SUPPLEMENTAL EXPERT REPORT

ORIGINAL INQUIRY (COI 1)

PREPARED BY

PROFESSOR DON McQUILLAN

Expert Witness appointed by the Commission of Inquiry into the Construction Works at and near the Hung Hom Station Extension under the Shatin to Central Link Project (formerly Commission of Inquiry into the Diaphragm Wall and Platform Slab Construction Works at the Hung Hom Station Extension under the Shatin to Central Link Project)

6 December 2019

Commission of Inquiry into the Construction Works at and near the Hung Hom Station Extension under the Shatin to Central Link Project (formerly Commission of Inquiry into the Diaphragm Wall and Platform Slab Construction Works at the Hung Hom Station Extension under the Shatin to Central Link Project)

Original Inquiry (CoI 1)

<u>Professor Don McQuillan</u>

Chartered Engineer specializing in Structural, Civil, Bridge & Marine Engineering;

Director of RPS

Specialist Field	Investigating and assessing defects and failures in buildings and other structures, arising from design and construction and extraneous sources, as further detailed in **Appendix I**
Appointed on behalf of	Commission of Inquiry into the Construction Works at and near the Hung Hom Station Extension under the Shatin to Central Link Project (formerly Commission of Inquiry into the Diaphragm Wall and Platform Slab Construction Works at the Hung Hom Station Extension under the Shatin to Central Link Project) (The "**Commission**")
Prepared for	The Commission
On instructions of	Messrs. Lo & Lo, Solicitors for the Commission ("**Lo & Lo**")
Subject matter / Scope of engagement:	To assist the Commission in discharging its duties under the Expanded Terms of Reference and by acting as an Expert Witness in the Inquiry hearings
Documents	I was given access to the documents in the hearing bundles. References in the text of this Report are references to pages in the hearing bundles.
Analysis integral to this Report prepared by the Author	See **Appendix II**
Meetings, visits and inspections	See **Appendix III**

THE EXPANDED TERMS OF REFERENCE OF THE COMMISSION ARE AS FOLLOWS:

Regarding the MTR Corporation Limited ('MTRCL')'s Contract No. 1112 ('Contract') of the Shatin to Central Link Project:

(a) (1) in respect of the diaphragm wall and platform slab construction works at the Hung Hom Station Extension,

(i) to inquire into the facts and circumstances surrounding the steel reinforcement fixing works, including but not limited to those works at locations that have given rise to extensive public concern about their safety since May 2018;

(ii) to inquire into the facts and circumstances surrounding any other works which raise concerns about public safety; and

(iii) to ascertain whether the works in (1)(i) and (ii) above were executed in accordance with the Contract. If not, the reasons therefor and whether steps for rectification have been taken;

(a) (2) in respect of the construction works at the North Approach Tunnels, the South Approach Tunnels and the Hung Hom Stabling Sidings,

(i) to inquire into the facts and circumstances surrounding any problem relating to the steel reinforcement fixing or concreting works, including but not limited to any lack of proper inspection, supervision or documentation of such works undertaken, any lack of proper testing of the materials used for such works and of proper documentation of such testing, and any deviation of such works undertaken from the designs, plans or drawings accepted by the Highways Department or the Building Authority;

(ii) to inquire into the facts and circumstances surrounding any works or matters which raise concerns about public safety or substantial works quality; and

(iii) to ascertain whether the works and matters involved in (2)(i) and (ii) above were executed in accordance with the Contract. If not, the reasons therefor and whether steps for rectification have been taken;

(b) to review, in the light of (a) above,

(i) the adequacy of the relevant aspects of the MTRCL's project management and supervision system, quality assurance and quality control system, risk management system, site supervision and control system and processes, system on reporting to Government, system and processes for communication internally and with various stakeholders, and any other related systems, processes and practices, and the implementation thereof; and

(ii) the extent and adequacy of the monitoring and control mechanisms of the Government, and the implementation thereof; and

(c) in the light of (b) above, to make recommendations on suitable measures with a view to promoting public safety and assurance on quality of works.

THE COMMISSION'S DIRECTIONS ON STRUCTURAL ENGINEERING EXPERT EVIDENCE ISSUED ON 29 AUGUST 2019 [OU7/9691-9692]

(Relevant extracts only)

"3. A soft copy of the Southward COI 1 Report **(**as defined in Messrs Lo & Lo's letter of 9 August 2019 [**OU5/3416**] and limited to the list of issues submitted under Messrs O'Melveny's letter of 16 August 2019 **[OU6/3738-3739**]("**Issues 1**")**)** shall be produced by Leighton to the Commission's solicitors by **5:00 pm on Monday, 30 September 2019**. The deadline for the submission of the Southward COI 1 Report as stated in paragraph 4 of Messrs Lo & Lo's said letter [**OU5/3417**] shall be varied accordingly.

