COI-1 Structural Engineering Expert Report

MTRCL Shatin to Central Link Contract 1112

by

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10 December 2019

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My Experience as a Structural Engineer

1. I am the Managing Director and Chairman of James Lau & Associates Limited.

2. Since my graduation from the Hong Kong Technical College in 1968, I have had over 50 years of experience in the field of civil / geotechnical / structural engineering, including in construction, design and research.

3. In 1968, I obtained a Hong Kong Bank Trustee Scholarship for 2 years' industrial training in the United Kingdom **("the UK")** as a graduate engineer. I then spent the next 7 years in the UK, studying (University of Manchester, 1971-1972), carrying out research (University of London, 1973-1977) and working for Redpath Dorman Long Ltd, then a subsidiary of British Steel Corporation and a leading UK steel fabricator. During the years with Redpath Dorman Long Ltd, I was a member of the group that constructed the Humber Bridge and the first Hong Kong Cross Harbour Tunnel.

4. I returned to Hong Kong in 1977. From 1977 to 1980, I worked as a soil engineer in the Buildings Ordinance Office of the then Public Works Department. During that period, I was transferred to the Geotechnical Control Office (later renamed the Geotechnical Engineering Office) in 1979 to work on geotechnical engineering matters. From 1980 to 1989, I worked in Wong & Ouyang (HK) Ltd first as their Chief Civil Engineer and later as the Director of Wong & Ouyang (Civil and Structure) Ltd.

5. At Wong and Ouyang, I was responsible for the design of many prestigious projects in Hong Kong and Asia. To name a few, the buildings included Pacific Place I and II; Bond Centre (now renamed Lippo Centre) and Far East Financial Centre.

6. In 1989, I started James Lau & Associates Ltd. **("JLA")** and Fong On Construction Group **("Fong On")**. I remain as the Chairman and Executive Director of the group of companies. On 16 October 2018, the group was listed on the Main Board of the Hong Kong Stock Exchange under the name of Shing Chi Holdings Ltd (Stock Code 1741). The holding company is now renamed Ri Ying Holding Ltd. For easy reference, I list out some of my more important and relevant experience in **Appendix JL1-A1** and my full curriculum vitae in

Appendix JL1-A2 of this report. **Appendix JL1-A3** contains some of the Hong Kong court cases in recent years in which I appeared as expert witness. I also appeared as expert witness in 21 arbitrations and 7 mediation cases. Due to confidentiality obligations I will not disclose details of these arbitration and mediation cases in the Appendix.

7. I obtained a PhD in geotechnical engineering from the University of London, King's College (1977); as well as a MSc in structural engineering from the University of Manchester, Institute of Science and Technology (1972). I am an Authorized Person (since 1983), a Registered Structural Engineer (since 1981) and a Registered Geotechnical Engineer (since 2005) in Hong Kong. JLA is the design arm of Fong On and is responsible for the design of the group's building projects. Since 1989, JLA and Fong On have been involved in many ground investigation, foundation, ground treatment, civil engineering, building (this includes renovation, alteration/addition works) and construction projects.

8. Since 1999, I was nominated by the Hong Kong Institution of Engineers **("HKIE")** as its representative to sit on various steering committees, overseeing the drafting of the first edition of the following codes of practice for Hong Kong: -

(a) Foundations;

(b) Code of Practice for Structural Use of Concrete 2004 (the "**Concrete Code**") **[H8/2818-3015]**;

(c) Code of Practice for Structural Use of Steel, 2005;

(d) Code of Practice for Precast Concrete Construction, 2003;

(e) Highway Slope Manual; and

(f) Code of Practice for Fire Safety in Buildings, 2011.

9. I am also appointed as an Adjunct Professor at the following universities in Hong Kong: -

(a) The University of Hong Kong;

(b) The Hong Kong University of Science and Technology;

(c) The Hong Kong Polytechnic University (2002 to 2017) (In 2017 I was appointed a Council Member of the University and I ceased to be an adjunct professor of the university);

(d) The City University of Hong Kong; and

(e) The Technological and Higher Education Institute, i.e. the Degree granting Institute of the Hong Kong Vocational Training Council.

10. I was Chairman of the Engineers Registration Board of Hong Kong (2004 to 2007); Vice President of the Institution of Structural Engineers in the UK (2002); and Chairman of the Structural Division of the Hong Kong Institution of Engineers (1998-1999). I am, at the moment, the Chairman of the Hong Kong Regional Group of the Institution of Structural Engineers.

11. I am a member of the Academy of Experts in the UK. I fully understand that I have a paramount duty to the Commission of Inquiry **("COI")**. I understand that I have a duty to be truthful, independent and impartial.

Appointment as Expert

12. I am appointed as an expert in structural engineering for the Government. I joined in the second stage of the Inquiry.

13. The need for expert opinion in structural engineering arose from defects and workmanship problems found in the Hung Hom Station Extension **("HUH'')** and the works in the nearby areas constructed by Leighton under Contract 1112 of the Shatin-Central Link Project. The job of the structural engineering experts is to assess the safety of the as-built structure, and whether it is fit for purpose, taking into account the effects of the defects. If necessary, the experts should also assess the need and extent of the suitable measures to be carried out on the structure.

14. Following the discovery of defects, a report was prepared by MTRCL entitled "Final Report on Holistic Assessment Strategy for the Hung Hom Station Extension" **("Holistic Report")** dated 18 July 2019 **[OU5/3229-3350]**. Based on the as-constructed conditions and taking into account the defects identified, MTRCL rechecked the structure using the original design assumptions and criteria **("Original Design"[1])**. MTRCL found that some areas of the structure would be overstressed. MTRCL then carried out a review on the original design assumptions. It is considered that a number of design assumptions and extra flexibilities / provisions can be rationalized as some of the uncertainties at the

[1] The "Original Design" in the Holistic Report was a scenario adopting the original design loadings over the original structural model including the defects and variations observed in Stages 1 and 2.

original design stage have either become more certain or no longer need to be accommodated. After review, MTRCL and its external consultants recommended a set of updated design assumptions and criteria **("Updated Design"[2])** as shown in Table 5 of the Holistic Report. When the as-built structure was checked against the Updated Design, the number of overstressed areas was reduced and the extent of suitable measures was ascertained.

15. At the time of my writing of this report, MTRCL has submitted detailed designs of the proposed suitable measures to the Government. The said designs were accepted in principle by the Government, subject to further modifications, if any, to address any comments the Government may have. The works are being carried out on site. According to the current work programme, by the time the COI hearing resumes in January 2020, some parts of the suitable measures will have been completed.

Relevant documents

16. I have been provided with the Holistic Report. I have also been given the Updated Design carried out by MTRCL, and the detailed designs of the suitable measures. I visited the site two times in September 2019, the first visit was with engineers from MTRCL and the Highways Department, and the second site visit was done jointly with the other structural engineering experts involved in the COI.

17. I have read the COI's Interim Report **[A2/711-879]**, the Holistic Report, the respective expert reports prepared by Professor Don McQuillan **[ER1/item 3]**, Professor Francis Au **[ER1/item 7]**, Dr. Mike Glover **[ER1/item 6]** and Mr. Nick Southward **[ER1/item 5]** in January 2019, and Mr. Southward's structural engineering expert report for the Original Inquiry dated 11 October 2019 **(Mr. Southward's "COI-1 report") [ER1/items 14.1-14.8]**. I have studied MTRCL's Updated Design and the supporting design calculations for the suitable measures proposed. A list of documents read by me is enclosed in **Appendix JL1-B**.

[2] The "Updated Design" in the Holistic Report was a scenario applying the variations of load assumptions agreed in Stage 3 structural assessment to the structural model of the as-built structure. The Updated Design takes into consideration of the defects and variations observed in Stages 1 and 2. Please refer to paragraph 4.14 to 4.15 of the Holistic Report for the narrative adopted for both scenarios.

Questions set by COI

18. On 12 October 2019, COI directed **[OU8/10561-10562]** that:-

    (a) the SE experts should focus on whether the as-constructed works are **safe** and **fit for purpose** from a SE perspective; and only if they are considered not safe or fit for purpose that such experts should then provide their opinion on whether the suitable measures (as agreed in the Holistic Report or Verification Report, or subsequently) are necessary for safety from a SE perspective; and

    (b) the SE experts shall not be required to look into the question of whether the suitable measures (as agreed in the Holistic Report or Verification Report, or subsequently) are required for statutory or code compliance.

19. I also understand that by an email dated 25 November 2019 **[OU9/10978-10979]**, COI further clarified that the aforesaid directions do not preclude any reference to relevant statutes or codes, in particular if such reference is necessary in order for the SE experts to explain and/or justify why they regard the structures are safe and fit for purpose.

20. As a practising structural engineer in Hong Kong, I have encountered incidents where various types of defects were identified during construction or after occupation of new buildings. For defects such as honeycombs, knowing that the defects have no major structural impacts, the structural engineer will normally instruct the contractor to rectify the defects without having to review or assess the structural adequacy of the rectified structure. However, for more serious defects that may affect the safety and stability of the structure, the first step is for the structural engineer to investigate the cause and the extent of the defects and then assess the effect of the defects on the structural adequacy of the structure. Depending on the nature and extent of the defects, the assessment invariably involves a review of the structural design and additional design calculations. If proved necessary, the structural engineer will propose additional works to be performed so as to compensate for the reduction in load bearing capacity caused by the defects. In the present situation, some serious defects were found at the HUH structure. MTRCL's consultants carried out a holistic study and design analysis based on both the Original Design and Updated Design. Based on the calculations performed, the consultants found that at some localized areas, suitable measures are required to be carried out.

In my view, the implementation of suitable measures at the HUH structure is in line with the usual practice of conducting defect rectification works for other buildings in Hong Kong.

21. Before I proceed to answer COI’s question (a), I must first explain what is meant by a structure being “**safe**” and “**fit for purpose**” from a structural engineer’s point of view. I believe that COI’s directions are intended to allow the experts to discuss the issues from a structural engineer’s point of view based on their own training and experience. Hence, the experts’ analysis on safety and fitness for purposes should not be determined purely by referring to the relevant design codes and/or statutory requirements. However, insofar as it is necessary to refer to the standards or requirements set out in any of the relevant design codes or statutory requirements in the analysis of safety and fitness for purpose, I should be allowed to and will do so. I note that Mr. Southward in his reports has also referred to the Concrete Code in his discussion on safety and fitness for purpose.

22. Obviously, different engineers, in view of their respective training and experience, may be inclined to adopt different approaches and look at the issues involved from different angles. However, in my experience and judgment, there are certain fundamental factors and parameters that one must look into before one can determine the “safety” and “fitness for purpose” of a structure.

