESS: I think so. Thank you.
(The witness was released)
MR CHOW: Just one minor point. I understand that the two books we managed to obtain to show to Prof Au and also to the Commission are from the library.
CHAIRMAN: We noticed that. Don't worry. In fact we were looking at the last time it was taken out!
MR CHOW: We will ensure that copies of the relevant part will be provided to the Commission.
MR PENNICOTT: Very good. Sir, on a slightly more serious note, the sketch that Prof Au has prepared, what I was proposing was to ask Mr Ko to, as it were, remove it to the legal commission's lawyers' room. I will ask somebody in there to just annotate the fact that it was prepared by Prof Au on a particular day. It may be we can give a transcript reference, actually write it on there. Then we will get some photographs taken from it. Then we will bring the diagram back into the room, just in case we need to look at it again, if that's okay.
CHAIRMAN: That's excellent. Thank you very much.
MR PENNICOTT: Sir, on that basis, I think the next witness

is Prof Yeung, that's China Technology's expert.
CHAIRMAN: Good. Thank you.
MR SO: Chairman, with your leave, I call Prof Albert Yeung.

PROF YEUNG TAK CHUNG, ALBERT (affirmed)

Examination-in-chief by MR SO

Q. Mr Yeung, for the benefit of the Commission, can you kindly state your full name?
A. Tak Chung Albert Yeung, Y-E-U-N-G.
Q. Can you also state your professional address, please?
A. Department of civil engineering, University of Hong Kong, Pok Fu Lam, Hong Kong.
Q. I understand that you are now provided with a copy of your expert report. Can I take you to page 47 of your expert report, which is page 49 of the PDF file, of bundle ER1, tab 8. Prof Yeung, that's your signature?
A. Yes, it is.
Q. On the next page, you have also signed on the declaration that you give to this Commission.
A. Yes, it's correct.
Q. The expert report is dated 7 January 2019?
A. Correct.
Q. Do you confirm the facts stated in this expert report to be true?
A. Yes, I confirm.
Q. And insofar as opinion is concerned, do you confirm that

those opinions are honestly held by you?
A. Yes, I confirm.
Q. Prof Yeung, I understand that you have prepared a set of PowerPoint slides to assist this Commission in your oral synopsis.
A. Yes.
Q. Can I trouble you to now give your oral synopsis to the Commission?
A. How much time would I have before lunch?
Q. I am given to understand it's ten minutes.
MR PENNICOTT: Sir, that's a very fair question for Prof Albert Yeung to ask, because certain representations were made to me yesterday which I confess I had overlooked to draw to your attention.

It is a fact that when Prof Au gave his synopsis yesterday, it lasted just short of an hour, and of course that raised some questions from behind me as to what the other experts would be given in terms of time, because we had obviously indicated, as Mr So has rightly said, ten minutes.

Now, clearly I've had a very quick look at Prof Yeung's slides, which I think run to 24 slides, and it seems to me pretty obvious that that's not going to be a synopsis that's going to be accomplished in ten minutes.

Sir, I'm in your hands. It seems to me that Prof Au having been given a fair degree of latitude -- of course it did involve questions from yourself and Prof Hansford, so that was bound to extend it --

CHAIRMAN: And he was the first.

MR PENNICOTT: And he was the first.

CHAIRMAN: So he's ploughing a new furrow, so to speak.

MR PENNICOTT: Indeed.

CHAIRMAN: As far as laypersons like myself are concerned.

MR PENNICOTT: Yes, and if I may say so, I think it's fair enough, in the light of what happened yesterday, for Prof Yeung to ask the question as to how long he's got.

Sir, I'm in your hands. I think it's simply not going to be workable to limit this to ten minutes for, frankly, anybody, any of the experts.

CHAIRMAN: I agree.

MR PENNICOTT: Therefore a degree of latitude ought to be given. I do think we should make a start, if I may say so, before lunch, but I don't think -- the Chairman will obviously give directions -- Prof Yeung should feel constrained to just have ten minutes.

Sir, perhaps you could give some indication.

CHAIRMAN: Peter?

COMMISSIONER HANSFORD: We are in Prof Yeung's hands, but it seems to me that certainly the first section of your

presentation, Prof Yeung, is some basics and perhaps you can give us those basics before lunch, and then we can digest them and come back to more details after lunch.
A. Shall I give some background on myself first?
CHAIRMAN: Yes, we'd like you to do that. Thank you very much.
A. I went to the University of Hong Kong and graduated in 1982, so a year after Prof Au, with first class honours, and then I joined a consulting firm by the name of Binnie & Partners. For those in UK may be familiar with it. We are looking for some of the water treatment works, service reservoirs, and that's the very first time I got exposed to some sort of seismic design. Even though back in the 1980s it's not required in Hong Kong for normal domestic residential buildings, because water treatment works and services reservoirs are very important structures and from the government's standpoint, that is from the Water Supplies Department's standpoint, if anything happened to Hong Kong, any disaster, we cannot lose water supply because that will make things even worse.
So that's one example of how a specific organisation may impose those special requirements even though it's not mandatory.
I left the company in 1984 and went to the

University of California at Berkeley on a Rotary Foundation international scholarship to pursue my master's degree, and then afterwards I stayed in the University of California at Berkeley and worked under the famous professor James K Mitchell, who is one of the household names for those who like to work on ground improvement, soil behaviour and so, then I received my PhD in geotechnical engineering and geo-environmental engineering in 1990.

Afterwards I went to Boston and started my academic career there, Northeastern University, and at the same time also set up my own consulting business.

A year later, I moved down to Texas, to Texas A&M University, because a large university, also we have a state research institute in transportation, so I had chance to do full-scale experiment, like a simple case like a car crash, how would a car crash a barrier, how would we respond to it, and stayed in Texas for seven years or eight years.

My former boss at Binnie & Partners -- because at that point the company was acquired by an American company and my former supervisor ended up becoming the managing director of the company. So he called me up and said, "Do you want to return to Hong Kong?", because at that time he got a huge project in Lamma Island, thinking about a huge reclamation and the government has concern about dredging. So we are thinking about doing some sort of ground improvement, like one of the techniques we mentioned earlier, surcharging. That's why my former boss wanted me to come home, to be in charge of the project.

I came back to Hong Kong in 1998 and started work on that project and also some other projects, and also I worked for the KCRC looking at all the slopes from Hung Hom to Lo Wu.

In the year 2000, for those in Hong Kong may realise there's a huge piling scandal in Hong Kong. Two buildings in Tin Shui Wai got tilted, and that exceeded the requirements of the Housing Department. The Housing Department decided to rectify it. So I led my team with CM Wong & Associates and Prof Harry Poulos of Australia. The three teams worked together, we ended up rectifying the building. It's kind of like a world-class project, but that's a 41-storey tall building, 123 metres high, we needed to correct it from tilting, back to an acceptable standard. That's the chance I get underneath the foundation and so, but this is probably all the story I can tell you because the project remains confidential.

After that one, I started working on some of the expert witnessing cases for the company, also for ICAC, a number of clients, and afterwards I decided to change my career into the government and become Assistant Secretary for Financial Services and the Treasury. So that's one thing I declare in my report. At that time, Mr Frederick Ma, the Secretary for Financial Services and the Treasury, was -- technically we serve together, at the same time.

Afterwards, I returned to academics, that's why I joined the University of Hong Kong in 2003, at the same time also doing some of my private practice, working on different type of research projects, also on expert witnessing, consulting projects for contractors and so.

So, in a short run, that's basically what I have done in the last 30-something years. I do look into very difficult projects and also some of those like in the case we are talking about now, an underground structure, how an underground structure reacts with the soil and also the rock. So this is something we call the soil structure interaction. We are looking into the geologic material.

CHAIRMAN: Good. Thank you very much.

COMMISSIONER HANSFORD: Just a question, Prof Yeung. So your area of professional and academic expertise is

what?
A. Geotechnical engineering, geo-environmental engineering. At the same time now, I also start to work on something in the information technology business.
COMMISSIONER HANSFORD: Thank you. Okay.
CHAIRMAN: Yes. Perhaps we can start looking at your slides.
A. Okay. First is, those who are in engineering or professors, try to excuse me because some of the concepts may be very basic, but I think yesterday we got advised by Mr Shieh we should assume them to be a five-year-old intelligent kid and try to give them some idea what engineering is about.
So what I try to start is -- because we've been talking a lot now, since Prof Francis Au was in the witness box, about different types of stresses, internal stresses and so. I think many who are not in engineering probably get confused enough, how can we get internal stresses and so into a material? So I try to clarify that a little bit to make sure everybody is more or less on the same platform when we move forward.
Next slide, please. This one is, on the left-hand side, you can see a specimen, and then you can apply a force to it, say upward force and downward force. In engineering terms, engineering students trying to show

off to their girlfriends will call something very complicated called equilibrium and then they will throw up an equation, summation, or there's something equal to zero, but the concept itself is very simple. What we mean by equilibrium is the force pulling up equals the force pulling down, and the force pushing to the left equals the force pushing to the right.

So now you look at on the left-hand side is a complete specimen. That's exactly -- next to it is what Prof Au mentioned about a free body. So this one takes a little bit of imagination now. Suppose now you cut a part of that material in your mind, and that's the way now we can find out what are the internal stresses actually in the material. So from the outside you see the two forces, one is pulling up, one is pulling down, but what are the stresses? Really in the material we need to do something like what we show here as a free body diagram, cut it open by imagination, so you expose the internal stresses, and then these stresses also need to be equilibrium with the applied forces, and that's how you get those stresses.

What will fail material is these internal stresses, when they exceed a certain threshold. So on the left-hand side, what I try to show you is a tensile force, tensile stresses. On the right-hand side is what we mean by shear force now. You get a material, then you try -- on the top, you try to push it to the right, and the bottom try to put it to the left, and that becomes a shear. Then you cut a part of it, cut the top half of it, you will find out now inside this free body, you will have shear stresses. And shear is not too difficult to understand, it's just like you cutting your hair. You cut your hair, that's a shear. That's why sometimes a pair of scissors, we also call it a shear. So that's the way you cut that material. This is the basic concept in shear stresses and tensile stresses.

Next, please. Here we see a simple test, we try to find out what is the tensile strength of material, so you can easily see now this one is we try to climb on the top and clamp it at the bottom and then try to pull it. So similar to the slide you see in the previous one, you are pulling a force from the top, from the bottom, and then the tensile stress will be existing within this material. When the tensile stress, which tensile strength material, the material fails, so this is a simple concept about tensile failure, something we have been talking a lot now in this Commission, about a reinforcement bar, coupler assembly, so this is exactly what we are talking about.

Next, please. The next one is talking about some of the shear now. So on the left-hand side is more drastic type of things, it's kind of like in a fault line, when they start to slip, and that's one of the generations for an earthquake. Then on the right-hand side, there are two simple examples, they got tightened together by a bolt. On the top you see two plates tied by a bolt, and the bottom you see three plates tied together by a bolt. Now you can easily imagine you apply a force P to the left and a force to the right, so one force on the top plate, one force on the lower plate, you will introduce a shear force on the bolt itself.

So this is what I want to demonstrate to you what a shear force looks like. We will come back to this a little bit later when we talk about those bending, internal shear or a horizontal joint type of problem.

Next slide, please. The next one we try to talk about is what we call the stress-strain now. You can see from here now, as you are applying the stress to a material, the material will get longer. The concept is not that difficult to understand. It's like when we look at our two Commissioners sitting on the chair, when you sit on your chair, the first thing you should feel now is your cushion goes down a little bit, under your weight, your cushion starts to deform. Then by deforming that cushion, the load goes down to the chair.

So now the structure of the chair will feel the stresses inside it. Then eventually that loading will go down to the floor. So that's exactly, when you look at a structure, how the force gets transformed downwards. Everything will get deformed when you apply a load to it. Steel is no exception, as you can see from here now.

In the initial part, you will see as you apply a stress, your deformation starts to increase, so this part is more or less linear. So it depends on your material, looking at steel, looking at plastic, they may deform in a different way.