.....

5. Leave shall be given to MTRCL and the Government to file an SE expert report respectively in COI 1 ... on Issues 1 ... and in response to the Southward COI 1 Report ... adduced under paragraph 3 ... above. A soft copy of such SE expert reports shall be produced by MTRCL and the Government to the Commission's solicitors by **5:00 pm on Friday, 6 December 2019.**

6. No further SE expert reports may be adduced without the leave of the Commission.

Kindly note that expert [report] of Professor Don McQuillan on matters concerning ... the Holistic Report ... will also be submitted to the Commission on 6 December 2019."

THE COMMISSION'S DIRECTIONS ON STRUCTURAL ENGINEERING EXPERT EVIDENCE ISSUED ON 20 SEPTEMBER 2019 [OU7/9970]

"1. A short extension of time is allowed for Leighton to file the Southward COI 1 Report ....

2. A soft copy of such expert report shall be produced by Leighton to the Commission's solicitors **by 5:00 pm on Friday, 11 October 2019**."

THE COMMISSION'S FURTHER DIRECTIONS ON STRUCTURAL ENGINEERING EXPERT EVIDENCE ISSUED ON 12 OCTOBER 2019 [OU8/10561-10562]

1. The Commission's directions on the filing of SE expert evidence in ... COI 1 ... as set out in Messrs Lo & Lo's emails dated 29 August 2019 [**OU7/9691-9692**] ... and 20 September 2019 [**OU7/9968-9970**] ... shall stand.

2. It is further directed, however, that in relation to the SE Expert evidence to be adduced pursuant paragraph 1 above:

   (a) the SE experts should focus on whether the as-constructed works are safe and fit for purpose from a structural engineering 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 subsequently) are necessary for safety from a structural engineering 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 subsequently) are required for statutory or code compliance.

INSTRUCTIONS

In relation to the Original Inquiry, I have been instructed to give my opinion on the matters under paragraph (a)(1) of the Terms of Reference. I have adduced my first Expert Report on 6 January 2019 (my "**First Report**") [**ER 1/Item 3**]. In addition, I have been asked to review and comment on the Holistic Report[1] [**OU5/3229-3350**] issued by MTRCL dated 18 July 2019.

I am asked to:

(1) respond to the Southward COI 1 Report [**ER2/Item 14**] and the 3 principal topics therein, namely:
- Coupler connections/coupler engagement
- Shear link reinforcement and partial utilization of shear; and
- The horizontal construction joint (CJ) in the EWL slab-to-D-wall connection; and

(2) comply with the Commission's Directions of 12 October 2019.

[1] Final Report on Holistic Assessment Strategy for the Hung Hom Station Extension

---

Expert's Declaration

I, PROFESSOR DON McQUILLAN DECLARE THAT:

1. I declare and confirm that I have read the Code of Conduct for Expert Witnesses as set out in Appendix D to the Rules of High Court, Cap. 4A and agree to be bound by it. I understand that my duty in providing this written report and giving evidence is to assist the Commission. I confirm that I have complied and will continue to comply with my duty.

2. I know of no conflict of interests of any kind, other than any which I have disclosed in my report.

3. 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.

4. I will advise the Commission if, between the date of this Report and the hearing of the Commission, there is any change in circumstances which affect my opinion given to the Commission.

5. I have been shown the sources of all information I have used.

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

7. I have endeavoured 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.

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

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

10. I understand that:

    (a) my report will form the evidence to be given under oath or affirmation;

    (b) questions may be put to me in writing for the purposes of clarifying my report and that my answers shall be treated as part of my report and covered by my statement of truth;

    (c) the Commission may at any stage direct a discussion to take place between the experts for the purpose of identifying and discussing the issues to be investigated under the Terms of Reference, where possible reaching an agreed opinion on those issues and identifying what action, if any, may be taken to resolve any of the outstanding issues between the parties;

    (d) the Commission may direct that following a discussion between the experts that a statement should be prepared showing those issues which are agreed, and those issues which are not agreed, together with a summary of the reasons for disagreeing;

    (e) I may be required to attend the hearing of the Commission to be cross-examined on my report by Counsel of other party/parties;

    (f) I am likely to be the subject of public adverse criticism by the Chairman and Commissioners of the Commission if the Commission concludes that I have not taken reasonable care in trying to meet the standards set out above.

Statement of Truth

I confirm that I have made clear which facts and matters referred to in this report are within my own knowledge and which are not. Those that are within my own knowledge I confirm to be true. I believe that the opinions expressed in this report are honestly held.