23. Insofar as the relevant parameters have been reflected in the relevant codes, I will refer to the same in my report and I will then proceed to discuss the minimum factor of safety which should be adopted for present purposes.

Safety vs fitness for purpose

24. To start with, it is difficult to draw a neat distinction between “safety” and “fitness for purpose”. For example, if a structure is considered not safe, it can hardly be regarded as fit for purpose since one of the obvious purposes of building a structure is that it has to be safe for occupation or the intended use. On the other hand, if any of the conditions regarding fitness for purpose (which are safety related) cannot be fulfilled, the structure is also not safe. In the circumstances, there are bound to be some common elements between safety

and fitness for purpose. However, purely as an example, some aspects regarding "purpose" may have been required primarily from the view point of comfort / ease of use or from an aesthetic perspective, which may not be directly relevant to the issue of the structural integrity.

25. From a structural engineering perspective, a structure is considered safe and fit for purpose only when it is able to meet the criteria (which will be discussed below) during intended design working life (50 years for the Concrete Code and 120 years under MTRCL's New Works Design Standard Manual **("NWDSM'') [OU6/3753- 3920]**).

Safety

26. In my opinion, when one talks about "safety" of a structure, one must have regard to the following aspects and/or concepts:-

(a) **Stability** – whether there is overturning of structure or buckling of individual members under the worst combination of different types of design ultimate loads. A structure that does not meet the requirement regarding stability would risk catastrophic collapse or failure. I would consider the checking of stability of the structure as a **"Stability Check"**.

(b) **Rupture of Section** – whether there is rupture at any section of the structure under any of the design ultimate load cases. This may be caused by the overstressing of individual structural elements, even if the stability of the structure is maintained. Checking for possible overstressing at section(s) (which may lead to rupture) would be a **"Section Check"**.

(c) **Robustness** – robustness in a structure refers to the quality of or provisions in a structure that prevent collapse of major parts of it caused by accidental damage to a small area or element of the structure. An example was the progressive collapse of the Ronan Point Tower in East London in 1968 where a gas explosion blew out some load-bearing walls, causing the collapse of one entire corner of the building. The incident caused 4 deaths and injured 17.

(d) **Ductility** – the structure must be ductile to allow for redistribution of internal forces and bending moments in the structure and to cater for seismic loads for the purpose of preventing brittle failure and sudden collapse of structure.

27. The local code in Hong Kong (i.e. Concrete Code) and many other international codes adopt a limit state design approach setting out detailed requirements for the above design and performance criteria. A limit state is defined as the state beyond which the structure no longer fulfills the relevant design or performance criteria. The structure should be designed to have acceptable level of probability that it will not reach a limit state.

28. Limit state design method comprises ultimate limit state **("ULS")** and serviceability limit state **("SLS")**. ULS is the state that associated with collapse or with other similar forms of structural failure. It is related to the safety of people and the structure, and therefore is concerned about the stability, strength, robustness, ductility and durability. SLS is related to the functioning of the structure under normal use, comfort of users and appearance of the structure, and therefore is concerned about deformation, fire resistance, cracking and vibration. Failure to meet some criteria for SLS may also eventually reach ULS (e.g. persistent and serious cracking would have durability concern, inadequate concrete cover to provide the required fire resistance protection would have stability or strength concern, etc.).

29. To achieve acceptable level of probability of not reaching a limit state, the structure should be designed to be able to resist the actions resulting from the design loading (which is the anticipated future loads acting on the structure with the application of partial safety factors) based on the material strength of the structure (which is the design strength reduced by the application of partial safety factors for different materials concerned). By applying such suitable safety factors to the loads and material strength (which may be slightly different between design codes of different countries due to differences in local conditions and society expectation), a structure would be designed to achieve the required safety level (or safety margin) and fitness for purpose in respect of the above design and performance criteria.

30. If one is only focusing on "stability", I agree that the as-built HUH structure is safe. **However**, apart from the question of strength (which appeared to be the main focus of Mr. Southward), the other requirements of serviceability (i.e. functioning of the structure under normal usage), robustness, durability and ductility also have a material impact on the question of safety. In this regard, the broader question of structural safety is not only a question of ensuring that the structure is able to take up the designed loads or a matter of degree in a physical sense. It also includes the question of providing sufficient safeguard against accidents and unforeseen conditions during its design life.

31. In this regard, an important distinction is made between (a) stability and (b) rupture of section referred to above. The former concerns the risk of catastrophic collapse of the structure. The latter concerns the risk of local overstressing of individual structural elements without the risk of causing stability problems. In both cases, safety implies a minimum factor of safety against collapse or overstressing elements of the structure (i.e. the risk of collapse or overstress).

32. Regarding the question as to what should be the minimum factor of safety that has to be allowed for from a structural engineering perspective, no structural engineer would consider a structure to be sufficiently safe if the structure only provides a factor of safety of one[3]. The structure must provide a higher factor of safety against failure or overstressing of structural elements. In my opinion, unlike the setting out of fundamental parameters for assessing the question of safety, the determination of the applicable "minimum factor of safety" varies from one place to another and it would be difficult to rely on one expert's opinion to set out the relevant standards. It should represent society's general expectation of how "safe" structures erected in that place should be. It must be a value accepted by a large number of experienced structural engineers practising in a particular location.

33. In my view, whilst there are parameters for determining the questions of safety and fitness for purpose from a structural engineering point of view, it is only

[3] Factor of safety can be taken as the ratio between the yield stress (i.e. how much the structural part under consideration is able to withstand) and the working stress (i.e. the actual level of loading the structural part in question is required to withstand under normal working condition). A safety factor of 1 means that the structure can only support the design load and no more. Any accidental increase in load on the structure would cause it to fail.

appropriate to adopt the minimum factor of safety prescribed in the relevant building design codes in Hong Kong.

34. I was a member of the steering committee overseeing the drafting of the Hong Kong Concrete Code in the early 2000s. According to my recollection, various stakeholders[4] were consulted before the Concrete Code was promulgated. Views were gathered on the appropriate factor of safety in view of the circumstances in Hong Kong. After months of detailed consultations, discussions and revisions, a set of minimum factors of safety was adopted in the applicable codes and it has since been codified in the Concrete Code **[H8/2818-3015]**. In other words, the set of factors of safety adopted in the Concrete Code represents the Hong Kong community's expectation of the minimum level of safety required of buildings in Hong Kong. In my opinion, it follows that it is only appropriate to adopt the set of minimum factors of safety as stipulated in the Concrete Code in evaluating the safety of the HUH structure. In the circumstances, I tend to think that for the present discussion, reference to any other standards is arbitrary. It is because (a) this is not a forum for any debate on whether the applicable codes in Hong Kong should be subject to review; and (b) without setting out the relevant parameters for the purpose of assessing safety and fitness for purpose, any dogmatic reference to other codes or standards adopted in other countries will not lead to any fruitful discussion.

35. For example, in the case of material strengths (concrete and steel strengths), the designer and experts in Hong Kong are required to adopt material strengths that are stated in the Concrete Code. The designers have no idea what other material strengths they can use apart from those specified in the Concrete Code. Furthermore, the applied loads to be used on the HUH structure should come from NWDSM of MTRCL. The structure in question is a train station. It is unique. The designers and the experts have no other choices but to adopt the train and other loads that are specified in NWDSM. Likewise, there is a very specific requirement that the structure must be designed against seismic loads. At the moment, there are no seismic design requirements for buildings (apart from highway structures) in Hong Kong. The designers and experts can only adopt the seismic loads specified in NWDSM. For the present analysis, it is important for experts to refer to one set of standards in considering the level of

[4] Under the direction of a steering committee comprising representatives of relevant stakeholder organisations, professional institutions, academia and relevant government departments.

minimum factor of safety in order to ensure the elements of consistency and certainty.

36. **Partial safety factors and Ultimate Limit State Design –** In line with the international practice of limit state design for structures, the factor of safety in the Concrete Code comprises two components, one component deals with loads (called partial safety factors for loads) and the other for material strengths (called partial safety factors for material strengths). A structure is considered safe with an acceptable minimum factor of safety when it is checked satisfactorily against loads and material strengths with the application of their respective partial safety factors. This particular check is called the check against ULS. A structure can only be considered safe when it is checked satisfactorily against ULS with the corresponding partial safety factors.

37. When a structural engineer checks the structure against ULS, he should carry out distinct checks in respect of the following fundamental aspects, which I have outlined above, namely (a) stability; (b) rupture of section; (c) robustness; (d) ductility (detailing); and (e) durability. This demonstrates that insofar as the fundamental aspects of safety are concerned, the parameters which should be taken into account from a structural engineer's point of view have, to a substantial extent, been reflected in the applicable codes.

Fit for Purpose

38. As I mentioned above, the fundamental parameters for "safety" and "fitness for purpose" overlap to a significant extent. A structure that is not safe can hardly be described as being fit for purpose.

39. That said, there are other factors in a structural engineer's consideration of whether a structure is "fit for purpose". The structural engineer should keep in mind the intended usage or function of a structure. This is the consideration of serviceability under the serviceability limit state design approach. The factors to be considered are: -

(a) **Durability** – A durable structure must meet the requirements of strength and stability throughout its intended design working life without significant loss of utility or excessive unforeseen/unusual maintenance. The structure must be designed and constructed to protect the embedded reinforcements from corrosion. It must be able to perform satisfactorily in the working environment for the design working life of the structure. The concrete cover for reinforcement in particular is very important to ensure durability. New concrete is alkaline in nature. In this condition, the rebars inside the concrete will not corrode. However, carbon dioxide in the atmosphere in mixing with moisture becomes weak carbonic acid. The weak carbonic acid will diffuse gradually into the concrete through the concrete cover turning the concrete into a carbonated zone. The result is that the carbonated zone in the concrete is no longer alkaline and the rebars lose the protection against corrosion.

(b) **Deformation** – This includes deflections due to applied loads. The structural engineer always wants to limit the deflections of structural members to make sure that the users do not feel uncomfortable whilst using the structure; that the deflection will not affect or cause cracks to finishes, partitions, glazing and claddings. The deflection will not affect the proper functioning of services.

(c) **Fire Resistance** – The structure and its structural elements must be designed to possess an appropriate degree of resistance to flame penetration, heat transmission and collapse due to fire.

(d) **Cracking** – Concrete is brittle and weak in tension. Cracks will be induced by the application of tensile forces. Tensile cracks in concrete are inevitable, it is important to provide enough steel (both main bars and distribution bars) to distribute the cracks across the length and widths of the structural elements so as to limit the width of an individual crack.