But then steel has a certain special characteristic, you can see from here now, if you keep increasing the stress, the strain will keep increasing, up to a certain point, you see in the drawing, called an upper yield strength. At this point you will find out the stress will start to drop. In engineering application, that's where we call the yield point. Yield point is one thing very important now, if you load something beyond the yield point, and then when you let go the load, it won't come back to the origin so you will create a permanent deformation. If you load up the steel and let go the load before you reach the yield point, it will come back to the origin. So something very important now is that's why in most design purposes, we don't want to design the steel beyond the yield point.

Then also on the same graph now you will see what is a tensile strength. So tensile strength is the maximum stress that your sample can sustain. At that point, you can easily see now that point goes well beyond the yield point. If you let go at some point before you reach the peak, the material will come back but with a certain permanent elongation that cannot be recovered. So that's what I'm talking about in those 0.1 mm requirement and so.

Next one, please. In terms of steel now, we got a number of different grades in engineering terms, we got a 460, 500, 500C, and what those numbers really are, those numbers are referring to the yield point. So that's what we call the upper yield point on the previous slide. At the same time, this one has a very, very important significance. We choose the number so that no more than 5 per cent of the sample we test will have a yield point less than that. That means, if I take 100 grade 460 specimen and I test all of them, no more than five can have a yield point less than 460.

COMMISSIONER HANSFORD: Prof Yeung, do we need to, for the purposes of this, understand the difference between 500B and 500C, or is that not really relevant?

A. They are different types of steel, but I think for this Commission we only talk about 500 and 460.
COMMISSIONER HANSFORD: Thank you.
A. And the next one comes up with some useful numbers I think this Commission has been seeing over and over again. So if you look at that 460, if you multiply that by 1.15, that's what you see all the time now about this 529. So 529 is increase 460 by 15 per cent.
What that really comes from is that comes from the CS2, Construction Standard of Hong Kong, back to 1995. So that's how we decide what the tensile strength is. From the yield point, you add another 15 per cent to it. So that's the old Construction Standard.
When you are looking at the problem we have in hand now, talking about ductility, talking about coupler assembly, if you look at the testing requirement, we need that coupler to have a tensile strength to be 25 per cent more than the yield strength. So that's why I showed you there are two numbers here, for the 460 steel, the tensile strength needs to be greater than 575MPa. If you are looking at 500 steel, you need to be greater than 625. So those are something very important when we move into talking about the ductility requirement.
Also in this particular Commission, we are talking

about 40mm bar all the time, so what I did is I take the stress, multiply by the area of a 40mm diameter bar and that's how we come up with the three numbers, 664.8 kilonewton, 722.6 kilonewton and 785.4 kilonewton. So those correspond to the tensile strength of the material.

So what I really means is a 40mm bar can take up so much load before it fails in tension.

COMMISSIONER HANSFORD: Prof Yeung, I can't remember if it's come up elsewhere in the Commission so far but can you tell me when Hong Kong changed from 460 to 500?

A. In fact, it's not only Hong Kong. The problem actually is if you look at the old CS2, the Construction Standard of Hong Kong, it was published in 1995, you find the 460 in it. If you look at the new one we are looking at for now, it's 2012, in that one you don't see 460 anymore now. So in between the certain evolvement, and what happened is, in the market, the manufacturer actually changed all the steel to 500, and simply because --

COMMISSIONER HANSFORD: When did they do that?

A. You are talking about now more than ten years ago, but then because the Hong Kong Code has not been changed into 500, so what happened is those manufacturers would not particularly make the steel for one small market like Hong Kong. So what they do is they are actually selling the 500 steel to Hong Kong and say it's 400 -- 460.

COMMISSIONER HANSFORD: So what you are telling us is, Prof Yeung, in the last ten years all the steel produced for Hong Kong -- in fact for everywhere -- has been 500?

A. That's correct.

COMMISSIONER HANSFORD: Thank you.

MR PENNICOTT: Sir, I will be asking Mr Yeung some questions about that particular topic a bit later.

COMMISSIONER HANSFORD: That's fine.

MR PENNICOTT: But I am bound to say that it would be very helpful to us if we actually knew -- and presumably Leighton apart from anybody else ought to be able to tell us -- what bar was used, both in respect of the bars for the diaphragm walls and the bars for the slabs.

COMMISSIONER HANSFORD: Yes.

MR PENNICOTT: I think as Prof Yeung has correctly identified, unfortunately, so far as the contract is concerned between the MTRC and Leighton, there is no doubt that it's the Code of Practice 2004, for concrete, and as Prof Yeung has just described it, the old Construction Standard, that is the CS2:1995, that as a matter of contract applies between MTR and Leighton.

Unfortunately, the work appears to have been carried out during this sort of transitional period that

Prof Yeung has made reference to, and it may be that despite the fact that all the contractual documents refer to 460, as a matter of fact something else may have happened.

COMMISSIONER HANSFORD: That's what I suspected.

MR PENNICOTT: So, sir, we are a little bit, I have to say, in the dark. One can make certain deductions from looking at certain documents that it must have been 500. But on the drawings, in the specification, in the bills of quantities, and by reference to CS2:1995, you will find the references to 460, not to 500.

COMMISSIONER HANSFORD: I understand that, but as I think you have just suggested, Mr Pennicott, Leighton, and presumably also Intrafor, ought to know what steel was delivered to them.

MR PENNICOTT: One would have hoped so.

COMMISSIONER HANSFORD: It would be quite useful, perhaps, for us to be advised of that.

MR PENNICOTT: Yes, sir. I don't want to belabour the point too much but you may recall in the evidence that there was an audit, the only audit done by the Buildings Department and Pypun, in January 2014.

COMMISSIONER HANSFORD: Yes.

MR PENNICOTT: It's interesting, if one looks at the results of that exercise, on the face of the documents you would have thought that a 460 bar or a series in fact, about 27 460 bars were tested. However, having discussed it with the Commission's expert, in terms of the results that were thrown up by that testing, you might conclude, despite the fact that it says 460, it was more likely to have been a 500 bar. But that's speculation on my part and just a deduction from the results that that document shows.

COMMISSIONER HANSFORD: Okay. What I'm unclear of at this point is how critical that is to the conclusions this Commission may be asked to reach.

MR PENNICOTT: I think there's one that's probably common, that if a 500 bar was used, that's stronger than a 460.

COMMISSIONER HANSFORD: I've got that. Indeed. Thank you.

MR PENNICOTT: Sorry, Professor.

A. That's fine. Indeed, to add a few points, you will find out Code of Practice normally evolves with time. So you can find out, if you look into the Code of Practice in 2004 or CS2:1995, when they try to test the coupler, you may not have those cyclic test and all those things. But then eventually you will find out now, in the QSP submitted by BOSA, they actually quote another standard in there, so that's what they call the AC133, if you look into that QSP. The AC133 practically is the later CS2:2012, because these things start to evolve and you see how people using a good thing, pretty much transfer that into your code. That's pretty much what's happening.

So you try to compare what BOSA is doing, they are actually doing most of the stuff according to the CS2:2012, rather than the 1995 version, simply because that additional requirement for AC133. Also, at the same time, in the QSP, BOSA also add in one particular requirement. It's the bar-break criteria. That means when you try to pull, you need to have the bar to break and not the coupler to break.

COMMISSIONER HANSFORD: But from what I've just heard from you, Prof Yeung, and indeed from Mr Pennicott as well, it seems that perhaps we've had a design with an expectation of one particular type of steel, but actually the construction was with a higher grade of steel. That appears to be the situation, and I just don't know how that affects the conclusions this Commission is going to be asked to reach.

But perhaps we'll leave that at the moment and perhaps the other experts might address that as well.

WITNESS: Okay.

CHAIRMAN: Yes. I'm thinking, Professor, it seems to me we've stopped for a brief discussion on matters and we are close to five past one now, so we might break for

lunch and then you know the spot you are in as far as you can launch yourself from that position when we return.
WITNESS: Okay.
CHAIRMAN: Good.
Although you are an expert witness, you are in the course of giving your testimony, and all witnesses, expert or not, are required, when they are in the middle of giving their testimony, to not discuss their testimony with anybody else; okay?
WITNESS: Fully understood.
CHAIRMAN: Good. Thank you very much. We will return then at 2.20. Thank you.
(1.03 pm)
(The luncheon adjournment)
(2.24 pm)
CHAIRMAN: Professor.
A. Okay. So I think these are the numbers we see all the time in this Commission, so this sort of explains where the factors come from, where the numbers come from.
Next slide, please. The next one we are looking at is what type of coupler is really required for this contract. This one is an MTR project, so MTR follow their own design standard manual, and the standards design manual states, starting from 2009, all the new

structures will be designed for seismicity, so that's why we need type II mechanical coupler. It's also stated in the QSP submitted by BOSA and then later submitted to the BD by MTRC. I think we have a lot of questions so far talking about what is mandatory, what is required. I think once you submit your drawing to the BD and once BD approve it, and then you apply for consent to commence work, that drawing becomes a legal document.

So I think by the Buildings Ordinance, chapter 123, you need to follow what is approved to construct whatever you need to construct, unless you want to submit amendment to it.

So once you get to that point, that becomes a requirement.

If you look at the QSP, the testing regime proposed by BOSA, and they also need to adopt the AC133, and that is where the 125 per cent comes from. It's not from the CS, because what we talked about, CS at that point, 1995, do not have that requirement. That's the reason why BOSA need to supplement the requirement by the AC133. As what I mentioned this morning, the AC133 requirement is technically the same as CS2:2012.

Next slide, please. The number of threads being engaged, I think we get a lot of discussion on this chart, and actually this chart I try to show all the data we have so far. So this chart may need a little bit more explanation.

The first thing we want you to look at is the solid circle. The solid circles basically are the calculation that we saw this morning, done by BOSA. So what BOSA did they assume a tensile strength of 529, as in the number I showed you earlier, it's 1.15 times 460. Based on that tensile strength, they deduced the shear strength of the threads. So the calculation they did is something -- a very simple scenario is -- they simply assumed the threads engaged each other, when you try to pull them, they just shear off all the teeth or all the threads. That's exactly how they calculate. So that's why they need the shear strength of the material.

That's exactly what you see now, from one turn, two turns, three turns, all the way to ten threads get engaged, and you see the straight line as what Prof Au mentioned this morning. It is simply a linear relationship between the available -- the tensile stress you can do in the bar to the number of threads engaged, but that may or may not be true, because BOSA simply put in a very simplistic model for the calculation.

Then now you can look at the open circle. The open circles are the experimental results from the five ones that we saw so far. So you can see now I try to adopt the same symbol you have seen in those reports, so "S" stands for slipout at four threads, and then the "C" stands for the failure in the coupler, and then the "B" stands for the fracture in the bar.

When you look at this circle, look at the first one with "S", it looks like it's very close to the theoretical calculation, but in fact now is, if you think about more detail, the solid circle is based on a tensile strength of 529, and this bar may actually be, as what we've been discussing so far, 500 with a tensile strength of 625. And if you use those numbers to recalculate, BOSA's calculate, those red dots should be a whole lot higher because you get a higher shear strength, so for each thread get engaged, they can take more stresses.

So from there onwards we can simply look at the experimental data, assuming they are all correct, and then you can already verify that the model they use for calculation is not really accurate.

The second thing you can see now is for the last one, and they purposely try to test the coupler strength, so they put in a very, very strong bar with a 900MPa, and this one, they test it, the coupler strength, they got 788-something. From this chart now, assuming everything is okay, you can still see now, for the three samples, that failed by fracture in the bar itself, the number fluctuates. And more important now, you can see now the two couplers, one coupler fracture at 6-something, the other one at 780-something. So that shows you now the variability of the material itself and so on.

If we move forward, try to look at the data in more detail, you will find out now there are more questions we want to ask.

Next slide, please. This is what I just talked about, this is what they assume and do all the calculations, and what we find out is the experimental data is actually smaller than what they calculate or in fact the threads are weaker than they assume in the calculations.

Next, please. When you look at the original report, a few things you should notice now is -- number one, you can still see the stamp "preliminary" so this is not finalised yet. And as we talked about this morning, CASTCO may be a reputable lab in Hong Kong and in Hong Kong, as all of us may realise, we've got a system called HOKLAS. So HOKLAS try to certify all the designs, but in all the reports now submitted by CASTCO we did not see their stamp it. So we are not quite sure what system they followed to run the tests and what's the procedure, how did they go along with that one, how did they need to report it, and also in the report I don't even see a picture of the failed sample.

COMMISSIONER HANSFORD: Sorry, I don't quite understand something you've just told us. You said the report is stamped "preliminary".

A. Yes.

COMMISSIONER HANSFORD: Is there an expectation we are going to receive a final report?

A. I think that's the normal practice, but so far I haven't seen a final report.

MR SO: Professor, if I can assist, the preliminary report actually turned up yesterday. It's in bundle H25, H44521, if we can take a look at the actual worksheet of it. They are consistent in five different worksheets for each sample, H44521 all the way to H44526.

COMMISSIONER HANSFORD: There's nothing on my screen yet.

MR PENNICOTT: H25/44521.

COMMISSIONER HANSFORD: Sorry for the interruption. I just wanted to understand this point.

A. Now --

COMMISSIONER HANSFORD: Hang on. We haven't got there yet.

MR PENNICOTT: The screens are all blank.

COMMISSIONER HANSFORD: The interruption continues.

MR SO: Professor, I don't want to lead you into give evidence, or give evidence myself over the bar table, but if you can take a look at the bottom of the sheet, can you explain yourself to the professor and to the Commission?
A. So this is a stamp we see now in the lower right-hand corner, "Preliminary report".
COMMISSIONER HANSFORD: Right.
A. Also on top now, you see the "Specified yield strength of bar", and there's no number recorded there. It was typed "900" and then it got crossed out and somebody initialled it.
COMMISSIONER HANSFORD: Okay. I understand what you're saying. I'm just puzzled as to why this is preliminary and just wondering whether there's an expectation of receiving something that's less preliminary. Maybe I'll just leave that hanging at the moment. Please --
MR SO: I can't assist in any way.
COMMISSIONER HANSFORD: Okay. Thank you. Please continue.
A. Also, if you look at the five sheets now you find out for those who try to run on a coupler, with normal running with different percentage of threads engaged and we don't see the strength of the bar, I think except for the very last page, there's an H44526, I think this is the one they purposely tried to put in a very strong bar

and tried to fail the coupler.

Here is another problem now I have. When they say the coupler failed, do they mean the body of coupler failure or they simply say the thread, the inside thread of the coupler failure? Because these bars are supposed to be very strong bars; I don't expect the thread of this bar would fail. So the failure mode will remain unknown and also now they should have a final report and typically they should have a picture of the failed sample so we can look at it to see what happened.

So these are some observations I make from this calculation and then also the lab data. The tensile strength of the test bar is unknown, so I probably assume it's something like 500MPa bar. Then, from this one, we can compare now, the strength of coupler assembly is lower than the calculated value.

So that shows one thing very important now is the model being used by BOSA for the calculation may be too simplistic. And also from here now you can see for the three bars that fail in the bar, it varies from 663 to 705. So it's quite a variability in the material.

Then more important now, you can look at the two couplers' failure, one is 630, one is 788. So you can also see now the huge variability, even for two samples.

If I can go back to the chart, so go back to slide 12, and by looking at this one now, even though you have no doubt on all the data, you will find out now, if I want to achieve 625MPa for a 500 bar, and you can see from here now is, you need more than six threads, even from this set of data, that we still do not have full confidence in. You can see the three Bs, they are all on the right side of "6".

Next slide, please. By looking at this data now, we have a couple of questions we need to ask. When you see the result, the first question I ask is: where do the samples come from? How representative are these samples to what we have constructed in the site? So there is no evidence or any indication where these samples come from. Then the number of samples, so far I have seen only one set of samples. So can we rely on -- because on this site we are talking about more than 20,000 couplers -- can we rely on one set of samples and try to make deduction on the behaviour of these 20,000 couplers?

Then this one is the standard of testing, we don't know, we don't know what's the testing protocol or maybe as the Chairman suggests now, they may try to do this in particular for the Commission, without thinking through all the detail. Because if we look at all the testing protocol, we spent years to develop them on every small detail. Also, we can look at the accuracy of reporting, as what I mentioned just now is, we don't even know what is the strength of the bar they are using and there's no document on how the sample fail except you get a letter B, a letter S. What do they mean by slipout? I still don't understand. Are they failing the threads or the bars simply slip out from the engagement, or have they really sheared the thread yet? So that will be a different failure mode that will shed more light on how we should do the calculations.

On the other hand is talking about how many threads we need is this letter we extract from 7 January, from the BOSA letter. So they put it here very clear, say:

"Please note further if rebars are not spliced butt-to-butt, the coupler assembly will be loose."

And also they try to answer a question by the Buildings Department and they say:

"Regarding your question on how a partially engaged coupler would perform in permanent elongation test [that is part of the AC133 test], static compression and tension tests and cyclic tension-and-compression tests, it is our opinion as explained in paragraph 4 above, that it is unlikely that such couplers, without being spliced butt-to-butt and are therefore loose, will survive permanent elongation, and cyclic tension-and-compression tests."

So although now BOSA has not done the tests yet, it's from their experience, and so they consider this one may not be able to satisfy the requirement for a type II mechanical coupler. Those are what we need for the cyclic tension/compression test and permanent elongation test.

But they did also make a statement there:

"... with sufficient partial engagement of threads, such couplers should survive static compression and tension tests in accordance with our design, subject to sufficient tests to be conducted for verification."

So even though they are not very confident on the small number of tests now, even though they show they might be able to survive the static compression and tension tests. So that's some of my observations from here.

Next slide, please. The next one I want to talk about is the measurement of embedment depth. If we look at the test we are using now is, we try to send in an ultrasound wave to the end of the bar and let it reflect, pick up a reflection. So actually what we measure is from the point of measurement to the end of the bar. We are not trying to measure how many threads get engaged. So that's one thing I think everybody should be clear about. We are measuring how long the bar should be embedded into the coupler, but we don't know how many threads are actually engaged, but then how much tensile force can be transmitted depends on how many threads get engaged. So this is something we may need to allow for certain allowance here.

Also, BOSA will say now we need ten full threads engagement for correct installation. In fact they also mention they need 40mm.

Next, please. So, at the same time now, they also mention, in their bars, they got a 2mm chamfer at the end. That means, at the chamfer location, they cannot start putting the thread. So basically, if they need ten threads, each one with a 4mm pitch, plus that 2mm chamfer, you need an embedment length of 42mm.

So, now, looking at the equipment they are using now, we are saying we've got a plus/minus 3mm. So currently the government try to accept 37, but when we accept 37 in the measurement, what we really mean is the actual embedment length is between 34 and 40. So you got a 50 per cent chance they are higher than 37, towards the 40mm, and at the same time they've got an equal 50 per cent chance they are less than 37 and acting towards 34.

But one thing very important now is there's no chance the embedment length is greater than 40, because you measure 37, you are ranging from 34 to 40. Even though it measures 40, there's still a 50 per cent chance it's less than 40.

COMMISSIONER HANSFORD: Sorry, Prof Yeung, how does that reconcile with your previous slide that said required embedment length 42 millimetres?

A. What I try to -- the point is if you want to get ten threads engaged, the actual embedment length needs to be 42.

COMMISSIONER HANSFORD: But it can't because the coupler -- if you have butt-to-butt connection --

A. You get 44 inside. The total length is 88, so each side gets 44.

COMMISSIONER HANSFORD: I understand.

A. But then coming to what we are measuring now, if we take 40, there's still a 50 per cent chance we are smaller than 40. So that's why I'm not supporting the idea of using 37. The 37, you can be as short as 34, and that's exactly what the plus or minus 3 means.

For the next slide, we are talking about the top of the connection between the diaphragm wall and the EWL slab, and for this picture I need to give credit to Mr Southward. I take this picture directly from his report. This report is very illustrative in the sense that you can see three different types of bars. When I went to engineering school the first thing I learned is to do engineering drawing, and the first thing I was told by my professor is even though you try to do a sketch, try to do things in scale. So I think Mr Southward may think the same way.

If you look at this one now, it's very interesting, if you look at the thickness of the diaphragm wall, it should be about 1.2 metres; we all know that. Then if you look at these bars now, they are probably a little bit more than 1 metre on one side and a little bit more than 1 metre on the other side. So one thing now I do not have evidence is: is this really the bar configuration? That means the bar is not really continuous but one bar with two lap lengths on the other side and then the steel from the EWL actually have a lap now with a bar sticking out from the diaphragm wall.

The second thing I'm looking at now are the blue bars. The blue bars are supposed to be vertical bars in the diaphragm wall, and my question is where do they stop? Are they stopped below this horizontal bar or they are stopped at the same level as the top horizontal it bar, as indicated in the drawing? I think this is the one thing we need to verify on site in the opening-up exercise.

That's one thing -- next slide, please -- if you look at this one now, the one on the right is currently what we open up, and that's where we see two bars. But at the same time, in that two bars, I don't see the vertical bar so far. So maybe that vertical bar -- or the vertical bar actually stops well below that horizontal bar.

At the same time now, on the proposed further opening-up location I show in this figure, you can see on the left that I colour yellow, and that's a very important thing, we need to see whether this bar is actually a straight through-bar instead of a lap at that location. I think about it is for good reason, because the original design is supposed to be a coupler there, they will hook up a lap bar and then the bar from the other side will lap right there. So, if the worker has already started cutting up all the bars, they might try to create a lap right there. So that's one thing we need to confirm now is, this is a really straight through-bar? The only way we can confirm it is try to open up the location shown in yellow in this figure, so we can look at that one to make sure there is no lapping of bar at that location, to confirm it's a through-bar.

At the same time, the current location still useful, that we can check up where did the vertical bar stop; do they stop below all the horizontal bars or they are really like this drawing and stopping on the top -- to the top of the structure of the diaphragm wall?

The other thing is we are talking about these construction joints and this picture is what I took from the proposal of MTRC. On the left-hand side, if you look at the original design, the diaphragm wall is right there and the EWL slab will key into the diaphragm wall full shear key, so it's not just connected to a construction joint. So this one, they already think about it in the original design.

Next slide, please. The next one is I try to explain quite a complicated concept, when we are talking about how the bending will induce shear stress inside a member. To make life simple, I try to look at a couple of cantilevered boards you see on the top. If you look at the three boards, they are in parallel to each other, they are horizontal. So you can look at this one now, look at each one of them, when they are horizontal, without any load, without any deformation, the top and the bottom are of the same length. You can see they are parallel lines, the top and the bottom, same length.

If you look at the lower left-hand corner, if you put a P on it, it will start to bend. Once they start to bend, you can easily see if these three boards are not tied together, you can see the second bar will try to extend a little bit out from the first one. The reason for that is when you start to bend that, for each bar on the top is under tension and the bottom is under compression, so the bottom is shorter and the top is longer, and that's why the top of the second one will be longer than the bottom of the first one.

Now, if you try to tie them together before you bend it, what's going to happen? What that would mean is, if you think about it from the second figure, that means the bottom of the first board will get lengthened a bit and then the top of the second board will get shortened a little bit, so that they've got the same length. So that's where the shear stress occurs. Through that shear stress, you change the length of the boards so they can bend together.

I think that is a good explanation for you to appreciate, if you get things like this, when they try to bend, that soon will happen, you get shear stress inside. If these three boards, when you start with, already are one board, the shear stress will be inside the board itself. But if you think about like in a construction joint, you got a weak spot in there, and that's where he we start to worry about whether the shear stress can be transmitted. I think that's what we are talking about why we need calculation for the construction joint in the diaphragm wall.

Another thing very important is, I think as Mr Shieh mentioned yesterday, a big chunk of concrete and that's not really a true picture of it, it's not a big chunk of concrete; those are actually three chunks of concrete. One thing very important I think we need to understand is when concrete hardens, that is a chemical process. That means once it's hardened, you can't add water to it and it will dissolve again because a chemical reaction is not reversible.

Once you form a construction joint, when you pour the next pour of concrete on it, in engineering terms, that new concrete may not bond to the old concrete, or in lawyers' terms they are not glued together, unless that surface, you need to do a special preparation on it, like you need to expose all the aggregate and so, so they can bond together, but you need a special treatment on it.

COMMISSIONER HANSFORD: Which is the normal process for a construction joint, isn't it?

A. Which is normal process for a construction joint, yes.

COMMISSIONER HANSFORD: Thank you.