(e) **Vibration** – Vibration can be caused by moving loads. Excessive vibration can cause discomfort to users. Sometimes vibration can also cause damage to the structure or the sensitive instruments housed in the structure.

(f) **Fatigue** – A structure may be subject to cyclic loads, including moving loads or seismic loads. If there is concern, it is necessary to consider the effect of fatigue.

40. Given the particular nature and characteristic of this structure, I also have the following additional observations concerning the specific parameters to be adopted in assessing whether the HUH structure is "fit for purpose":

    (a) **Seismic Design** – Seismic design is a requirement that comes from NWDSM **[OU6/3843-3844]**. In Hong Kong, apart from highway structures and some other buildings that the owners specifically require, there is no requirement for seismic design or seismic load checks.

    (b) For the structures in HUH Extension, as part of MTRCL's design requirements, they have to be designed to resist seismic loads, which is a client's requirement. It follows that seismic design needs to be taken into account in considering whether the structure is fit for purpose. From a structural engineering perspective, client's requirements shall be considered.

41. Structural engineers have to address (or have regards to) all the issues I have set out in paragraph 39(a) to 39(f) above in designing or checking their structures under serviceability limit state. Some of the requirements may affect the comfort of users. However, other requirements such as those in relation to crack widths and durability may also lead eventually to (structural) safety issues.

42. In my opinion, a structure should only be considered as "fit for purpose" if it is designed and built in a way that all the above factors are sufficiently addressed and would not give rise to any concern. I also note that these considerations are in fact reflected in the requirements stipulated in the Concrete Code. As a matter of illustration:-

    (a) **Durability**: There were long debates on this topic at the steering committee of the Concrete Code. Because of Hong Kong's special climate and environment, there may be issues of concrete spalling during the design life of buildings. This would pose particular risks to public safety because Hong Kong has many tall reinforced concrete buildings. Spalled concrete falling from height of 30 to 40 storeys could cause severe injuries to persons and damage to properties. The result was that the Concrete Code requires thicker (than that as required under BS8110) concrete covers in our concrete structures.

(b) **Cracking**: As mentioned above, it is important for the structural engineer to ensure that the width of the cracks that may appear in a structure is controlled within allowable limit. In so doing, the structure engineer would adhere to detailing rules that are specified in the design codes. The rules in the Concrete Code are meant to prevent cracks and provide good bond strengths to the rebars. The provisions in the Concrete Code reflect the views of experienced engineers and are in line with the results of extensive laboratory tests on different structural elements.

43. In conclusion, in considering the issues of safety and fitness for purpose, it is of paramount importance to take into account all the parameters set out above from a structural engineering perspective. Then, in considering the level of factor of safety, the standards laid down under the relevant codes in respect of those parameters would need to be applied. In other words, a structure will only be considered safe and fit for purpose if the standards governing the factor of safety in relation to each of the relevant parameters (as set out in the relevant codes) are met.

The Updated Design by MTRCL

44. I have been provided with the Updated Design calculations carried out by MTRCL **[OU6/3942-8578, 8579-9679]**. I believe that the other experts also are in possession of these calculations. My report relies on this set of calculations. From the Updated Design calculations, it is clear that the as-constructed structure has no structural stability problem. The only concern is the overstressing of local areas in the structure. The local overstressing can be overcome by the implementation of suitable measures. The structure, without the implementation of suitable measures cannot be considered safe or fit for purpose. This is my view having looked at the Updated Design. The extent of coupler and shear link irregularities that has led to insufficient structural capacities have been indicated in Atkins Assessment Report, relevant abstracts of which can be found in **[OU6/4493-4496, 4719-4720, 4779-4780, 4784-4785].**

45. The structural adequacy of the as-constructed station was checked against the Updated Design taking into account the defects found in the as-built structure. The as-built station box structure was found to be safe in terms of stability under

ULS (i.e. the Stability Check in paragraph 26(a) above). There was also no concern on robustness of the structure. In the checking of sections, MTRCL found that in some areas, the structure is locally overstressed (i.e. the Section Check in paragraph 26(b) above). MTRCL proposed suitable measures with a view to providing the structure with enough safety margin so as to satisfy the fit for purpose requirements including safety. The Updated Design and the required suitable measures proposed (albeit in schematic form) were submitted to and accepted by the Government. The detailed designs of the suitable measures have now been carried out and some of such measures are being carried out on site. According to the present working schedule, by January 2020, when the experts attend the hearing of COI, some of the suitable measures may have been completed. I note that when MTRCL carried out the structural assessment based on the Updated Design, they referred to the requirements of the Concrete Code and NWDSM. To comply with the directions of COI, I also looked at the issues from a structural engineer's point of view. I assessed the adequacy of the structure based on engineering principles, like force equilibrium, behavior of structural elements (made of concrete and steel reinforcements) under the prescribed design loads etc. As discussed above, I have also considered the provisions under the relevant codes in the process insofar as they are relevant to my structural engineering analysis.

46. Briefly, my answer to COI's question (a) is that the relevant works as constructed have not reached the level which structural engineers in Hong Kong would consider as "safe" and "fit for purpose". Suitable measures are therefore required to strengthen various parts of the as-built structures. In the latter part of my report, I shall give my comments on Mr. Southward's COI-1 report.

My views of the issues identified in the Holistic Report

47. The Holistic Report described the investigation of defects under Stage 2 of the holistic assessment.

48. External engineering consultants were engaged by MTRCL to carry out the Stage 3 structural assessment **[OU6/3942-8578, 8579-9679]** under the Holistic Proposal. The consultants' design was based on defects and defective rates found under Stage 2 investigation. Two scenarios were considered by the

consultants. The structure was assessed based on (i) the Original Design and (ii) the Updated Design.

49. When the structure was checked based on the Original Design, it was found that some areas of the structure would be overstressed (i.e. pursuant to the Section Check).

50. After reviewing the Original Design assumptions, MTRCL and its consultants recommended a set of revised design assumptions and criteria for the Updated Design to be used in the Holistic Assessment. The set of revised assumptions and criteria can be found in **Table 5** of the Holistic Report **[OU5/3280-3282]**. This Table is very important and I reproduced it in **Appendix JL1-C** of my report. There are 10 revised assumptions and criteria in the table. Out of the 10 items, 6 of them involved imposing restrictions on future use or require precautionary arrangements to be made. The result was that after the adoption of the revised set of assumptions and criteria, the consultants can then demonstrate that with the implementation of a more limited scope of suitable measures, the as-built structure can be strengthened so as to comply with the requirements of the Concrete Code and NWDSM, hence be considered as safe and fit for purpose.

51. Three major types of defects are found in the HUH structure, namely (i) defective couplers, (ii) non-complying shear links, and (iii) defects at the construction joint inside the connection between the east diaphragm wall and the EWL platform slab. There were other less serious problems, such as water seepage, and honeycombing. The Holistic Report provided a detailed strategy in dealing with the said three major types of defects **[OU5/3284-3285]**.

Defective Couplers

52. The type of defects in relation to coupler connections includes partially engaged coupler assemblies, unconnected couplers and thread cuts.

53. In the EWL and NSL slabs, the defective rates of couplers were estimated to be 36.6% (Table 1, 3.3.24) **[OU5/3255]** and 33.2% (Table 2, 3.3.24) **[OU5/3256]** respectively. Detailed results of the tests are set out in Appendix B3 of the Holistic Report. The defective rate of coupler connections in the EWL slab of

Area A was estimated to be 68% **[OU5/3728]**. As regards the unconnected couplers, as mentioned in slide 11 of the Oral Synopsis of Professor Yin, the 95% (1-sided) confidence interval upper bound of the rate of unconnected coupler in the EWL slab is 15.5% **[ER1/12.2]**. There was a reason for the adoption of 68% as the defective rate for coupler connections in EWL at Area A. This is the area where the EWL slab was connected to the capping beam on top of the diaphragm wall. The sequence and method of installation of the coupler connections in question were different from the rest of the coupler connections between the platform slabs and diaphragm wall. According to the evidence received during the hearing of the Original Inquiry, the coupler assemblies connecting the platform slabs to the diaphragm wall were originally cast in the diaphragm wall by Intrafor. They were subsequently exposed by Leighton with high pressure water jets. It was then discovered that some of the exposed couplers were disoriented or damaged. These caused a lot of difficulty in the subsequent screwing in of threaded bars and ensuring proper alignment of reinforcement by the steel fixers. Whereas in the case of the coupler connections at the capping beam, the couplers had never been cast in concrete, these coupler assemblies were therefore not damaged and without any issues of misalignment or disorientation. The reinforcement details are shown in the capping beam diagram below (which is reproduced from **[OU5/3430])**. In such a perfect working condition, one should not have (or would not expect) any difficulty to properly connect the threaded bars to the couplers. It is then obvious that the working methods adopted were different, the working conditions were different, the levels of difficulty involved in the respective installation work were also very different. The corresponding defective rates of the coupler connections installed therefore would unlikely be the same. In the circumstances, it was considered that the defective rate at the area of capping beam should be assessed separately. A 68% defective rate was then worked out based on the number of defective couplers found. I understand and agree with the principle behind this method of assessment.