A. So if you can see enough evidence that has been done, that may not be a problem, and otherwise you may try to do a check now, to see whether, if that shear strength of that layer gets reduced, can the structure still remain safe?

This is the last one, try to demonstrate the same concept to you, so when you get two stacked together, if the interlayer surface is not bonded together, you can see what happens on the left. If you get it bond together, you can see the led line there, and that's how you use the shear stress, to make sure the two will bond together and then they bend together and become a stronger element.

So it's the same concept as what I tried to explain in the previous slide and that's exactly why we are talking about when you try to bend an element, you get shear stress inside. I hope everybody got the idea. It's a simple illustration.

CHAIRMAN: Thank you very much.

MR SO: Prof Yeung, just before I pass the ball to another counsel, I wish to raise a last matter with you.

Can you be brought to OU314, please. Professor, this is the result of the opening-up up until 12 January 2019. I use this version because Prof Au was given this version yesterday.

A. Okay.

Q. Prof Yeung, what is your expert opinion in light of these opening-up results?

A. These results now, we can see from here, on the fourth column -- the fourth column says "Purpose". As you can see from the MTRC proposal, they did say about the opening-up for two purposes. Purpose number one is try to confirm the as-constructed detail, and purpose number two is try to confirm the workmanship and also to see whether some of the threaded bars have been cut or not.

You can look at this one now, to say the engagement length, most of them are less than 40. So what that means is it's less than the number recommended by BOSA, and also that means now you do not have that spliced butt-to-butt as required by BOSA.

So depending how many are here -- probably I don't have the calculator to do a calculation -- and also the second one you can do from this number, if you look at the second column -- sorry, the sixth column and the seventh column, that means the last two columns, you can actually deduce the total length of the threaded section of the bar. So because you know what's embedded inside and what's the number of threads exposed, and also we know the pitch of the thread is 4, 4mm. So by taking the number of threads exposed times four, you know what is the total length of thread outside the coupler.

Using that one, added to the one embedded in the coupler, you can find out what's the total length of the threaded section. Theoretically, there should be 44 to 48, according to BOSA, because they say design for 44, with 4mm tolerance possible, and then from here you can find out the total length to see how they fit in that range, and then you can see now whether the threaded section is shorter than the design, and there may be possibility it has been cut. But at this point I don't want to use that term because we cannot find the cut section to prove they actually cut, because it can also be a manufacturer defect, they did not make the thread long enough or whatsoever. But by looking at these numbers, we can check out whether the total length of the threaded section fits the specifications.

COMMISSIONER HANSFORD: Just on that last point, Prof Yeung, are you saying that there is any indication here of anything being cut?

A. May I refer to my analysis?

COMMISSIONER HANSFORD: My question is: can you see if there's any indication from these two columns as to whether there's anything cut?

A. I would say that shorter than 44. Whether they are cut or not, I don't know. If you look at --

COMMISSIONER HANSFORD: So you are saying they could be

shorter than 44, but then you have also told us about tolerances.
A. Yes.
COMMISSIONER HANSFORD: So my question is: can you see definitively whether anything here is cut?
MR SO: Perhaps, Prof Yeung, can I draw your attention to sample 48, for example.
A. Sample 48, okay.
Q. We saw here the engagement length is 33.98.
A. Yes.
Q. And the number of exposed threads is zero.
A. Zero.
Q. So, according to your expert analysis, what will be the conclusion?
A. This one, we measure 33.98; right? Let us be fair to the measurement. 33.98 is close to 34; right?
COMMISSIONER HANSFORD: In fact it's pretty difficult to measure 33.98, isn't it?
A. So let us say it's 34.
COMMISSIONER HANSFORD: Let's call it 34.
A. Then we understand, we may get an error of plus or minus 3mm; right? So, when you measure 34, the maximum length will be 37, if I give them all the benefit of the doubt; right? 37 is still 7mm shorter than 44. I'm not talking about the tolerance 48 and so. Let's take the

shortest possible length of the thread that we expect and give them the longest possible length as we measure. Then you are still 7mm short.

In my report, I try to do the analysis, what's the minimum possible length, the maximum possible length of the threaded section and what's the average.

COMMISSIONER HANSFORD: So is the answer to my question that on that particular sample, there is, in your view, a possibility of it being cut?

A. Yes.

COMMISSIONER HANSFORD: Thank you.

MR SO: Thank you. Prof Yeung, the remaining procedure would be like this. Counsel for the Commission will get to ask questions to you first, and counsel from other parties may or may not have questions for you. The Chairman and the professor would, when they deem fit, ask you questions. Please remain seated. Thank you.

Examination by MR PENNICOTT

MR PENNICOTT: Prof Yeung, good afternoon.

A. Good afternoon.

Q. As I think you know, my name is Ian Pennicott, I'm one of the counsel to the Commission. I know we've met before.

A. Yes.

Q. Thank you very much for coming along to give evidence to the Commission, and thank you for your report.

Prof Yeung, can you just for the record tell us when you were first contacted by or on behalf of China Technology to give evidence to the Commission?

A. Probably second week of December.

Q. Right. So between about 7th and 10th, 12th --

A. About 14th, because I got accepted by the Commission on the 14th.

Q. Well, you got accepted by the Commission. I know in your report you have this notion that there's an expert panel. Well, there isn't.

A. Okay.

Q. The Commission has, as you know, its own expert, in Prof McQuillan, and various parties have their experts as well, and you have been appointed by China Technology --

A. Mm-hmm.

Q. -- and you were obviously accepted by the Commission as an independent expert. Nobody is querying your independence, Prof Yeung.

Can I ask you this next. In your report, in a couple of places, you make reference to the fact that you have not seen certain photographs that were taken on the two visits that the experts made to the site. Do you recall that?

A. I have seen those on the 17th. I'm saying I did not see those on the 19th.
Q. Do you know why you haven't seen them? Were they not given to you, made available to you, by China Technology's solicitors?
A. China Technology passed the photographs of the 17th, some of the 17th photographs, to me. I'm not sure those they are all the photographs that we took on that day, because there was only one person taking all the photographs.
Q. I understand that. So far as the 19 December photographs, what about those? You haven't been given those at all?
A. No.
Q. For the record, they are photographs that have been made available to everybody, all the firms of solicitors acting for the parties.
But you haven't seen them; you still haven't seen them?
A. No.
Q. All right.
A. There are not too many on that day, because actually, to inspect that location, it's only Mr Wade and I climbed down to that air duct to look at it.
Q. All right, and Mr Wade took the photographs?

A. Yes. He was appointed by the Chairman to take the photographs.
Q. By Prof McQuillan?
A. That's right.
Q. All right. Can I just ask you a couple of questions about ductility.
A. Mm-hmm.
Q. In particular I wonder if you would be good enough to first of all look at Prof McQuillan's report. I assume you've had an opportunity of reading Prof McQuillan's report; is that right?
A. Very quickly.
Q. And what about the other reports, from Dr Glover and Mr Southward?
A. I did.
Q. Okay, good.

Could you therefore go, please, to Prof McQuillan's report. It's a similar point, or it's the same point in fact that I put to Prof Au yesterday -- I have no idea whether you were here at the time. If you go to paragraph 89 on page 38, please.
A. Yes.
Q. What Prof McQuillan says there is this:

"The following summary facts inform my opinion".

And for present purposes I'm just interested in 1 and 2. He says:

"There is no requirement for the structures to be specifically designed for seismicity provided the design is code-compliant in respect of the ductility and bottom steel continuity clauses."

I assume you agree with that?

A. I don't.

Q. What part of it do you not agree with?

A. I think, for seismicity, currently in Hong Kong we do not have a code requirement for it, but this one, for the MTR station, they are following the MTR standard manual themselves, and that's why they base on that to do the design, submit it to the Buildings Department and get approved, and that set of drawing become the legal document they are supposed to follow to construct.

Q. So you are relying on the material that was submitted by MTR to the Buildings Department. So as a matter of what was approved you say they had to follow those requirements?

A. Those are the choice of MTR for their design requirement and they submit it. Once you get approved and they apply for consent to commence work, that is legally binding.

Q. Okay, but can we just read the words very carefully here:

"There is no requirement for the structures to be specifically designed for seismicity ..."

Just pausing there, they didn't have to do the design is the point, but they did, to some extent?

A. That's exactly what I mentioned to this Commission this morning. When I started as a young engineer, designing a water treatment works in a service reservoir, it's WSD's choice that they think the structure is so important, they design for it. I think the same for this station. I think you need to imagine how many people will go through that station every day. The MTR's concern is not overconservative.

Q. Then the more important point is this, because it goes to the joint statement that was agreed, paragraph 2:

"The geometry of the connection between the EWL slab and the east D-wall, however, precludes any ductility. The structural 'plastic' deformation which might occur during seismic activity will develop lower down the D-wall. Ductile-grade couplers are not therefore required where used in the EWL slab to D-wall joint."

The point there, and it's the same point made by Dr Glover, is that if there's seismic activity, the D-wall, to put it rather bluntly, will go first, before the slab. Do you agree with that general proposition?

A. That will depend on the failure mode, and depend on the loading on the slab at that moment. There's a lot of different combinations. I'm not saying absolutely they are right or they are wrong, but other possibilities do exist.

Q. Okay. That's fine.

In your report, Prof Yeung, if we could just go to that, please, you spend some time looking at the Code of Practice for Concrete 2013 --

A. Mm-hmm.

Q. -- and CS2:2012. See, for example, paragraph 76 of your report on page 17. Do you see that, Prof Yeung?

A. You mean paragraph 76?

Q. Yes. You say in the last sentence there:

"... recommendations of CoP 2013 and CS2:2012 on reinforcement steel bars should be followed."

Do you see that?

A. Yes.

Q. As I understand it, you do accept, do you not, that so far as the contract between MTRC and Leighton is concerned, in fact the relevant code is the Code of Practice 2004, so far as this concrete is concerned, concrete structure is concerned, and it's CS2:1995 that is also applicable, as a matter of contract between those two parties; do you accept that?

A. Yes.

Q. There is therefore a necessity, can I suggest to you, that analysis, insofar as it's required, should take place by reference to those two documents, not the later documents, and I'm just wondering why it is that you seem to have focused very much on the later documents which actually, as a matter of contract -- and I'm not going to get into a debate with you about the contract -- don't actually apply.

A. The issue is what we discussed this morning, the steel they actually use on site are 500, and the requirement of grade 500 steel did not exist in the 2004 Code of Practice or the CS2:1995.

Q. But you can't, whatever might happen with the steel that's available, actually alter the contractual requirements for loading, for tensile strength, and so forth. You might have to interpret the contract in the light of the steel that's available, but it's not the other way around. You can't change the codes that you're referring to.

Again, as I say, I don't want to get into a debate with you, but I'm just concerned that I don't want to spend time asking you questions by reference to contract documents that simply don't apply. Do you understand?

A. I understand.

Q. You might say to me, I don't know if you do, "Well,

Mr Pennicott, in fact, whether you look at the 2004 or 2013 document, it doesn't make any difference." Is that your position?
A. They do.
Q. They do make a difference?
A. They do make a difference.
Q. Okay. As clearly does the CS2:1995 and the 2012?
A. Yes.
Q. Okay.
A. Also, if you look at the 2004, you may not find the requirement of couplers.
Q. Yes, exactly.
A. But then the problem is now, in this contract, they also require couplers.
Q. Yes.
A. And then, now we are going to come into what you have mentioned about a contractual problem. If you look at the coupler, where are we going to go? Then that's why they generate that QSP.
Q. I agree, and I don't have any problem with that, Prof Yeung. If you want to look at the QSP, which I accept has the requirement for couplers, and that's where it's generated, that's fine. I have no problem with that. But what I do have a problem with is looking at other documents that simply don't apply to the

contract. Do you understand?
A. I understand.
Q. And you are right, insofar as the QSP is concerned, that requires the couplers, the ductility couplers, because that -- it's required because that's what was submitted to the Buildings Department and approved?
A. Yes.
Q. And that is the basis upon which those ductility couplers were used?
A. And also those are basis also from the design standard manual of MTRC.
Q. Indeed. I accept that.
I think in your slides -- I think possibly this is the easiest way of dealing with this topic, Prof Yeung, if I may.
A. Mm-hmm.
Q. Could we look at the slide that has the graph on it with the BOSA-calculated -- yes, there we go -- I'm afraid I --
A. You are talking about 12?
Q. Is it number 12?
A. Yes.
Q. Thank you very much. That's very helpful.
In terms of the testing that BOSA did in conjunction with CASTCO -- and, as we understand it, witnessed by

the Buildings Department -- in November 2018, what documents have you seen in relation to those tests?
A. I've seen those preliminary report, that's six pages, and also looked at one of the letters from BOSA to the BD that includes pictures of the specimen before testing.
Q. Right. Have you seen, for example, the 84 photographs that were taken on the occasion of those tests being carried out?
A. No.
Q. Would you like to see them?
A. Certainly.
Q. Let's go to H25. As soon as I can find them, of course. If you go, please, to H25/44485. You've helpfully been given a hard copy, Prof Yeung, which will make life a bit easier for us.
Just flick through these photographs. They run for a number of pages. They run up to 44519, and there are, as I just indicated, 84 photographs. You haven't seen these before?
A. No, I haven't.
MR PENNICOTT: I'm sorry, sir, we are just catching up on the screen.
COMMISSIONER HANSFORD: Is it something you suggest we flick through on the screen?