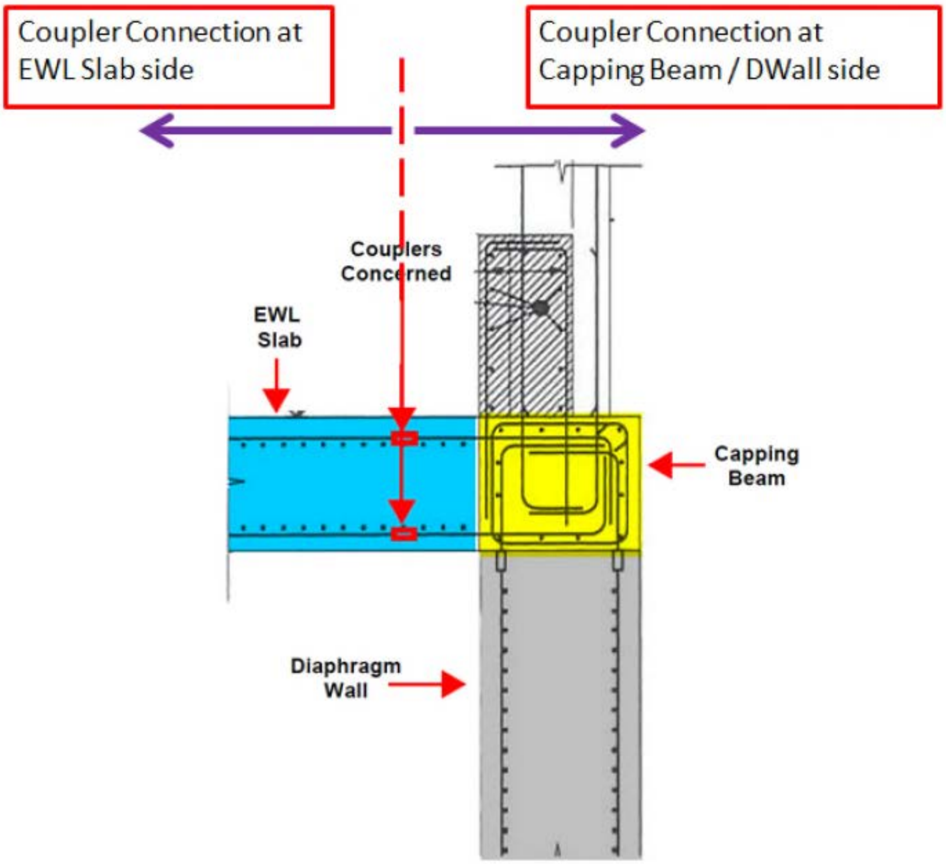

Figure 1 – Illustration of the coupler connection at the EWL slab side and capping beam / diaphragm wall side

54. Because of the defective coupler connections, a large amount of design calculations was carried out by MTRCL's consultants, namely Atkins, Arup and AECOM. The consultants carried out an assessment based on the Updated Design of the structure and found that because of reserve capacities, no suitable measures were needed in many areas. The Stage 3 Assessment Report, however, indicated that there were locations on the structure where the strength utilization ratios were greater than unity, meaning that these locations would be overstressed under one or more of the design loading conditions **[OU6/4496, 9308]**. This was the reason why suitable measures were proposed at these locations. The suitable measures were needed to increase the structural capacity of the structure at these locations so as to lower the utilization ratio down to below 1.

55. In Stage 3 Assessment, MTRCL and its consultants only took into consideration of the fully engaged coupler connections, the possible contribution of the partially engaged coupler connections were ignored. To comply with the requirement of the supplier/manufacturer, BOSA, the threaded bars must be connected "butt to butt" inside the coupler. Only fully engaged coupler connection (i.e. with "butt-to-butt" connection) can satisfy all the requirements of various tests specified in the Concrete Code. These tests include strength tests, permanent elongation tests and cyclic loading tests. The partially engaged coupler connections could not pass the permanent elongation tests.

56. I was provided with a total of 7 samples of the BOSA coupler assembly for my inspection. I studied and tested the samples of coupler assembly. To produce the butt-to-butt condition, I tightened the assembly by hand. Upon tightening, I could not cause any noticeable movement between the threaded bar and the coupler by pulling i.e. it was locked. However, when I allowed the samples to be partially engaged with different number of threads, I could always cause relative movements between the threaded bar and the coupler simply by pulling the assembly by hand. These are out-of-slip movements at the coupler connection because of partial engagements. Such relative movement is noticeable. These out-of-slip movements would be in addition to the permanent elongations manifested by the coupler assembly when it is stressed under loading. This can be a problem if the partially engaged couplers are used and embedded in concrete structure. According to the results of the tests commissioned by MTRCL, the permanent elongation of the partially engaged coupler connections could be up to 0.51mm **[OW1/240]**. Adding the said permanent elongation to the out-of-slip movement, the total deformation of the coupler connection would be a lot more than the 0.3mm maximum crack width allowed by the Concrete Code under the serviceability limit state **[H8/2928]**. Hence partially engaged coupler connection with a possible total deformation of more than 0.51 mm is unacceptable. I therefore have no confidence in the coupler assembly if it is only partially engaged. It is also difficult to come up with reliable methods to meaningfully calculate and ascertain crack widths or deflections in the structural element if the reinforcements are connected by partially engaged couplers.

57. A rebar fit with a coupler should behave as much as possible as a continuous rebar in terms of strength and deformation. A structural engineer should not

just look at the strength side of the test. He should also satisfy himself with the deformation of the rebar that is connected by a coupler. Clause 3.2.8.2 of the Concrete Code **[H8/2852]** places testing criteria on permanent deformation of not more than 0.1mm after it is loaded to 60% of the yield strength of the reinforcement. As reflected in the test reports, this requirement can only be satisfied if the connection is "butt-to-butt", i.e. with full engagement. Large deformation in a rebar fit with partially engaged couplers might cause large deflection and large crack width in the structural element. As I mentioned above, as per the requirement of the Concrete Code, the maximum acceptable crack width is 0.3mm **[H8/2928]**. The crack width in the HUH structure should be kept within this limit for durability.

58. In the Updated Design, MTRCL's consultants adopted a design approach that is accepted by most structural engineers. The consultants ignore the contribution of the partially engaged couplers. They assume the partially engaged couplers to be redundant in their calculations. I find the approach acceptable as a prudent engineering judgment.

Shear Links

59. Defects in shear reinforcements were discovered at 22 locations where there were honeycombs on the surface of the concrete structure. The extent of honeycombing is shown in Appendix B6 of the Holistic Report **[OU5/3328]**. There was additional opening-up done at further 18 locations for the investigation of the condition of shear links in the platform slab. In these locations, the following shear links defects were found: (i) complete absence of shear link, (ii) shear links of inadequate anchorage length, (iii) undersized link diameter, and (iv) over-spacing of shear links. Atkins' report Section 4.1c (reproduced in **Appendix JL1-D**) stated that "*Shear links were completely missing …"*. The findings are listed out in Atkins' Report (the relevant section, Section 3.8, is attached at **Appendix JL1-D**) and summarised in Appendix B8 of the Holistic Report **[OU5/3332]** (which is extracted at Figure 2 below for easy reference). According to the Holistic Report, suitable measures are required to be carried out to about 2.5% of the total floor area of Areas A, B, C and HKC. These are areas with high shear loads.

Findings of Shear Links Defects

EWL Slab

Type of Defects	Number of Locations in Honeycombing Inspected Areas	Number of Locations in Additional Opening-up Areas
No shear link	10	6
Inadequate anchorage length	2	5
Undersized link diameter and Inadequate anchorage length	1	1
Over-spacing of links and inadequate anchorage length	6	3
Undersized link diameter, over-spacing of links and inadequate anchorage length	3	3
Total	22	18

Figure 2 - Appendix B8 of the Holistic Report

60. There are photographs of the defective shear links shown to me. Some of the photo records showing no shear links, which are shown to me, are now extracted and attached in **Appendix JL1-E**. From the photographs, I believe that the deficiency may be caused by poor workmanship. The rebars were not tied together by steel wires. During concreting and the compaction operation by way of vibration, the shear links were probably displaced. In fact, it is not difficult for a competent contractor to fix the shear links of a thick concrete slab. It is common to build thick concrete slab in Hong Kong. Indeed, there are many 3 meter thick pile caps and transfer plates (which are basically concrete slabs) built in Hong Kong. It would not be difficult for steel fixers to fix similar shear links in a concrete slab of a thickness similar to that of the platform slabs provided that there is proper planning and proper work sequence is adopted.

61. Regarding anchorage length, I am uncomfortable with insufficient anchorage lengths (as compared with what is required under the Concrete Code) found with the shear links. Apparently, the construction drawings were not followed by the

steel fixers and their foremen. If that was the case, the shear links defects found on the structure would not be isolated cases.

62. The concern was not just on the insufficient length of anchorage (as stated in Mr. Southward's report) but rather more about the finding of no shear links at all at certain locations upon investigation. I understand that out of the 40 locations exposed, no shear link was found in 16 of them. Absence of shear links could cause shear cracks to develop across the part of concrete structure which is unreinforced.

63. Based on the deficiency discovered, MTRCL's consultants re-analyzed the structure under the Updated Design and found that in general the shear strength of the platform slabs in areas other than Area A are structurally sound [**OU6/4719-4720** and **OU6/4784-4785,** the relevant pages of which are reproduced at **Appendix JL1-F**]. It is because there is enough reserve shear capacity in the structural elements. As such, there would not be overstressing and no suitable measures would be required in those areas.

64. However, the consultants found that some locations of the platform slab in Area A would be overstressed in shear when the design loads are applied. As such, suitable measures are proposed at those locations. The overstressed areas are listed out in **Appendix JL1-F** of this report. These are mainly locations with heavy punching shears or areas with openings in the slabs.

Construction Joint

65. Originally, there should not be any horizontal construction joint within the connection between the east diaphragm wall and the EWL platform slab. However, Leighton, as part of the Second Change made during construction (without prior acceptance of the Building Authority), trimmed away the top of the diaphragm wall panels and re-cast it together with the platform slabs. In so doing a horizontal construction joint was formed at the top of the diaphragm wall and attention was since then put on the internal stresses generated at the said horizontal construction joint.

66. Furthermore, I note that in Part 1 of the Inquiry, some concerns had been expressed as to whether the joint would be overstressed because of the missing L-shaped rebars at the top of the diaphragm wall to transfer the internal forces to the platform slab.

67. Without the L-shaped rebars, tensile forces in the fixed moment joint had to be transferred through concrete. It is not desirable to rely on concrete to take tensile force. MTRCL’s consultants then carried out analyses of the joint in question based on the Updated Design. It is now found that the concrete at the construction joints is not overstressed under the Updated Design despite the lack of L-shaped vertical reinforcements at the top of the diaphragm wall. It is probably due to the fact that the internal stresses generated under the Updated Design are lower than those under the Original Design. I shall explain in greater details when I make comments on Mr. Southward’s COI-1 report.

68. To investigate the condition of the horizontal construction joints, 4 holes were cored through the joint. In the investigation, defects were found in 2 out of the 4 cores extracted from the holes **[OU5/3266]**. 2 out of 4 is a very high defective rate indeed. The concerns at the construction joint became one of poor quality and the need for defect rectifications.

69. MTRCL recommended suitable measures to be implemented by way of installation of dowel bars and grouting over a stretch of about 60m of diaphragm wall **[OU5/3284]** where the utilization rate of shear capacity within the diaphragm wall is high.

Suitable Measures

70. In the Holistic Report, suitable measures are proposed in a number of areas. Because of the defective coupler connections, suitable measures (in the form of drill in dowel bars and local thickening of slab) are required to strengthen the connections between the capping beam and the EWL slab at Area A, details are shown in Appendix C5 of Holistic Report **[OU5/3342]**. As to the lack of or non-complying shear links, suitable measures (in the form of localized thickening of slabs/walls, addition of load bearing walls and columns) are shown in Appendix C6 of Holistic Report **[OU5/3344-3348]**. The area where suitable

measures are required represents about 2.5% of the total floor area in Areas A, B, C and HKC. For construction joint, suitable measures (by installing dowel bars over a stretch of about 60m of diaphragm wall in Areas B & C) are shown in Appendix C7 of Holistic Report **[OU5/3350]**.