MR PENNICOTT: If you haven't got the hard copy -- they are all there.
COMMISSIONER HANSFORD: We will just note that for the record.
MR PENNICOTT: Yes, thank you very much.
Just go, please -- because I think you indicated earlier, Prof Yeung -- one point you made was in relation to the coupler that was loaded to destruction --
A. Mm-hmm.
Q. -- you hadn't seen any photograph and you weren't sure precisely what had happened. I think that's right, is it?
A. Yes.
Q. If you go to 44518, do you see the bottom photograph there, "Destructive test coupler"?
A. Yes.
Q. And if you go to the next page, I think you've also got it open there, Prof Yeung, you can see again a coupler. It just seems to have sheared, broken, right down the middle; do you see it?
A. I think that's a tensile failure of the coupler body itself.
Q. Okay. That's right. We agree that's right. So you can now see the type of failure that occurred, from these

photographs?
A. Is this the one that they are using a 900MPa bar?
Q. I believe so, yes.
A. Or the other one, because we've got two coupler failures.
Q. It's the same.
A. You mean the two fail in the same row?
Q. There's just one to destruction, as I understand it.
A. I'm looking at the one with the 50 per cent of threads engaged.
Q. Sorry, can you just tell me where you're looking?
A. If you look at this page --
Q. Ah, the last page, the table.
A. -- you've got two with a coupler failure, and the one you have just shown me, which one is it, the last one or the second one?
Q. I understand it's the last one, Prof Yeung, yes. And I deduce that because if you go back to 44517 -- do you see that?
A. Okay.
Q. If you look at the test results, the bottom photograph, just pick up the figure of 990.41 kilonewtons; do you see that?
A. Yes.
Q. If you go to the table on page 44520, you will see that

was the tensile load applied to the last item or the last test?
A. Yes.
Q. All right. If we go, as it were, back in the photographs to 44514, we can see in the bottom photograph, this is the test that was carried out at 70 per cent of the thread; do you see that?
A. Yes.
Q. And as we know it was the bar that broke in that circumstance, and we've got a photograph of it there.
A. Yes.
Q. The point you were making earlier is that normally, on these types of tests, you would expect to see the bar, you would expect to see the photograph of the result, and so forth. So you would accept -- I know we haven't looked at all of them, but you would accept that that's in fact what happened and there is a proper record of this test or these tests?
A. Yes. I assume these are taken by the staff of BD.
Q. I frankly don't know. It looks as though -- it's got the Buildings Department logo at the top left-hand corner. I suspect you might be right.
But this is what you would expect to happen in terms of these tests, these sort of photographs illustrative of what tests were carried out?

A. And also the graph that you see, that should also go in the final report too.
Q. All right.
Prof Hansford, I think, asked you this earlier. So, from your perspective and from your experience of this type of test -- and you've pointed out to us, rightly, that the six sheets of paper have the words "Preliminary report" in the bottom right-hand corner -- you would expect to see some sort of final report, would you?
A. I do, after they check everything, they confirm the results and so, and with all this documentation in it.
Q. Right. Well, a couple of months have nearly gone by since these tests were taken, but I'm afraid I can't show you any final report. This is what we have to work with.
A. Does that mean then they follow the standard procedure, somebody will verify, check the result, before they use the final report?
Q. I'm afraid I can't answer your questions, Prof Yeung, particularly because I don't know the answer. All right.
A. Then that will cast doubt on the validity of the results.
Q. Well, we've got what we've got, Prof Yeung, and we have to make of it what we can.

The next point is this. Mr So took you to the results that have been coming out on an almost daily basis of the opening-up that has been carried out at the station and which you have witnessed some, a limited amount.
A. (Nodded head).
Q. Really the position is this, isn't it, Prof Yeung: if one just looks at the results and in particular focuses on the engagement length in the table -- do you want to have a look at it again?
A. Yes.
Q. We've actually got the very, very latest results. Mr So I think probably sensibly took you to the one that Prof Au looked at, and perhaps we will go back to that one. It's at 314 in the bundle, OU314.

In terms of compliance, in terms of working out how many of the tests comply or don't comply, fail or don't fail, it all depends upon your starting point. Your starting point, as I understand it, is essentially 40 millimetres.
A. Correct.
Q. The Highways Department/government appears to be content to take a figure of 37 millimetres.
A. That's what appears to be.
Q. Both of those figures, as we've seen with Prof Au,

ignore the strength tests that BOSA and CASTCO carried out and that we've just been looking at, in the sense that if you accept that at 60 per cent engagement you get a factor of safety of 1.14, that's strong enough, and therefore, as a matter of strength, if you can take 26 millimetres, then all bar two, as we've seen with Prof Au, of these samples pass. So it all depends on when your starting point is.

A. You make a very important point in terms of keyword. If you consider only strength and that is not BOSA mentioned in its letter of 7 January 2019 -- because they say when you are not butt-to-butt, you will not be able to pass the elongation test, will not be able to pass the cyclic test, although that's their opinion, they haven't tested it, they don't evidence to show either way.

Q. We can see that, and what they say will have to be weighed up along with all the other evidence as well, Prof Yeung.

But in terms of simply working out percentages, as I say, it depends on where you start?

A. It depends the criterion used.

Q. All right. I'm happy with that.

Lastly, Professor, from here -- in your report -- perhaps we could just look at this briefly -- you spend

quite a bit of time discussing the question of laps. Could I ask you, please, to go to paragraph 126 of your report. That's at page 39.
A. Yes.
Q. You say there:
"The confirmation that the top reinforcement steel bars are through-bars and not laps ..."
Where have you got this whole idea that there might be laps and not through-bars? Where does that come from?
A. If you look at the original design, here is a diaphragm wall, the very, very first original design by Atkins, there's supposed to be a bar in here with coupler at the end, and then what they do is they will screw in the bar here (demonstrating with hands), with enough lap length and then put another bar right next to it. So this is where the lap is, right in the location that I showed in figure 6 in the original design.
Q. Right.
A. So it depends on the progress on site, because when you prepare those bars, you starting the cutting of bar at the right length well ahead of time. So that's why I get a feel that may be the case. That's why I want to say, if we are opening up, trying to confirm this detail, if we do some opening like what I show in

figure 6, we can confirm this is a real through-bar and everybody is happy.
Q. Can you confirm that that evidence that you have just given comes from your analysis of the original design drawings and not from your client, Mr Jason Poon?
A. No, that's also from my experience of building diaphragm walls in Macau.
Q. I say that because advisedly, Prof Yeung, to give you the opportunity of dealing with it, Mr Poon, when he gave evidence many, many days ago, raised this question of the potential of laps.
A. Mm-hmm.
Q. What that led to was the MTRC producing some evidence and in particular a witness statement from a Mr Derek Ma. Have you read that witness statement?
A. No.
Q. In which he deals with this whole question of laps, saying that it simply didn't happen and that the through-bars were the through-bars. That's not evidence you've looked at? You haven't looked at that evidence?
A. You mean about Derek Ma's statement; right?
Q. Yes.
A. Did you see -- I haven't seen that statement. I haven't seen his witness statement. But do you see any, like, in this case, open up, you see the through-bar without

the lap? It's very simple, take out a 250 by 250 box.
Q. I understand what you are saying, Prof Yeung. I firstly want to find out where the whole notion of this came from, and now you've explained it and I understand that, but so far as the Commission is concerned it doesn't want to be chasing -- it's got enough on its plate to consider without having to look at matters which frankly I thought had been dealt with and finished, and since there wasn't any cross-examination of Mr Ma on that particular topic --
A. Because also --
Q. -- I rather thought we could move on, and I suspect the MTRC and others were thinking the same.
A. I also saw a similar picture in Mr Southward's report, but that one -- if you want to show a very long bar, that picture should show a very long bar and maybe put a cut line there, as what an engineer will do, to indicate it's a very long bar. But that one you can see now is more or less the one I'm talking about, there may be a lap for another bar on the EWL slab.
That's what I think, if we want to confirm the as-built condition, we can pick some opening in there and everybody is happy about it.
MR PENNICOTT: Understood.
Sir, I have no further questions. It's 3.30 so

perhaps that would be an appropriate moment, sir.
CHAIRMAN: Yes, certainly. 15 minutes.
(3.29 pm)
(A short adjournment)
(3.51 pm)
CHAIRMAN: Yes.
MR CONNOR: No questions from me, sir. Thank you.

Cross-examination by MR SHIEH

MR SHIEH: Good afternoon, Professor.
A. Good afternoon.
Q. A few areas to explore with you. Without the need to turn up any documents first, the works, the contract in this case, were contracted for by reference to grade 460?
A. Grade 460, yes.
Q. So using -- even if on the facts some grade 500 rebars were used, it is not required by the contract; you would take that, yes?
A. Yes.
Q. And the designs were done on the basis that grade 460 would be used; correct?
A. Yes.
Q. In your report, you refer to the Concrete Code 2004; do you remember that?
A. I refer to both, yes.

Q. Can I ask you to look at the code at H8/2818. That is the 2004 Concrete Code.
A. Correct.
Q. Can you look at clause 3.2.8.2. That's at page 2853.
A. Yes.
Q. If you want to see that this is indeed 3.2.8.2, look at the previous page, at the bottom, 3.2.8.2; do you see that?
A. Yes.
Q. Then over the page, at the top:
"the coupled bar assembly tensile strength should exceed 287.5 newtons per square millimetre for grade 250, and 483 newtons per square millimetre for grade 460."
Do you see that?
A. Yes.
Q. I know you have something to say as to whether or not this code were the only standard that applies, but assuming that we only look at the standard prescribed by this code, a tensile strength of 483 newtons per square millimetre for the coupled bar assembly would be enough?
A. You mean refer to table 3.3?
Q. No, the sentence above 3.3.
A. So table 3.3 gives us --
Q. It's not a table.

A. Yes, it's a table.
CHAIRMAN: Where's the table?
COMMISSIONER HANSFORD: It's a bullet.
MR SHIEH: The top of ...
A. Okay. Got it.
Q. According to this code and simply looking at this code, if we are using grade 460 steel rebars, then the requirement was that the coupled bar assembly need have a minimum tensile strength of 483 newtons per square millimetre; correct?
A. According to this code.
Q. Thank you. But you say this is not the only standard that applies, right, in your view?
A. That is what, in the QSP, I think MTR choose to put in additional requirement.
Q. I know.

Then we look at the QSP. I think the devolution of this point can be seen from your expert report, at paragraph 76. Let me see if I get you correctly. At paragraph 76, internal page 17, you say:

"Although grade 460 reinforcement steel might be adopted in the design of the Hung Hom Station Extension prior to 2013, all the reinforcement steel bars available in the Hong Kong market by 2013 is grade 500B ... Therefore, recommendations of CoP 2013 and

CS2:2012 on reinforcement steel bars should be followed."

Then you went on to say:

"In fact, BOSA ..., the supplier of type II mechanical couplers ... issued a clarification ... in response to a media report stating that the tensile strength of the coupler assembly manufactured by BOSA was not less than 625 megapascals in compliance with the requirements of the BD of the Hong Kong SAR government. The tensile strength requirement of the coupler assembly of 625MPa indicates that the characteristic strength of the reinforcement steel bar is 625 divided by 1.25 equals 500 megapascals."

The way in which you divided -- the reason why you divided that by 1.25 comes in later. I think it's by reference to the QSP.

A. The QSP.

Q. The QSP, yes. But let's go through that process of deriving how you get your theory from.

So you say that if we apply -- if we proceed on the basis that grade 500 rebars are used, then you say the coupler assembly should have a tensile strength of no less than 625 megapascals?

A. Correct.

Q. What I'm suggesting to you is this. If in fact grade

500 were used, it is an optional extra; it is a bonus. The project need not be built by reference to standards applicable for 500. Do you accept that?
A. Correct.
Q. Can I then invite you to look at your paragraph 85 at page 21. You say:
"Appendix A of the QSP made reference to [the Concrete Code 2004 [which was the one we looked at] ..."
A. Correct.
Q. "... as it referred to grade 460 ... However, it also made reference to [acceptance standard] 133 ... published by the International Code Council to supplement the deficiencies of the 2004 Code ..."
Yes?
A. Correct.
Q. So that's why I say, correct me if I am wrong, if you simply look at the Code 2004, you may get a certain number representing the minimum coupler tensile strength, but because of this route of appendix A of the QSP -- sorry, because of this route of the reference to AC133, it brought in what may be a higher standard. Is that a fair way of describing your view?
A. Also more tests.
Q. More tests?
A. I think you got a fair way to describe it, yes.

Q. You then set out at paragraph 86 your interpretation of AC133, and you appended AC133; yes?
A. Yes.
Q. Then you said at paragraph 87 that the table in AC133 is actually similar to or the same as the content of the subsequent code, the 2013 Code, 3.2.8.4.
A. Correct.
Q. So if we look at 3.2.8.4, which is set out in your next paragraph, paragraph 88 --
A. Yes.
Q. -- then we see a whole host of requirements and tests and standards, et cetera; yes?
A. Yes.
Q. If you look at the bottom of page 22 --
A. Mmm.
Q. -- basically, there are two subparagraphs where we get this concept of 125 per cent; yes?
A. Yes.
Q. It's really 3.2.8.4 subparagraphs (b) and (c); yes?
A. Yes.
Q. These both bring in the concept that the requisite tensile strength of the coupler assembly should be 1.25 times that --
A. The yield strength.
Q. -- of the yield strength of the bar used?

A. Yes.
Q. If you turn over the page, you then express your view at paragraph 89:
"... the coupler assemblies must develop in mean tension the greater of ..."
Then you gave two extreme figures, the upper end and the bottom end of some tensile strengths.
A. Because in appendix A, they also put in a requirement of bar-break first.
Q. Right.
A. So if on site they use a grade 500 bar and when they run the test, they need to make sure the bar will fail first. That's a requirement in the QSP.
Q. I have a rather simple mind, so let's use 500 times 1.25, you see, because 500 is grade 500, yield strength grade 500, times 1.25?
A. Correct.
Q. So that would give a theoretical required minimum tensile stress of the coupler assembly, if we apply those two paragraphs?
A. Correct. There are some leeway. You can have some sample less than that.
Q. Right. So that is why, again, at paragraph 91, at the bottom, you said, the third line from the bottom:
"As elaborated earlier, it is very likely that grade

500 reinforcement steel bars are being used for the construction of the Hung Hom Station Extension, resulting in the requirement of a minimum tensile strength of the coupler assembly of 625 megapascals ..."

Do you see that? That is why you used that figure of 625?

A. That's correct.

Q. But as a matter of minimum, if the contract only required 460, should the calculation be done on the basis only of 460 multiplied by 1.25?

A. But then, when they run the test -- if you put yourself in the place of the contractor, when you run the test, you assume it's a grade 460 bar, and then, when you run the test, you get another requirement the bar-break first, and then when you are running, running, actually you end up with your coupler fail first, if you use a weaker coupler, weaker than the bar. That's from a practical consideration, because you require to have a bar-break failure mode in the contract.

Q. Let's concentrate on the requirement of the coupler assembly strength, because that is the relevant quantity to look at when we look at the significance of how many threads you need to screw in. Do you understand?

A. I got your point.

Q. If we look at the question of the strength of the coupler assembly, would you accept that if the contract was contracted by reference to and designed by reference to using grade 460 rebars, then for the coupler assembly to be regarded as pass the contractual requirement, then it would be good if the coupler assembly were to achieve 460 times 1.25? Forget about bar-break first.

A. Forget about bar-break first?

Q. Forget about bar-break first.

A. Don't look at the real test first. On a theoretical basis, yes.

Q. Thank you. 460 multiplied by 1.25 would be 575?

A. Correct.

Q. The unit would be megapascals?

A. Yes.

Q. But of course I know you have done some calculations by reference not to 460 times 1.25. You have done calculations by reference to 500 times 1.25, which yielded the figure of 625.

A. Yes.

Q. May I then ask you to look at your report at paragraph 93. You refer to Mr Yim's statement in the MTR press conference, where he stated that "it was structurally adequate for the reinforcement steel bar to engage only six full threads of the coupler, ie 60 per cent of all the threads recommended by the manufacturer. His postulate fails in two aspects: (1) the tensile strength of the coupler assembly of 1,003 megapascals has not been proven experimentally ..."

1,003 megapascals would be assuming complete screwing in?

A. Yes, by the MTR.

Q. "... and (2) even if (1) can be proven experimentally and the tensile strength of the coupler assembly is proportional to the extent of engagement, the engagement of six threads is still inadequate to provide a tensile strength of 625 megapascals ..."

That is your thesis. If you set the required tensile strength higher, then you need more threads to be screwed in; yes? Do you accept that?

A. I say 60 per cent is not adequate, but as we've been talking all along today, and also sometime yesterday we close that out, the tensile strength is not the only factor you need to consider for the acceptance of the coupler assembly.

Q. I understand. You say there are other qualities or attributes which may have to be taken into account, elongation, et cetera. We'll debate that separately -- others may debate that separately. But let's focus on strength. In terms of strength the concept is the

higher the tensile strength that is required, contractually or by reference to whatever code, then the more number of threads will have to be screwed in; that's the concept, yes?
A. Not necessarily so, because if you use a stronger bar, the bar thread is also stronger, proportionally. So it doesn't matter you use which one to calculate. You may end up with the same number of engagement threads.
Q. Well, actually, if the contract is designed by reference to 460, and a certain set of numbers are arrived at as to how many threads you need, if I happen to use a better or stronger rebar, maybe I need to screw in less; right?
A. But then you fail the bar-break criteria in the test.
Q. Yes, but I'm focusing on the coupler assembly strength.
A. Theoretically -- yes.
Q. Let's look at some actual numbers; right? What you say is that by using your 625 megapascals as the requisite tensile strength required for the coupler assembly, six threads would not be enough?
A. Yes.
Q. Let's look at the BOSA table, at Prof McQuillan's report, page 84, internal page 84 of Prof McQuillan's report.
This is a table of BOSA, telling people its view,

basically, of the strength of its system.
A. The threads.
Q. Yes.
A. Based on a simplistic model.
Q. If you look at "Number of threads" -- yes, if you look at "Number of threads" on the left, 6 --
A. Yes.
Q. At six threads, and incidentally, the table here was by reference to using grade 460; do you accept that?
A. Yes.
Q. Because you can see that the steel bar specified tensile strength was 529. If you divide that 1.15, you get 460; correct?
A. But only 1.15, as you said.
Q. I know. But this was prepared on the basis that the bar used was grade 460?
A. But that won't satisfy the ductility of 1.25.
Q. I know. But this was prepared on the basis that -- it may not satisfy some other test, but in BOSA's mind, when it prepared this, it was preparing it on the basis that grade 460 would be used?
A. Correct.
Q. Now, look at "Number of threads". At six threads engaged, all the way to the right --
A. Counsel, I think you can rotate that page so it's not so

difficult to read.
Q. There's something wrong with my iPad because I tried rotating it and I couldn't, for some reason.
At six threads, it's 601.5 megapascals; yes?
A. Yes.
Q. So it doesn't reach 625, which is the minimum tensile stress, according to your method of calculation, because you need 500 times 1.25; yes?
A. That's correct.
Q. But let's give it one more thread, seven threads; that would be enough, yes? Correct?
A. According to this calculation.
Q. We can debate whether this calculation is correct until the cows come home, but I'm just testing something arithmetically. Assuming -- on the basis of BOSA's calculation or its own laboratory view of the quality of its couplers --
A. I would put it this way. This is one of, you may say, a numerical model put together by BOSA for calculation purposes. As what I show in my presentation, BOSA's presentation has already proved this table is not correct. If you still remember, what I present for that five data that BOSA has done already proved this table is overestimating.
Q. We can discuss the interpretation of that graph that you

have produced, but my question is a simple one. If we look at BOSA's own representation as to the strength of its couplers, six threads won't give you 625, but seven threads would give you enough; do you accept that?
A. According to this calculation.
Q. According to this calculation. Seven threads, on the basis of 4 millimetres per thread, would be 28 millimetres?
A. Correct, plus 2mm chamfer, so you get 30.
Q. I will come to chamfer now, if you want to talk about chamfer, because -- so that we actually know the dynamics of this, the higher the tensile strength, the more threads, in theory, that one has to screw in in order to reach that requisite tensile strength; yes? Do you accept that, all things being equal?
A. You need to look at the problem in a way that if you get a stronger bar, you always get stronger thread. You use a weaker bar, you get a weaker thread. Look at this calculation, you can see from here, if look at the fourth column, the shear strength is 264.5; right?
Q. I do need a new iPad because I kept shaking it and it wouldn't rotate.
A. Actually, you rotate the PDF.
Q. It doesn't matter. Just go ahead.
A. If you look at this one, look at the fourth column, you

get 264.5.
Q. Yes.
A. That comes from 50 per cent of the 529.
Q. Yes.
A. So if you use a higher tensile bar, that number will be increased at the same time; right?
Q. Yes.
A. So, if you check that out, you find out, for the same number of threads, it doesn't really matter what tensile strength you put in, for the same number of threads, you get the same factor of safety.
Q. I understand.
A. The problem is because this model by itself has been proven to be incorrect by their own experiment.
Q. I wouldn't debate with you whether it has been "proven to be incorrect", because that, I'm quite sure, will be taken up by others. But as I say, I'm a very simple person and I just want to say, simply looking at this, if seven threads would reach 625, then the practical implication is you would need 28. You would also add 2 to represent the chamfer, so that would mean 30; agree?
A. Agree.
Q. So let's look at chamfer. You say that according to BOSA, at the end of a thread there is a 2 millimetre chamfer?

A. Correct.
Q. A chamfer basically means that it's a sloped part which doesn't actually operate by engaging the thread; yes?
A. Yes, to make the engaging easier.
Q. So sometimes it would be in a sloped form, like a cone?
A. Actually, the chamfer is a 45 degree cut.
Q. Like a cone? It ends up like a cone, ice cream cone; agree?
A. Yes. Your geometry is good.
Q. Yes, it's a cone, an ice cream cone.
Look at bundle H25, page 44856.
A. Yes.
Q. It says "T2" on top. T2, type 2, would be ductile?
A. Yes.
Q. And then below that we have "type A" and "type B". Easily confuses everyone. Type 1/type 2 is non-ductile/ductile; type A/type B is short and long --
A. Yes.
Q. -- to put it bluntly?
A. Right.
Q. So T2 is the type of bars that we should be looking at for ductile; yes?
A. Correct.
Q. Looking at type A and type B, I need to be enlightened but I don't seem to see any sloping or protruding or

chamfer-looking feature at the threaded end.
A. If you look at the middle one.
Q. I'm looking at the top and the bottom.
A. Yes, but if you look at the middle one, that's why they make the chamfer first before they make the thread.
Q. Yes, but once the thread is done, you don't actually see any wastage of 2 millimetres.
A. No, but your thread cannot get on the chamfer.
Q. I will ask my question again. It may start off with the middle bar --
A. Yes.
Q. -- but once the threads are created on the bar and we look at the threaded rebar that's created, my point is that, look, the threaded bars in the form at the top and the bottom are the rebars that are screwed into the coupler; correct?
A. Correct.
Q. And your point of there being a 2 millimetre chamfer is that even though in theory there may be 2 millimetres there, but that 2 millimetres simply won't engage, so it won't have any threading effect, so you have to ignore 2 millimetres. That's your thesis; correct?
A. As you mentioned, when you make the chamfer, what will happen is -- this is original diameter (demonstrating with hands), right, and then you make the chamfer. Then

at this part, at the chamfer, the diameter actually gets smaller. As you say, it's a cone. Then your thread needs to have a certain constant diameter, so it must be on the straight part of the bar.