Mr. Southward's COI-1 Report

71. Mr. Southward in his report provided his views on partially engaged couplers, shear links and construction joint. He said that the HUH structure is safe. The following are my comments on his report.

Section 5 – Structural assessment of Works

72. In section 5, Mr. Southward provided his views on the work performed by MTRCL (and its consultant) under Stage 3 structural assessment for the as-built works.

73. **Section 5.3 – Findings of the Consultants** - *Mr. Southward said that "five separate and independent companies have carried out structural analysis and checking of the station structures, and all typically reach the same conclusions, that the design is safe and is over-provided by a considerable margin. That is, they conclude that there is a substantial amount of spare capacity in the Works."*

74. My comment: - I accept that the station box structure is safe as far as stability is concerned. This is what I refer to above as safety under the Stability Check. I also agree that there is spare capacity in the structure in general. But MTRCL's Detailed Design Consultant, Atkins, in Stage 3 structural assessment found that there were local areas of overstress because of the defects found in the as-built station box structure. And Atkins recommended that suitable measures be carried out to strengthen the overstressed areas. I have reviewed the details of Stage 3 structural assessment carried out by Atkins and I agree with its conclusions.

75. **Section 5.5 – Bending strength of slabs** - *Mr. Southward said, "All of the consultants' reports show that the bending strengths of the platform slabs are significantly in excess of the design requirements. Of particular interest however,*

is the EWL slab in Area A. As a result of the opening up investigations and the statistical analysis carried out by MTRCL, a large reduction factor of 65% [sic] has been applied to the strength of the concrete section at the location of coupler connections in the slabs in this area. EIC have reviewed the calculations of Atkins, Arup, AECOM and Ceek in this area. The results of this analysis have been extracted in Appendix C. These show a large variance in the percentage utilizations of concrete section in similar areas, but of note they all demonstrate that the design of the platform slabs is safe and code compliant."

76. My comment: - The figures set out in **Appendix C** of Mr. Southward's report did not incorporate any strength reduction factor in the calculation. Had he applied the strength reduction factor (to account for the defective couplers in the structure) in his assessment of the capacities of the structural elements, a number of sections of the structure would be overstressed **[OU6/4496, 9308]**. I also enclose the summary table in Appendix **JL1-G**.

77. **Section 5.6.2** – *Mr. Southward said that Atkin's shear calculations are conservative for the following reasons: - (a) Atkins did not use the correct tensile steel area. (b) the beneficial contribution of the axial load compressing the slabs has not been considered; and (c) the actual concrete strength supplied was higher than Grade 40. Mr. Southward further said, "I do not therefore consider the Atkins conclusions to be realistic and representative of the structure that has been constructed."*

78. My comment:- I consider that the issues stated in paragraph 77 (a) and (b) above may have certain contribution effect. I understand that the said matters have been taken on board by Atkins and have now been addressed or reflected in MTRCL's latest amendment submission, which nevertheless concluded that suitable measures are still required for a small area of the platform slab.

79. However, I have reservations on Mr. Southward's comment about the use of a higher concrete strength in the structural assessment. The use of a higher concrete strength for structural assessment is not acceptable unless Leighton had decided to and did order a higher grade of concrete (than Grade 40, which was the design grade and specified in Contract 1112) for the actual construction work. The strength of concrete depends on the design mix of concrete adopted in the design. In the HUH structure, the designer at the time adopted Grade

40D/20 concrete in the design. If the design mix proposed by Leighton at the time of construction was Grade 40D/20, and the design mix was accepted by MTRCL as Grade 40D/20, then Mr. Southward can only use the concrete strength of Grade 40 in his design checks. The higher concrete strengths obtained from laboratory tests on concrete cubes should not be relied on for the determination of the actual concrete strength in the structure. Strengths obtained from concrete cube tests are always (in fact inevitably) higher than the actual concrete strengths of the structure. It is because the concrete cube samples were separately compacted and cured in on site curing tank under ideal conditions before they were tested. Thus, the results can only be used as a means of quality control. They do not represent the actual concrete strength in the structure. If Mr. Southward can prove that at the start of the project, Leighton had submitted a new design mix for higher concrete strength, for example Grade 60, and was accepted by MTRCL, and the higher strength concrete design mix was shown on the documentation (e.g. delivery dockets, concrete cube test reports, etc.) issued by the concrete supplier, then he can now adopt the higher concrete strength in his structural assessment or design check. As a structural engineer, I do not agree to the use of the cube strength results in design check. The cube strength is higher than the strength of concrete in the structure.

80. **Section 5.6.3 – Revised calculations**. *Based on the above corrections of steel areas, axial loads and higher concrete strengths, EIC prepared a set of revised calculations. The revised calculations were presented in Mr. Southward's Appendix D. The table in Appendix D shows the corresponding utilization ratios. If the utilization ratio is greater than 1, suitable measures would be required. If the utilization ratio is less than 1, then no suitable measure is required. In one example, Mr. Southward showed that when Atkins's utilization ratio is 1.017, the revised utilization ratio due to adjustment for "actual steel", "Axial Compression" and "Actual strength" were 0.736, 0.640 and 0.531 respectively. Mr. Southward said that, "The EIC calculation proves that only 2.5m^2 of platform slab require minimum shear links, out of a total of 23,647m^2 of slab. This is 0.01% of the total. However, in my [i.e. Mr. Southward's] opinion, these areas have already been constructed with a satisfactory amount of minimum shear links which are code compliant."*

81. My comment: - Based on the detailed design in the amendment submission, 1% of the total area requires suitable measures for shear link defects. However, MTRCL's investigation revealed that 16 out of 40 locations (22 locations exposed due to honeycombing and 18 further locations opened up for investigation) were of missing shear links **[OU5/3332]**. Because of the exceptionally high percentage of locations where no shear links were found, MTRCL assumed in its structural assessment and subsequent detailed design that there is no shear link in the slab. In my view, the approach adopted by MTRCL in the circumstances is a prudent approach because shear failure is sudden (i.e. without sign or warning) which is an issue relating to Section Check mentioned above. The conditions of the shear links exposed in the honeycomb and opened-up investigations were indeed very poor. There are doubts about the contributions of these shear links to the shear strength of the slabs. Ignoring the contribution of those shear links is, in my view, an appropriate approach.

Couplers

82. **Section 6.1 – MTRCL testing of coupler assemblies**. *Mr. Southward said that "MTRCL carried out two sets of comprehensive tests on partially engaged coupler assemblies. Static tension tests and cyclic tension tests were carried out as well as measurement of the permanent deformation of the coupler at the completion of the static tension test……"*

83. My comment: - According to the test results, nearly all the test samples with partially engaged couplers failed the permanent elongation test **[OW1/93-108, 239-268]**. It was BOSA's, i.e. the coupler manufacturer's, requirement that the threaded bars should be connected "butt to butt" inside the coupler. It is only in this condition that the coupler assemblies can satisfy both the static tension and permanent elongation tests. Satisfaction of these tests is required by the coupler manufacturer and the Concrete Code. If it is the manufacturer's requirement that the installed couplers should satisfy the "butt to butt" condition, then it is imperative for the contractor to stick to this condition. The manufacturer of couplers is the expert on coupler installations and they know their own proprietary product better.

84. **Section 6.4 – What length of embedment is specified by design codes**. *Mr. Southward said that, "the HKCOP makes no mention of embedment lengths, tolerances on embedment lengths or anything that could be such interpreted."*

85. My comment: - What Mr. Southward said is true. The Concrete Code would not specify any embedment length because the embedment length required depends on the brand of couplers used. Leighton should follow the manufacturer's recommendations on the use of couplers. The Concrete Code can only control or verify the performance of the couplers used by performing tests on sample coupler assemblies. The tests in this instance are the static tension test and permanent elongation test.

86. **Section 6.5.1 – The HKCOP requirements**. *Mr. Southward said that the "HKCOP requires that couplers in tension are to meet the following:*

- *have a tensile strength exceeding 483 MPa*
- *exhibit a permanent deformation of not more than 0.1mm when loaded to 0.6f_y (where f_y is the yield strength of the rebar). HKCOP specifies no tests for couplers used in compression. On the basis of the HKCOP alone, it can be seen that the deformation test is not applicable for couplers that are to be used in compression."*

87. My comment: - Clause 3.2.8.1 of the Concrete Code for coupler **[H8/2852]** specifies that "*3.2.8.1 Bars in compression – The load may be transferred between butt jointed bars by means of end bearing where sawn square cut ends are held in contact by means of a suitable sleeve or other coupler. The concrete cover to the sleeve should not be less than that specified for normal reinforcement.*". This implies that the connection must be "butt to butt".

88. **Section 6.5.2 – BD additional requirements.** *Mr. Southward said, "only couplers with a ductility requirement that need to fulfil the cyclic tension and compression tests and be required to fail in bar break mode."*

89. My comment: - This is correct.

90. **Section 6.5.3 – Where do couplers with a ductility requirement need to be used in the Project.** *Mr. Southward said, "There were no ductility zones shown*

in the drawings for the couplers used within the slabs. As such, none of the couplers used in the slabs were subject to a ductility requirement".

91. My comment: - The use and the locations of ductility couplers (i.e. mechanical couplers for steel rebars for ductility requirement) are clearly specified in the accepted drawings **[H2/440, H4/840-843]** accepted by the Buildings Department.

92. **Section 6.6.1** - **Static tension tests**. *Mr. Southward said that the partially engaged couplers can meet the static tension test requirement.*

93. My comment: - According to the test results **[OW1/93, 97, 99]**, some of the partially engaged couplers failed to meet the static tension test requirement for couplers with a ductility requirement.

94. **Section 6.6.2 - Elongation tests**. *Mr. Southward said that the 0.1mm permanent deformation is not met by any of the partially engaged coupler assemblies.*

95. My comment: - This is the main concern of MTRCL. This is also my concern. Failure to meet this requirement has implication on ductility, crack width, durability and deformation which are parts of the requirements in respect of "fitness for purpose" including safety.

96. **Section 6.6.3 - Cyclic tests**. *Mr. Southward said that "the cyclic tests are satisfied where the bar being tested had 7 threads or 8 threads engaged in the coupler. The cyclic tests failed where the bar has 6 threads engaged in the coupler because the assembly did not fail in bar break mode."*

97. My comment: - The 7 thread or 8 thread partially engaged couplers are not suitable for the work because they still failed to satisfy the requirement of the permanent elongation tests. The test sample with 6 threads engaged also failed not only just because the assembly failed at the bar- coupler connection rather than in bar break mode, it also failed to provide the required strength and could not sustain the cyclic loads under the required test **[OW1/110-111]**.

98. **Section 6.7 - What embedment length is safe to use for a coupler assembly?** *Mr. Southward said that "all 40mm bar couplers with continuation bar that has 6 or more engaged threads are safe to be used in the works."*

99. My comment: - If Leighton or Mr. Southward wants to demonstrate to COI that a structural element constructed with partially engaged couplers has the same strength as that of one with fully engaged couplers, they should carry out loading tests of a sufficient number (for statistical purposes) to actual structural elements built with partially engaged coupler connections. It is only in such circumstances that the actual behavior of the structural elements in terms of safety and fit for purpose can be properly and adequately studied. In this connection, I understand that the Government has requested MTRCL **[OW1/152-155]** to submit relevant proposal on using the partially engaged couplers in the HUH structure, however, no such proposal has been received by the Government up to the present. To allow for the use of partially engaged couplers in structure solely on the basis of the tensile strength obtained from a limited number of tests is not a prudent approach.

100. **Section 6.8 -Are the constructed couplers fit for purpose?** *Mr. Southward said that all coupler connections with bars that have 6 or more threads engaged are fit for purpose.*

101. My comment: - Mr. Southward's definition of fit for purpose is different from mine. For me, fit for purpose means a number of things, safety being one of them. Fit for purpose also means the satisfaction of other criteria, such as deformation, control of crack widths and durability etc. If Mr. Southward wants to demonstrate that his partially engaged couplers can satisfy the requirements of deformation (deflection), crack widths and durability, he needs to produce calculations or research reports to demonstrate that a structure constructed with partially engaged couplers would behave in the same way as a structure built with fully engaged couplers. Mr. Southward had not done this.

102. **Section 6.9.2 - What is the purpose of the HKCOP?** *Mr. Southward said that "the HKCOP is not a statutory document". The requirements in the Concrete Code are not mandatory. Following the requirements of the HKCOP will be "deemed to comply with statutes".*

103. My comment: - Although the requirements in the Concrete Code are not mandatory, the level of safety of a design should be shown to achieve the performance requirements acceptable to the local society and compatible with local environment. In other words, all buildings in Hong Kong must be designed and constructed to a standard not inferior to that specified in the Concrete Code.

104. **Section 6.9.3 - What is engineering judgment?** *Mr. Southward talks about engineering judgment in his report.* I agree that engineering judgment would need to be exercised as and when necessary and appropriate. *He quotes an example about a wrongly constructed column with a size of 0.8m by 0.9m instead of 1m by 1m. He said that "the engineer will exercise engineering judgment in assessing the strength of the reduced size of column and if it can be proved by calculation that the reduced size can take the design loading, then by exercising such engineering judgment, one would keep the column and not require the contractor to knock down the column and re-build to the originally intended size."*

105. My comment:- In fact, what Mr. Southward's example shows is what MTRCL was doing under Stage 3 structural assessment. Engineering judgment has been exercised by MTRCL in the Updated Design. The HUH structure satisfies the Stability Check. Many areas of the HUH structure satisfies the Section Check and fit for purpose requirements. However, a number of areas were overstressed and failed to satisfy the Section Check and the fit for purpose requirements. Only these limited areas require the implementation of suitable measures. This is engineering judgment. I have to emphasize that engineering judgment must be exercised while ensuring the relevant standard is complied with. Going back to the example used by Mr. Southward's – the 0.8m by 0.9m column, the structural engineer must check that despite the smaller size, the structure still has the reserve strength to take the column loads. The reserve strength is to be computed by reference to the Concrete Code. If there is not enough reserve strength in the column, the engineer should exercise engineering judgment to instruct the contractor to strengthen the column. Whichever way he exercises his engineering judgment, he still needs to comply with the requirements of the Concrete Code. The final product of his engineering judgment cannot be one that falls below the standard of the Concrete Code.

106. **Section 6.9.4 - Code compliant couplers**. *Mr. Southward said, "If the results of MTRCL's recent tests on the couplers are accepted, it appears that BOSA couplers would need to have a "butt to butt" connection in order to satisfy the strict standards set out by BD for testing of couplers. But, in every situation, it is possible to incorporate defects into the Works, provided that they have been considered and checked in the design."*

107. My comment: - I accept Mr. Southward's argument of incorporating the defects (meaning the partially engaged couplers into Works) provided that they have been considered and checked in the design. But Mr. Southward did not check the design of the Works that incorporated the partially engaged couplers against all the fit for purpose requirements. Mr. Southward provided no such calculations in his report.

108. **Section 6.10.1 - Results and findings of the Holistic Report**. *Mr. Southward disagrees with the defective rate of coupler connections of 36.6% and 32.2% of the EWL and NSL slabs respectively. He also disagrees with the higher defective rate of 68% for the coupler connections in locations where the EWL slab connects to the east diaphragm wall via capping beams. The defective rates were determined statistically using the binomial analysis based on a measured engagement length of 37mm (i.e. an actual engagement length of 40mm with an allowance of 3mm for measurement tolerance). In Professor Yin's analysis, coupler connections with a measured engagement length of less than 37mm is considered defective. Dr. Wells, however, adopts a much lower acceptance criterion, namely a minimum engagement of 28mm only, in his analysis. On this basis, Dr. Wells worked out the corresponding defective rates to be 16.3% (for EWL), 6.9% (for NSL) and 10.2% (for combined EWL and NSL). Dr. Wells also said that a number of "acceptable test results" had been discarded by Professor Yin which affected Professor Yin's analysis. He proposed to adopt "Missing Values Approach" to account for the discarded data. By adopting the "Missing Value Approach", the defective rates could be further reduced to 14.5% (for EWL), 6.5% (for NSL) and 9.4% (for combined EWL and NSL). Dr. Wells further considers that the correct approach is to combine the test data for the EWL and NSL slabs in the statistical analysis. By setting 28mm engagement length as the threshold for acceptance, the defective rates would be reduced down to 9.4% (if Missing Value Approach was adopted) or 10.2% (if Missing Value Approach was not adopted).*

109. My comment: - There is a large variance between the defective rates adopted by MTRCL and Mr. Southward. The difference arises from the adoption of a different threshold of defective engagement lengths. In MTRCL's case, it was 40mm and in the case of Mr. Southward's, it was 28mm. 40mm engagement is "butt to butt" and full engagement. 28mm is partial engagement. I do not agree with Mr. Southward's view that the acceptance criterion should be set at 28mm engagement. I have set out my reasoning in paragraphs 99 and 101 above. Further, I do not agree with Mr. Southward that the defective rates (thus strength reduction factors) to be adopted in the structural assessment should be 9.4% (if Missing Value Approach was adopted) or 10.2% (if Missing Value Approach was not adopted).

110. **Section 6.12 - Summary**. *Mr. Southward said that there is no engineering justification for any of the proposed "suitable measures" that are purported to be necessary due to partially engaged couplers.*

111. My comment: - In my opinion, the defective rates to be used in the assessment of suitable measures depend on what constitutes defective couplers. Mr. Southward said that partially engaged couplers (with an engagement length of 28mm or more) are not defective. I disagree. Mr. Southward has not carried out any calculations or test to demonstrate that the partially engaged couplers can satisfy the requirements in respect of structural **safety** and **fit for purpose** or when they are used and cast in concrete structure, they would behave in the same way as fully engaged coupler assemblies. For the reasons I explained above, all the partially engaged coupler connections should be considered as defective, the defective rates determined statistically by Professor Yin should be adopted as what MTRCL did in its Stage 3 structural assessment. Using Professor Yin's defective rates, MTRCL found that suitable measures are required to be implemented at certain locations of the EWL slab in Area A. I agree that this is the correct approach. I refer to paragraphs 99 and 101 above.

112. *By way of summary, Mr. Southward further opines that despite the presence of defects in the couplers, the HUH structure at the EWL slab in Area A is not overstressed and it is safe.*

113. My comment: - As explained in paragraph 54 above, certain areas of the EWL slab in Area A would be overstressed when it is subject to the required design loads. Further, Mr. Southward did not check the condition or behavior of the structure under serviceability limit state, nor did he demonstrate that with the extensive use of partially engaged coupler connections in the structure, relevant requirements for **fit for purpose** are satisfied.

Shear Link Reinforcement

114. **Section 7.1 - MTRCL stage 2 opening up investigation**. *Mr. Southward criticized MTRCL's opening up investigation of shear links. He uses photographic records of HZ01 to HZ18 in support of his criticisms. He said the opening up was not 1m by 1m as claimed by MTRCL. The area that was opened up was actually in an L shape of 1m along each direction.*

115. My comment: - If we are only interested in the presence of shear links within the 1m by 1m opening, then a 1m by 1m L shape opening is sufficient. The L shape opening was proposed by MTRCL. When compared with the 1m x 1m patch, this would cause less destruction to the slab soffit. According to the method statement submitted by MTRCL for shear link investigation, non-destructive scanning was firstly conducted within the 1m x 1m area so as to locate the embedded main rebars. These rebars are placed at 150mm centre-to-centre and an area of 300mm x 300mm was opened up to expose two main rebars. According to the design drawings, the spacing of shear links in both longitudinal and transverse directions could be 75, 150 or 300mm. Therefore, at least one shear link would be exposed in this 300mm x 300mm area if shear links had been constructed according to the design. Based on the location of exposed shear link, two strips of 200 mm wide and around 600 mm long were further opened up to investigate the arrangement and condition of shear links installed in both directions. More shear links should have been exposed if Leighton had constructed the work in accordance with the design drawings.

116. **Section 7.2 - LCAL Opening Up**. *Mr. Southward shows in Figure 6 one single location where Leighton has opened up an area of 1m by 1m at the soffit of the slab. He further seeks to demonstrate that it is possible that despite the presence of shear links in the slab, they could well be missed because of the positioning of the L shape opening.*

117. My comment: - Defective shear links were discovered because of the existence of honeycombs at the soffit of the EWL slab and the further investigation carried out subsequently. There are many photographs showing the defective shear links after the poor quality concrete at the honeycomb areas was removed. Altogether 40 locations were investigated by MTRCL and every one of them showed defective shear links. I attach in **Appendix JL1-E** to this report photographs showing the conditions of the defective shear links in some of those locations. The defective rate is basically <u>100%</u>. MTRCL has no confidence that the shear links were properly installed. There were doubts as to whether shear links were actually installed by Leighton in other areas that were not opened up **[OU5/3731]**. As explained in paragraph 115 above, a 300mm x 300mm square was opened up to expose two main rebars and at least one shear link would be exposed in this square if shear links had been actually constructed. As the width of the legs of the L-shaped openings is more than 200mm [See **Appendix JL1-E**], the presence of shear links will not be missed under this L-shaped opening configuration. The dimensions of the superimposed right angle slot shown in Figure 6 of Mr. Southward's report are wrong and do not correctly represent the actual extent of the opening-up investigation.