Q. My question is I don't see any cone under type A and type B here. These are the actual threaded bars used.

A. But if you start to count the threads -- because some of those actually may get into some of the so-called what they call the starting thread, may get into part of the chamfer and cover that one up, so you may not see it.

Q. But if you look at type A, for example, type A, if you look at the top part of type A, there is the beginning of the thread.

A. You look at where does it start? It doesn't start at the edge.

Q. At the top, left, top left-hand corner. I don't see any sloping or cone.

A. Because that sloping part actually comes into -- you look at the thread, you get a ridge on the top, right, and that one actually goes on that one.

UNIDENTIFIED SPEAKER: It's a physical item there. Maybe you can show us.

MR SHIEH: Can I just show it to the professor so he can show us where the chamfer is?

A. Do you want to see it? It's right here.

MR SHIEH: That's non-ductile. We need type 2.
MR CHEUK: This is type 2.
COMMISSIONER HANSFORD: You need to screw the piece out.
A. I need the project director of MTR. (Demonstrating screwing the coupler).
You see?
MR SHIEH: Can I see that?
A. That's the way you can make a screw, that will get in more easier, if there's more chamfer there.
MR CHEUK: It's very small.
MR SHIEH: It would obviously be a matter of interpretation. I think the most important persons to see that must be Mr Chairman and Prof Hansford. (Handed).
COMMISSIONER HANSFORD: I'm wondering if this 2 millimetres is actually significant.
MR SHIEH: I will be saying that it isn't, but since the witness has got it -- because I'm going to be take him to a table, because ultimately if we look at the opening-up table and see --
COMMISSIONER HANSFORD: I'm very happy to go here, it's just that we've spent a lot of time and I'm wondering if 2 millimetres has any significance.
MR SHIEH: The witness was keen to mention 2 millimetres of chamfer so I went straight to the chamfer.
COMMISSIONER HANSFORD: That's fine. I understand entirely.

A. Yes, this is a type B.
COMMISSIONER HANSFORD: Does anyone else want to see?
MR SHIEH: It's all right.
Professor, the significance of a chamfer is that if there indeed is a 2 millimetre chamfer which is sloped, then for the purpose of examining how many millimetres of embedment, we would need to take off 2 millimetres because those 2 millimetres of chamfer did not count as part of the engaged length; is that correct?
A. I think you may not need the whole 2mm, because some of the ridge of the thread actually takes up some of the chamfer, so you may not lose 2mm.
Q. At most 2?
A. At most 2, but I don't think you get 2.
Q. Let's say at most 2?
A. Yes, yes .
Q. Thank you. Let's look at the opening-up bundle at page 338. Other people have been looking at 315, I think. I don't know why I'm looking at 338. It's a more up-to-date one, I think, so I've been looking at 338.
COMMISSIONER HANSFORD: We may as well look at the most up-to-date one.
MR SHIEH: Up-to-date one, yes.
Now, Professor --

A. Yes.
Q. -- this is a table from the record of result of opening-up; do you see that?
A. Yes.
Q. As we discussed just now, I think between Mr Pennicott and you, whether you call something a pass or a no pass depends on what pass mark you apply?
A. Or the passing criterion.
Q. Or the passing criterion you apply.
We've been through the significance of the number of threads and the interpretation of the BOSA table, but what I'm now trying to do with you is to apply some numbers to the table and hopefully we will agree on the interpretation. If you look at the "engagement length" section, the second column from the right.
A. Yes.
Q. You see there are two single-digit items, which are items 5 and 22.
A. Correct.
Q. Those would fail, whether we use 30 or 24 or 26 or 28.
A. Or even 10.
Q. Those would fail. So those two, let's take them away.
A. Mm-hmm.
Q. Now, you can do the counting but I've done my own counting. Out of these 75 samples, if we apply 24,

being 6 times 4 -- do you understand?
A. Yes.
Q. Six threads with 4 millimetres each, if we apply 6 times 4 equals 24, if we use 24 as what I would call the structural -- the strength pass mark?
A. The tensile strength pass mark.
Q. Yes, the tensile strength pass mark, all pass except those two? You can count if you want.
A. If you use 24.
Q. If I use 24. If I use 26, ie the 24 plus the chamfer, it's still the same?
A. Yes.
Q. In fact, if I use 7, seven threads, 7 times 4 would be 28?
A. Correct.
Q. Even if I use seven threads, it would still be the same; only two fail. I have counted, but you can count yourself. There are a number of 28-point-something, but if you use 28, they pass. For example, at item 40.
A. Yes, 28.5.
Q. Item 40, if you use 28, it passes. So my point is even if you use 28, 7 threads times 4, all pass except two?
A. And you take the number as it is, without thinking about the error of measurement.
Q. I know, the 3 millimetres and all that, and maybe manual

error.
A. 3 millimetres, your 28.54 will fail if you move it to the other side.
Q. Yes, I know, 50 per cent both ways.
A. That's right.
Q. Let's leave all those to one side because if we start getting into those then we will get all kinds of different permutations. But we look at the number as is.
A. Okay.
Q. If you use 28, all pass except those two. If you apply 28 plus the 2 millimetre chamfer, then there would be three more fails?
A. Yes.
Q. And I'm telling you that that would be item number 2, because that's 29.65, that's below 30; there would be the two single-digit items, and there would be item number 40, which is 28.50, which would be below 30?
A. Yes.
Q. And also item number 72, which is 28.79, again below 30. All the rest would be above 30.
A. 30, yes.
Q. So, Professor, my point to you really is that even assuming that we use seven threads, even if we were to add the 2 millimetre chamfer at the highest, five out of

75 fail, looking at the matter as is; do you accept that?
A. That's 6 per cent; right?
Q. 6.666 per cent, and the pass rate, depending on how you put it, the pass rate would be 93.333?
A. If you look at Prof Lam's[?] calculation -- because your 6.66 per cent is in the sample, and then plus another error for the population, and then that was -- what I mean is out of that 20,000-something couplers, about 10 per cent failed, even using your method of calculation.
Q. Yes, but I have a number of different permutations.
A. Yes, I'm assuming your 30.
Q. You are assuming my 30?
A. After your 30, your 30 is 6.66 per cent in the sample, and then you need to plus a margin of error for the population, so you end up to be about 10 per cent.
Q. I understand. As to whether we pick seven threads or six threads, as to whether we add on the 2 millimetres of chamfer or only 1 millimetre, those are all things we don't need to debate, because others may discuss with you or they may have already been discussed. But what I'm trying to demonstrate is exactly why these things matter in the overall scheme of things. You move a bit of number here, you may change the pass rate; that's

what I'm trying to illustrate.
A. Okay.
Q. Finally, can I ask you to look at paragraph 100 of your expert report. There, you quoted:
"... Mr Frank Chan, Secretary for Transport and Housing emphasised in his response to the oral questions raised by the honourable Yiu Si Wing during the special meeting of the Panel ... that safety of absolute certainty was required for the operation of the railway. His exact wording was '我哋要確保鐵路運作係百分百安全.'"
For Mr Chairman and Prof Hansford, basically, literally, he said, "We have to guarantee that the operation of the railway is 100 per cent safe."
"In his opening remarks at the meeting of the subcommittee ... of the Panel ... Mr Frank Chan stated again repeatedly and clearly that safety is the top priority of the administration. In accordance with his statements, the actual embedment length should not be less than 40 millimetres. As a result, the measured embedment length should not be less than 43 millimetres to achieve the target -- safety of absolute certainty."
Did you write this yourself?
A. Yes.
Q. What has Frank Chan to do with structural engineering? I know he studied structural engineering, but apart from

that, what does Frank Chan and Yiu Si Wing have to do with structural engineering?
A. I think he is as a government official and Frank Chan is also an engineer himself and he understands the concept of safety and the concept of reliability. In fact I can tell you he studied electrical engineering.
Q. I know.
A. They talk about reliability probably more than civil engineers do.
Q. I know.
A. So that's his statement, if you want to make sure -- actually, in the next paragraph, I think that statement is too tough. If you want 100 per cent safety, there's no such thing.
CHAIRMAN: Sorry, the last sentence is a little ambiguous to me. Did Mr Frank Chan actually say -- did he quote the figure of 40 millimetres, or is this your deduction saying, "In accordance with what he has said, we must achieve this sort of figure"?
A. That's my deduction based on what he said, if I want to achieve 100 per cent absolute certainty for safety.
CHAIRMAN: Then it must be 40 millimetres plus 3.
A. That's right. Then we are sure we get 40.
CHAIRMAN: All right.
MR SHIEH: I'm going to go through this reasonably quickly.

Mr Chairman and Prof Hansford may know where I'm getting at. If it's simply a matter of your own expertise, calculation, strength, whatever, it shouldn't really depend on what a political minister said to a political body; right?

A. No. I think it's the level you want to achieve, and it's something very simple, just like you are an engineer and a layman talks to you saying, "I want to take the elevator, get to my floor in 18 seconds", you design accordingly. This client may know nothing about elevators but the engineer takes the instruction and moves forward. This is what you get from the minister, he says, "I want the railway operation to be 100 per cent safe."

Q. What if he said in a metaphorical way, "I want 200 per cent safe", you would then say everything failed because Frank Chan wanted 200 per cent?

A. Then you tell the minister that's not going to be achievable.

COMMISSIONER HANSFORD: (Unclear words).

CHAIRMAN: I think, from our point of view, that reading, using a little hyperbole, is simply this is a railway, it's a public utility, safety is a priority; it must be safe. If it's safe, it's safe. It doesn't become safer by being 127.5 per cent safe.

MR SHIEH: I'm going to my real point, which is statements of that nature, trawling through statements made by ministers in a political arena, only serves as sound bites. It has no place in a report of an expert character. Do you accept that?
A. No. That's why I moved into paragraph 101. I think that is something really too stringent. That's why I moved back to 40.
CHAIRMAN: Again, I'm interrupting -- I apologise -- from your perspective, this is what I would like to hear, from your perspective, as an engineer, as an expert in these things, what would be safe? Do you see what I mean? That I'm prepared to listen to, along with all the other evidence, as opposed to very natural hyperbole by a political figure in the light of public concern.
A. I got your point. That's why I would take 40. On the average, I know 40 means I can be between 37 and 43, but then on the average I got 40, and that also follows recommendation of the manufacturer. I would take the 37. If the real embedment is only 37, but the measurement shows 40, I would still take it.
MR SHIEH: Right. I suggest to you that 24 millimetres/six threads is already structurally safe. Do you accept that?
A. You only satisfy in your -- that's exactly what I've

been talking about all afternoon. That table was calculated by a simplistic model, and then BOSA has already demonstrated by their own laboratory work that method of calculation has overestimated the strength of the threads, as what I present in my graph.
Q. Lastly, did Mr Jason Poon give you that reference to Frank Chan and Yiu Si Wing?
A. No, actually, because if you know my background, I used to be an AO2, so I know how to track down those meetings. In fact, I know better than Jason Poon how to file all those things in LegCo.
Q. But you know he likes it?
A. I don't really care whether he likes it or not, I'm an independent expert.
MR SHIEH: No further questions.
MR SO: What is the sound bite?
CHAIRMAN: Yes, thank you.

Cross-examination by MR BOULDING

MR BOULDING: Good afternoon, Professor.
A. Good afternoon.
Q. I represent MTR, and whilst my friends have covered much of the ground I wanted to cover, there are just a couple of matters I would like to take up with you.