118. **Section 7.3 - MTRCL conclusion from Stage 2 investigation and Stage 3 assessments**. *Mr. Southward said, "On the basis of the observations made on 18 opening up locations MTRCL have unilaterally disregarded the presence of all shear links in the EWL and NSL slabs. With these shear links disregarded, the Atkins calculations show that in some isolated areas, strengthening of the EWL and NSL slabs [OU6/4720 and OU6/4785] is required in order to satisfy the HKCOP requirement of minimum shear links."*

119. My comment: - I looked at the photographs of the exposed shear links at the honeycomb areas as well as photographs of the 18 further locations opened up for the purpose of investigation. There are 40 locations in total. In every one of these locations, defective shear links were found. The defects include missing shear links, wrong diameter of the shear links, wrong shear links spacing and inadequate anchorage lengths. In my opinion, MTRCL's lack of confidence in having the correct amount of shear links at the right positions to take up the applied shear forces is justified in the circumstances. The shear forces come from punching shears of columns or shear due to openings in the slabs. If one

has doubt about the presence of shear links in a certain location, one cannot assume that they are there and correctly placed. It is more reasonable to assume that they are not there and use engineering judgment to carry out analysis based on such assumptions. It is prudent for a structural engineer doing the design to ignore the contribution of the defective shear links in his calculation in these critical areas. Mr. Southward proceeded with his structural review on the basis of the result found from that one single area opened up by Leighton and assumed that all the other areas are of the same good quality. The poor quality of shear links exposed and the lack of shear links discovered in the 40 locations under investigation (some of them were randomly selected) clearly show that Mr Southward's assumption is not reliable.

120. **Section 7.4 - Were the shear links required to be extended to the bottom layer of reinforcement?** *Mr. Southward said, "It is not necessary for minimum shear steel to extend all the way to the bottom mat of the reinforcement, especially in this instance where the Atkins design has not considered the presence of all the tension reinforcement in the slab in their calculation of the concrete component of the shear capacity of concrete. It is important however, that the shear link is long enough that the 90-degree bend in the link occurs above or below (depending on the location of the tension face of concrete) the centroid of the tension reinforcement. This is necessary in order that the shear strut and tie model is valid. This model being the standard manner in which shear analysis and design is carried out in the design codes."*

121. My comment: - I agree. There is no need for the shear links to be extended to the bottommost layer of reinforcement. As I understand it, the problem MTRCL was facing at the time of Stage 3 structural assessment was that because of the high percentage of location showing a complete lack of shear links, it has no confidence that the required shear links had been provided **[OU5/3731]**.

122. Since Mr. Southward mentioned the difficulties in placing the shear links as designed, I must make my response based on my personal experience both as a designer and contractor. I run a construction business in Hong Kong and I have experience in the construction of 2.5m to 3m thick pile caps and very often 3m thick transfer plates (which are similar to the platform slabs in question). There are usually high shear forces in these structural elements because of large punching shear. In the case of transfer plates, it is further complicated by

the presence of many openings. The rebars in these structures are even more congested than those in the platform slabs. Yet, steel fixers did not have problem to install shear links properly in these heavily reinforced structures. In my opinion, it is just a matter of proper planning and sequencing of the steel fixing work. There is absolutely no excuse for the poor shear links work found in the platform slabs of the HUH structure. When I look at the photographs, I found that the shear links were not tied to the main rebars by steel wires. Good workmanship requires that all steel bars be tied together at their intersections. This is to prevent the rebars from being displaced during concreting and compaction of concrete. The insufficient anchorage length in the shear links is common in this project. My concern is that the construction drawings were not followed by the steel fixers and their foremen. If this was the case, the defects could be widespread all over the site. It is therefore prudent for MTRCL to ignore the shear links in critical/sensitive areas.

123. **Section 7.5 - Review of HKCOP requirements**. *Mr. Southward said, "The chief concern is that the as-constructed shear links are not code compliant and this is the reason for them being disregarded in the stage 3 calculations.''*

124. My comment: - The concern is not the shear links not being code compliant. The concern is the absence of shear links at the required locations to take up the design shear forces.

125. **Sections 7.5.1, 7.5.2, 7.5.3 and 7.5.4 - Determination of anchor length**. *Mr. Southward uses calculations to demonstrate that the as-constructed shear links have enough bond strength as anchorage.*

126. My comment: - I have no disagreement with Mr. Southward on this. The as-constructed shear links certainly have certain degree of anchorage. I do not disagree. But as I said, the concern is simply that there is doubt as to whether shear links were installed at the right locations.

127. **Section 7.6.2 - AASHTO approach for anchorage of shear links.** *Mr. Southward argues that the as-constructed shear link anchorage complies with the provisions in AASHTO.*

128. My comment: - I do not disagree that the as-constructed shear links have some degree of anchorage. I reiterate that the concern is the possible absence of shear links at the required locations, in particular the critical locations, to take up the design shear forces.

129. **Section 7.6.3 - BS8110 approach to anchorage of shear links**. *Mr. Southward uses BS8110 as comparison.*

130. My comment: - There is no need for comparison. In any case, the concern is not the anchorage length or the extent of anchorage. I reiterate that the concern is the possible absence of the shear links at the required locations, in particular critical locations, to take up the shear forces.

131. **Section 7.7 - First Alternative Method to consider anchorage of shear links**. *Mr. Southward challenged the assumption of shear distribution adopted by Atkins, other consultants and the applicable codes. He said that in reality, the shear stress distribution across the depth of the section should not be even. He said that according to EIC's finite element analysis (in his Appendix E), the shear stress distribution across the depth of the section should be parabolic. If that is the case, the load required to be carried by the 90-degree bend at the end the shear link is small. In other words, it does not matter if the length in the bend is small.*

132. My comment: - The assumption of strut and tie action is allowed in the Concrete Code and adopted by most design engineers in practice. Strut and tie models were tested and verified numerous times in laboratories all over the world. There is good reason to trust the method adopted in the Concrete Code. Mr. Southward proposed a different approach of using finite element analysis. I do not know the details of the finite element model adopted by EIC in its analysis. It is possible that the parabolic shear stress distribution is the result of a plain uncracked concrete section without any reinforcements. I believe the model was not tested in the laboratory. If this was adopted, there should not have any anchorage bend requirement according to the AASHTO, Eurocodes and British Standards. The stress distribution across the depth of the section will be totally different if a non-linear elasto-plastic finite element analysis is carried out with all the reinforcements in the section properly accounted for and modeled, which is what ought to be done. The stress distribution will not be parabolic in this case.

Concrete is a strain-softening material. As soon as loads are applied to the reinforced concrete section, concrete cracks and shed loads to the rebars and other concrete elements. The final result is still represented by a strut and tie model.

133. In any case, the concern here is not the stress in the anchorage bars. The concern is the possible absence of shear links.

134. **Section 7.8 - Second Alternative Method to consider anchorage of shear links**. *Mr. Southward quoted a paper published by Professor Stephen Foster. Based on Professor Foster's paper, EIC recalculated the effectiveness of the as-constructed shear links. EIC found that in Area A, the shear reinforcement provided was 95% effective. In Area C, the arrangement was 86% effective.*

135. My comment: - I have no disagreement with Mr. Southward that the as constructed shear links (if they are presence) would have some shear capacities albeit somewhat reduced due to insufficient anchorage. However, as I repeatedly said, MTRCL's concern was the possible absence of shear links at critical locations. This is also my concern. We cannot be sure that there are reinforcements at the locations that shear links are required.

136. **Section 7.9 - Are the suitable measures for the shear links actually required?** *Mr. Southward said, "no". He said, "the limited investigation measures of MTRCL do not prove that the shear links were not installed in the relevant parts of the Works."*

137. My comment: - In many locations, there is enough shear resistance in the as-built section. However, at critical locations of the structure where shear forces are high, MTRCL has concern about the possible absence of shear links. This was the basis of the design of suitable measures. I agree that this is an appropriate approach.

Construction Joint

138. **Section 8.1: - As constructed design of joint**. *Mr. Southward said, "the joint has been demonstrated by several consultants to be adequate in a similar*

manner to the evidence presented by Professor McQuillan, Dr. Glover and myself in the hearing in January 2019."

139. My comment: - There was a history about the irregularity found at the connection between the EWL platform slab and the east diaphragm wall. In the design of the diaphragm wall as a retaining structure to be built by way of top down construction, the top of the diaphragm wall was assumed to be fixed to the EWL slab as a rigid joint. As the top of the diaphragm wall was designed to be fixed, the bending moment in the diaphragm wall was drastically reduced. So, this requirement of fixity and the corresponding fix end moment at the top of the diaphragm wall is an important consideration in the detailing of the joint.

140. To allow the joint to take the large fixed end moment, vertical reinforcements with a L-shape bend at the end (U-bars also help to some extent) were specified by the designer at the joint. The L-shape ends of the bars help to transfer the tensile loads from the vertical rebars of the diaphragm wall to the horizontal rebars in the EWL slab and vice versa. It is now found that both the L-shape bars and U-bars have not been provided at the joint. So, the tension from the vertical rebars in the diaphragm wall can only be transferred to the horizontal rebars in the EWL slabs through the concrete material.

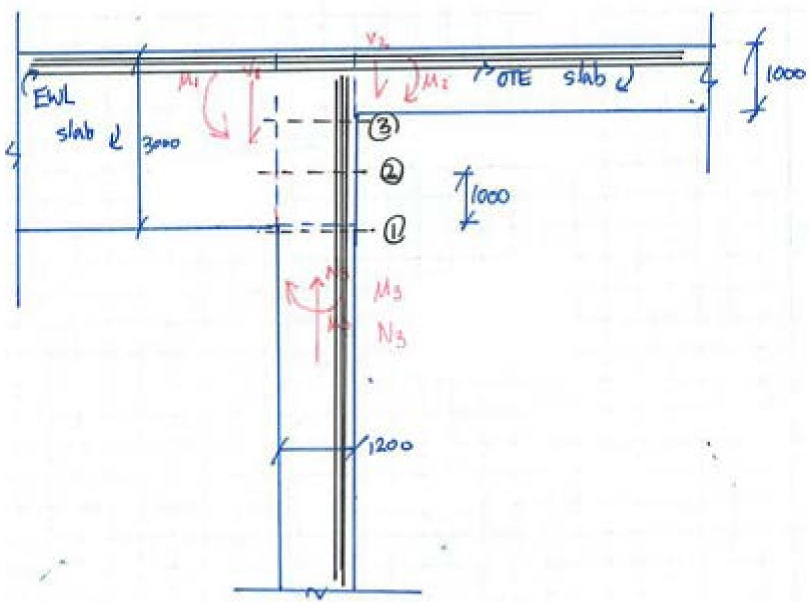

Figure 3 - Typical section of joint showing the main longitudinal rebar

141. It is not desirable to rely on concrete to take tensile force because concrete is weak in tension. The other way was to allow the forces to be transferred by

shear through the section. This raised the concern of high stresses in the concrete at the joint. Different methods were used trying to ascertain the internal stresses generated at the joint. The analysis was made complicated because of the introduction of a somewhat irregular construction joint at about 500mm below the surface of the EWL slab.

142. Based on the assumptions of the Original Design, the analyses showed that the joint was overstressed. Under the revised design assumptions of the Updated Design, the results of the simplified hand calculations and finite element analyses by the consultants showed that the joint was no longer overstressed **[OU6/4107-4110]**.

143. To investigate the condition of horizontal construction joints, 4 holes were cored and samples were extracted for inspection. In the process, defects were found in 2 out of the 4 cores extracted from the holes. The attention then went to defect rectifications.

144. The defective rate of 2 out of 4 is indeed very high. Because of the uncertainty in workmanship at the said construction joint, MTRCL recommended suitable measures be implemented by installing dowel bars through the joint and grouting over a length of about 60m of diaphragm wall.

145. **Section 8.2 – Condition of horizontal construction joint** – *Mr. Southward said in Paragraph. 4, "These two workmanship defects are not of any structural concern."*

146. My comment: - The gap found in the horizontal construction joint may create a path for the ingress of water into the reinforcement zone and it may lead to corrosion of reinforcements. It is an issue of durability of structure.

147. Mr. Southward proposed the use of pressure grouting into the drilled holes across the construction joints. I do not think that the proposal is better than the use of grouted dowel bars. The real issue here is the doubt as to the workmanship of the construction joints. One cannot just apply pressure grouting at the two locations where defects were found, as there may be other locations subject to the same issues which were not opened up. The result of the investigation indicated that 50% of the cores extracted are defective in one

way or another, the workmanship of construction joint in other areas remain doubtful. Further, the dowel bars to be installed would provide a mechanical key at the construction joint which would not otherwise be provided.

148. **Section 8.4 - Finite element analysis**. *Mr. Southward carried out a finite element analysis for the connection between the EWL slab and east diaphragm wall. He uses two models. In one model, he assumes that shear cannot be transferred over the unreinforced section. He has another model, which can transfer shear through the unreinforced section. He then purports to demonstrate that the small gap at the D-wall interface makes little difference to the overall performance of the joint.*

149. Before I comment on Mr. Southward's finite element analyses, I need to give some background about my past involvement with finite element analyses.

150. From 1973 to 1977, I was employed as a research assistant at the Stability Analysis Group of King's College, University of London, under Professors Nash, Gibson and Dougill. My job was to carry out laboratory tests and write computer codes using non-linear elasto-plastic finite element method to model the development of cracks and their propagations in strain softening materials, such as concrete and rock. Strain softening material is another name for brittle material. In the research, I also studied the interaction between the strain-softening material and the embedded steel, such as rebars and rock bolts. In 1977, I wrote up results of the research and obtained my PhD at the University of London. During the four years of my research, I have written over a thousand lines of computer codes on the elasto-plastic finite element method. I tested run my codes on finite element models of different structures. I then verified the results using real structures by testing (i.e. loading) them to failure in the laboratory. I have a good understanding of the use and limitation of finite element method. I think I am qualified enough to make comments on Mr. Southward's finite element results.

151. **Section 8.4.1 - Model description**. *Mr. Southward said, "a linear elastic concrete material model was used without explicitly modelling rebar." Figure 16 in his report showed the geometry, the finite element mesh and the overall deformation of the joint. The moment and shear applied to the joint are 7090kNm/m and 1350kN/m respectively.*

152. **Section 8.4.2 - Comparative results**. *Mr. Southward said the results show that the stress distribution for two models are almost identical. A slight variation in stress around the construction joint can be seen in the low stress range. He shows the contour plot of equivalent stress for the two models in Figure 18 and Figure 19. In Figure 20, Figure 21 and Figure 22, he further shows the positions and values of horizontal stress and vertical stress at selected points of the joint.*

153. I have the following comments on Mr. Southward's Figures 20, 21 and 22. Figure 22 are vertical stress outputs at 13 points in the joint. Mr. Southward does not indicate which points have tensile stress and which points have compressive stress. Figure 16 tells me that points 13,12,11,10 must be in tension and points 1 and 2 in compression. Figure 21 shows horizontal stress and Figure 22 shows vertical stress. There are shear stresses in the output. So, a better presentation is to combine the stresses so as to show the principal stresses. As Grade 40 concrete was used, the maximum allowable tensile strength in the concrete joint is 4.0 N/mm^2. So, point 13 (showing 16.9 N/mm^2) and point 12 (showing 11.6 N/mm^2) must be severely overstressed and tensile cracks should have appeared. These tensile stresses cannot be sustained by the elements in the model and another iteration must be performed by Mr. Southward to distribute the forces. In the further iteration, a much-reduced stiffness value should be used for the cracked elements. This helps to spread the stresses to other uncracked elements. Without the re-distribution, the force patterns in his output are not of significance. Mr. Southward shows very low stress at points, 6, 7, 8 and 9. Mr. Southward uses a linear elastic finite element model. Based on linear elastic theory, these should be points of high stress concentrations. At the tips, the elastic stress approaches a value of "infinity". Because of high stress concentrations, these are the likely points for crack propagation. Mr. Southward's analyses failed to reflect the above stress condition. I am also concerned about the cracked elements in Mr. Southward's analysis.

154. My other comment on Mr. Southward's analysis is that it is wrong to use a 2D plane stress analysis. He should have used a 3D model. If he decides not to use a 3D model, he should at least use a plane strain analysis. The 2D plane stress analysis adopted by him is inappropriate. I do not need to explain what is 3D. One can simply convert the 3D model to 2D by adopting either 2D plane

stress or 2D plane strain. Plane stress means a thin sheet of material and you look at and analyze the material on the plane sheet. 2D plane strain means a continuous long row of the same material. You look at and analyze the material on a plane of this long row. It is more appropriate to choose 2D plane strain rather than 2D plane stress in his analysis.

155. Not modelling the rebar in Mr. Southward's model is another major problem. If a non-linear elasto-plastic finite element method had been used, Mr. Southward would have modelled the development of tensile forces in the model and the resulting stress patterns would have been totally different from those currently shown in Mr. Southward's figures. The results from Mr. Southward are not useful at all.

156. In layman's terms, as the analysis has ignored any possible failure of concrete itself and did not consider the interactions between rebars and concrete, what it predicts is unreliable.

157. Hence, unless one fully understands the behavior of materials and the limitation of the finite element method, the results and outputs provided by Mr. Southward are unreliable.

Concrete Code

158. As stated above, I do not think this is a proper forum for discussing the appropriateness of the Concrete Code. However, as there were comments to the effect that the Concrete Code was too conservative, I only wish to highlight a few points.

159. In the past, Hong Kong used British Codes, such as the CP114 which adopted permissible stress design approach. In 1987, Hong Kong issued its first code of practice on structural use of concrete, viz Code of Practice for Structural Use of Concrete 1987 which also adopted permissible stress design approach. Later, CP114 was superseded by BS8110 which adopted limit state design approach for design. After joining the European Union, Britain adopted the Euro Code for design. There was thus a choice for Hong Kong. We could either follow Britain's example to use Euro Code or continued to use BS 8110. Alternatively,

Hong Kong could have its own design codes. The decision was to produce Hong Kong's own design codes. The Concrete Code was therefore drawn up by a consultant under the direction of a steering committee comprising representatives of relevant stakeholder organisations, professional institutions, academia and relevant government departments. Formal consultation with the building industry through the established consultation channels had been made before the promulgation of the Concrete Code in 2004. The Concrete Code incorporated the latest results of concrete research. Some provisions in the Concrete Code specifically deal with or address the local conditions in Hong Kong. It is therefore a standard specifically drafted for use in Hong Kong after thorough and extensive consultation with the relevant stakeholders in the building industry.

160. The Concrete Code incorporated comments from the Association of Consulting Engineers of Hong Kong, which comprises local and international consulting firms. It also incorporated some latest research findings from local and overseas universities then available.

Declaration

I, Dr. James Lau, declare that:

1. I understand that my duty in providing written reports and giving evidence is to help the COI, and that this duty overrides any obligation to the party by whom I am engaged or the person who has paid or is liable to pay me. I confirm that I have complied and will continue to comply with my duty.

2. I confirm that I have not entered into any arrangement where the amount or payment of my fees is in any way dependent on the outcome of the case.

3. I know of no conflict of interest of any kind in taking up this case.

4. I do not consider that any interest which I have disclosed affects my suitability as an expert witness on any issues on which I have given evidence.

5. I will advise the party by whom I am instructed if, between the date of my report and the trial, there is any change in circumstances which affect my answers to points 3 and 4 above.

6. I have shown the sources of all information I have used.

7. I have exercised reasonable care and skill in order to be accurate and complete in preparing this report.

8. I have endeavored to include in my report those matters, of which I have knowledge or of which I have been made aware, that might adversely affect the validity of my opinion. I have clearly stated any qualifications to my opinion.

9. I have not, without forming an independent view, included or excluded anything which has been suggested to me by others, including my instructing lawyers.

10. I will notify those instructing me immediately and confirm in writing if, for any reason, my existing report requires any correction or qualification.

11. I have acted in accordance with the Code of Conduct for Expert Witnesses as contained in Appendix D to the Rules of the High Court (Cap 4A).

Statement of Truth

I confirm that, insofar as the facts stated in my report are within my own knowledge, have made clear which they are and I believe them to be true, and that the opinions I have expressed represent my true and complete professional opinion.

Signature ______________________

Dr. James Lau, PhD, BBS, JP.
Authorized Person
Registered Structural Engineer
Registered Geotechnical Engineer