I'd like to stay, if I may, with the question of acceptance criterion of the embedded length in the

coupler. For that purpose, I wonder if we can go, please, in ER1, tab 8, that's your report, and if we can start at page 24.
A. Yes.
Q. This is the section where you deal with that, and going on to paragraph 96, you tell the Commission:
"The acceptance criterion recommended by the manufacturer is shown in figure 4. It can be deduced from figure 4 that the minimum embedment length is 40 millimetres and the minimum number of threads engaged should be ten."
I just want to see if I understand that. You're talking about figure 4 on page 26, are you not?
A. Correct.
Q. We can see what is said there by the manufacturer, BOSA; correct?
A. Correct.
Q. You'll know, won't you, that this guidance, this recommendation, was in play at the time the works were carried out; correct?
A. This is in their -- this is the recommendation, yes.
Q. Yes. It's the sort of thing that an inspector or a worker who wanted guidance, if he happened to have it in his back pocket, he might pull it out and have a look at it?

A. So they can look at it easily.
Q. Exactly. We know, do we not, that the rebar which has to go into the couplers has ten threads on it; correct?
A. 10 to 11.
Q. In fact, if you have a look at BOSA's table, H44527.1, this is a table Mr Shieh discussed with you, and you can see the BOSA calculation table there, can you not?
A. Yes.
Q. And the calculation stops at ten threads, does it not?
A. Yes.
Q. It was for that reason, amongst others, that I suggested to you, that it was ten threads that were on the end of the rebar to go into the coupler.
But in any event, let's look at this recommendation together. We can see, can we not, that we go from zero tolerance to what BOSA regard as their maximum tolerance; correct?
A. Correct.
Q. Looking at the summary:
"1. After connection has been fully tightened, one should see a maximum of two full threads to ensure a proper installation."
Correct?
A. Correct.
Q. That is shown in the photograph on the far right-hand

side of the recommendation; that's correct, isn't it?
A. Correct.
Q. So if one has a maximum of two threads as being acceptable, on the basis that there are ten threads on the rebar, that means, does it not, that eight threads have to be engaged?
A. No.
Q. Why is that?
A. Because this one, they are talking about the maximum tolerance. They manufacture with 44. Maximum tolerance will be 48, and that's why you get the two threads coming out. You still get 40 going in, because this total length is 88. So assuming both bars get the maximum tolerance, that means the threads on the two bars, both 48, the bottom one goes in 48, total length 88, instead of 40 on the top, so that's why still ten threads go in.
Q. I don't think that's correct, Professor. We've got ten threads on the bar; correct?
A. If they are maximum tolerance, they get 48mm long, so they got 12 threads, for those with maximum tolerance.
Q. That's not the situation. This is prepared on the basis of ten threads, and we have looked at the calculation prepared by BOSA --
A. How would this one relate to this one?

Q. It's the same manufacturer, for a start.
What I'm suggesting to you is that each of the threads has a pitch of 4 millimetres, doesn't it?
A. That's correct.
Q. If you've got an acceptable situation of two threads being exposed, that would give you 2 times 4 equals 8, would it not?
A. No.
Q. 2 times 4 four equals 8?
A. What I'm saying is this bar is not of 44 -- the length of the threaded section is not 44 with maximum tolerance. You are talking about the first one, the one on the most left.
Q. I'm talking about the one on the far right.
A. The one on the far right, each threaded section is not 44.
Q. No, it's 40. 10 times 4 equals 40.
A. What they mean by maximum tolerance is their threaded length is longer than the design.
Q. Well, with respect, they don't. What I suggest to you is what they mean is that with a rebar with ten threads on it, each thread is 4 millimetres, that's the pitch. If you have two threads exposed, which is acceptable, you've got eight engaged. 8 times 4 gives you 32 millimetres.

A. I would refer you to figure 3 in my report on page 25, for all those dimensions.
Q. Well, even in paragraph 95 you say that the number of full threads is between 10 and 11 -- well, I say 10 -- and at 10, I'm got to suggest to you that 10 times 4 would equal the 40.
A. Because we say the threaded section is 44mm.
Q. With respect, I've got to suggest to you that's simply incorrect.
A. And that's what is shown in the figure.
Q. Well, that's not what's shown on the BOSA calculation, though, is it? Because the BOSA calculation refers to there being ten threads. This was at H44527.1.
A. I think this is -- probably they are being conservative, they stopped at ten. But then in reality that's what -- the configuration of the threads are shown in figure 3 of my report on page 25.
Q. What I've got to suggest to you is the engagement is 32 millimetres and nothing like the 40 millimetres or even 43 millimetres that you are contending for.
A. So how long is the other one? How do you do a splice butt-to-butt then?
Q. Well, the butt-to-butt fact would not be a requirement during the course of the contract, would it? The butt-to-butt reference comes in in the letter of 2019;

that's correct, isn't it?
CHAIRMAN: Sorry to interrupt. Both of these have ten.
MR BOULDING: That's the basis of my question, sir.
And the butt-to-butt reference comes in, does it not, for the first time in 2019?
A. Can you check the manual of BOSA?
Q. Yes.
A. Because I was told by BOSA years ago it needs to be butt-to-butt.
Q. Well, the fact of the matter is that if you are inspecting or if you are a conscientious worker and you happen to have this recommendation in your report, in your back pocket, and you pull it out and you can see that there are two threads not engaged, you would say to yourself, "Job done."
A. No.
Q. "I don't have to go any further and check for butt-to-butt."
A. Unless you don't want to read point number 1, has to be fully tightened. If there's still gap inside there, you can still tighten it; it will still go in.
Q. So you say that fully tightened means butt-to-butt; is that your evidence?
A. If you fully tighten until butt-to-butt, your bar cannot go in anymore.

Q. All right. Anyway, my suggestion to you is that your contention that it ought to be 40 millimetres or 43 millimetres is simply nonsense.
A. I leave your opinion to you.
Q. There's one other matter that I would like to take up with you. That's paragraph 84.
Here, you are dealing with the strength of the coupler assembly, and you refer, do you not, in paragraph 84, to appendix A of the QSP?
A. Yes.
Q. You say that it provides that:
"The application is permitted for inter-storey columns provided that the following performance criteria are met".
Then you proceed, as I understand and read your report, to draw certain conclusions from that in the ensuing paragraphs; correct?
A. You mean in the following paragraph or --
Q. Yes, the following paragraphs, paragraphs 85 and 86 onwards.
A. Yes.
Q. Thank you. The situation, I suggest, is that this part of appendix A of the QSP, which is specifically for inter-storey columns does not apply here because it is intended to cover a situation, for example, where you've

got a multi-storey building where sway and other stress reversals would occur? That's correct, isn't it? It's intended -- the reference to inter-storey columns is intended to be a reference to a structure like a multi-storey building where sway and stress reversals would occur; that's correct, isn't it?

A. I think this is the only one in the QSP for the installation of coupler in this contract.

Q. Well, it might be, but obviously you don't follow it blindly; you've got to see what it applies to, haven't you? And if it says that the application is permitted for inter-storey columns, what I suggest is that it's not intended to cover the situation we had here which is a stiff underground box. That's what we are talking about here, isn't it, a stiff underground box? That's what the structure is, isn't it?

A. The diaphragm wall also functions as columns, actually. Think about that structural form.

Q. We've all seen the section through the structure, and what I have to suggest to you is that it's a stiff underground box which would not suffer from sway or indeed undergo stress reversals. Presumably that's something you would agree? You've got a stiff underground box here and it wouldn't experience sway or indeed undergo stress reversals; that's correct, isn't

it?
A. Where do you find that stress reversal?
Q. It's something I'm suggesting to you.
A. That's what you suggest. It's not in writing here.
Q. I know, but I'm suggesting that to you as a matter of engineering experience, engineering practice, that we are talking about a stiff underground box here, whereas appendix A of the QSP is referring to inter-storey columns, and that's something different from a stiff underground box; that's right, isn't it?
A. In what aspect?
Q. What is set out here, because it's directed at inter-storey columns, is inapplicable to the sort of structure we have here, which is a stiff underground box, which does not experience sway or undergo stress reversals.
A. I don't see why you say it doesn't under stress reversal, because the ductile coupler is mainly for seismic design for vibration and so. I don't see why you say there's no stress reversal.
Q. I'm saying there's no stress reversal in the structure we're talking about because it's a stiff underground box. So therefore this part of appendix A of the QSP which applies specifically to inter-storey columns is inapplicable.

A. I don't agree to that.
MR BOULDING: Thank you very much.
CHAIRMAN: Are there any further questions?
MR CONNOR: Nothing, sir.
MR CHOW: Yes.
CHAIRMAN: Could I ask how long you're likely to be, Mr Chow?
MR CHOW: Maybe ten minutes, sir.
MR PENNICOTT: I haven't publicised this but I appreciate you have a meeting this evening and that you wanted to go at 4.50.
CHAIRMAN: I was happy to leave it until the 5.00 full hour, but I'm a bit stuck after that.
MR PENNICOTT: Yes. I'm in your hands. I'm bound to say that whilst I have no intention whatsoever of preventing Mr Chow from asking some questions, the government have not given us notice that they wanted to cross-examine Prof Yeung so far as my table is concerned that I was given. So I did have that in mind as well, to allow Mr Boulding to continue.
MR CHOW: Can I just say this. We did indicate to Mr Pennicott that we did not intend to ask any questions arising from what Prof Yeung has put down in the report, but what I intend to do, I only have a few questions and it relates to the answers that Prof Yeung has given.

MR PENNICOTT: As I say, I'm not stopping him but I think in the circumstances perhaps we could come back tomorrow morning.

CHAIRMAN: Yes. I do apologise. I've got a meeting. It's one where I suspect I don't have to be there on the dot, but I do have to be there reasonably after the dot. So I think if we are now only about three minutes to five, we can close for the day.

Prof Yeung, I'm so sorry that we have to ask you to come back tomorrow.

WITNESS: It's fine with me.

CHAIRMAN: But we will start tomorrow again at 10.00. You are still in the course of giving your evidence and I sincerely trust that we don't have to detain you for too long. Thank you very much indeed.

WITNESS: Thank you.

MR SO: Before we adjourn, sorry I have to detain you for a while because I have instructions to make, an observation we have just had.

The observation is in regard to the exchange between Mr Shieh and professor at [draft] page 177 of the transcript, lines 1 to 14. I'm glad to understand and have assurance from Mr Pennicott of the Commission this morning that there were no objections and slightest doubt as to the independence of Prof Yeung towards China

Technology. But I just wish to clarify the Commission's stance or Leighton's stance as it stands now, because as what Mr Shieh indicated at [draft] page 176, he is going to a real point when he was cross-examining Prof Yeung, and I do not know whether it is just a slip of the tongue or whether it is an intention that Leighton now doubts the independence of Prof Yeung, and if not, it is our respectful submission that those exchanges simply could not help us to go anywhere and the implication that Prof Yeung is doing something or knowing whether Jason Poon likes a particular thing or not, and particularly this might bring Prof Yeung's independence and integrity into question. I simply want to know what's the position of the Commission and Leighton.

MR SHIEH: I'm not seeking to disqualify him on the ground of lack of independence, but these matters all go to weight, in the same way as the demeanour and attitude and all kinds of things occurring while a witness is giving evidence and in writing his report can be all matters that go to weight, but I'm not seeking to disqualify the professor as an expert.

CHAIRMAN: Was this related to Mr -- I think in the area of Mr Frank Chan?

MR SHIEH: Yes.

CHAIRMAN: That's very much a collateral area. It's got nothing to do with the expertise of Prof Yeung, which has not been challenged other than on the basis of professional challenge between experts as to what weight to be given.

The issue raised was, as Mr Shieh says, entirely oblique to that. It went, I suppose, to the matters that Mr Shieh himself has raised. I don't see that it's an issue.

I can understand you rising to your feet just to make sure. Certainly, as far as the Commission is concerned, we are satisfied absolutely in the professor's independence and his independence as an expert witness.

MR SO: I'm grateful for that. I just put that as a matter of record as to our stance.

CHAIRMAN: Good. And as a matter of record the answer is there.

Thank you very much indeed. Professor, tomorrow morning, I look forward to seeing you.

WITNESS: See you tomorrow.

(4.59 pm)

(The hearing adjourned until 10.00 am the following day)

INDEX