Professor Don McQuillan
6 December 2019

BACKGROUND

1. The Hearing for the Original Inquiry concluded in January 2019 with several matters still under investigation and consideration by the Government and MTRCL, including:

   a) The opening-up and in-situ testing of the engagement lengths of coupler assemblies

   b) The strength of partially engaged coupler assemblies

   c) The structural adequacy of the top east EWL slab-to-D-wall connections

   d) Miscellaneous repairable defects and, in particular, shear link irregularities in the EWL slab

2. My First Report dated 6 January 2019 dealt only with Areas B and C and in particular with Area C where malpractice had been alleged [**ER1/Item 3/§26**]. It highlighted several key issues in arriving at a conclusion on the safety of the structures, namely:

   a) Structural utilization throughout is generally low resulting in a significant reserve of strength in the various elements [**§§33-41 and §89**].

   b) Dead load accounts for some 90% of the total load and in service live load for only 10% [**§33**].

   c) The structures have already been subjected to a significant part of the live load i.e. the testing and commission of trains [**§6**].

   d) There is no visual evidence of any distress or cracking in any of the structural elements [**§107**].

   e) Ductile-grade couplers are not required where used in the EWL slab to D-wall joint because the geometry of the connection between the EWL slab and the east D-wall is so rigid that it precludes any ductility. The structural "plastic" deformation which might occur during seismic activity will develop lower down the D-wall. In other words normal couplers could have been used in this location [**§89.2**].

f) The top of the EWL slab at each D-wall is always in tension and the bottom is always in compression [**§30**]. Conversely, for the NSL slab the top at each D-wall is always in compression and the bottom is always in tension [**§107**].

g) From the perspective of structural adequacy, no bottom couplers are required in the EWL slab and no top couplers in the NSL slab [**§91**].

h) To comply with code requirements in respect of seismicity/ductility only a maximum of 50% of the EWL bottom and NSL top coupled connections are required [**§89.3**].

3. On the basis of all the evidence available my First Report concluded [**§126**] that:

a) The structural integrity of the EWL and NSL slabs had not been compromised, as a result of changes of detail and sub-standard workmanship incidents, and that there were no safety issues or concerns.

b) Code compliance had been achieved in respect of ductility.

<u>COUPLER IN-SITU TESTING</u>

4. Around the time when the hearing of the Original Inquiry ended, the original PAUT (Phased Array Ultrasonic Testing) proved, as suspected by the variable results, to be inaccurate and unreliable. This resulted in a refinement to the methodology, following which all the opened-up samples were re-tested under the enhanced PAUT and the full programme of testing outlined in the Holistic Proposal was completed.

5. The full results of the enhanced PAUT are contained in Appendix B3 [**OU5/3308-3319**] of the Holistic Report.

6. The acceptance criteria stated by the HyD[2] [**OU1/584**] was for:

[2] Holistic Assessment Strategy for Hung Hom Station Extension Stage 2 Investigation – Results of Verification of Workmanship Quality for Coupler Connections dated 29 January 2019.

- A maximum of two full threads exposed i.e. 8mm; and
- A minimum engagement of 40mm i.e. 10 threads with an allowable tolerance, due to the Enhanced PAUT technique, of 3mm, i.e. almost one T40 thread, giving a minimum acceptable engagement length of 37mm.

7. The explanation for this tolerance is stated in the revised method statement no. MS-155 of AES Destructive & Non-Destructive Testing Ltd at 7.4[3] [**OU2/896**] as "…*this testing may involve an error of +/- 3mm*." It is difficult to comprehend, however, how a method, which reports results to an accuracy of one tenth of one mm can be subject to a tolerance of 3mm.

8. Notwithstanding, validation of enhanced PAUT readings was carried out against Police actual measurements from unscrewed samples[4][**OW34-35**][**OW53**]. Table 2 [**OW53**] shows that 5 valid samples in Stage 2 and 6 valid samples in Stage 3 were, according to the Police, within the 3mm tolerance criteria leading to the conclusion at §5.1 [**OW35**] that "*…the enhanced method statement for PAUT is effective and reliable for measuring the embedded length of threaded bar in a coupler within +/- 3mm tolerance.*"

<u>MTRCL partially engaged coupler testing</u>

9. MTRCL subsequently proceeded to carry out a comprehensive suite of tests on partially-engaged couplers using two different HOKLAS approved centres:

- The MTR Corporation Project Laboratory ("**MTR Lab**"); and
- Geotechnics & Concrete Engineering (H.K.) Ltd ("**GCE**")

10. The coupler assembly tests comprised:

- Tensile strength test (MTR Lab and GCE)[**OU2/ 907.30-32**][**OU2/ 907.34-39**]

[3] Investigation Report on the issue of discrepancies found in PAUT on site assessment of coupler connections
[4] MTRCL Executive Summary for the event of discrepancies found in Phased Array Ultrasonic Testing on site assessment of coupler connections dated 13 March 2019

- Permanent elongation test (MTR Lab and GCE) [**OU2/907.30-32**][**OU2/907.34-39**]

- Cyclic tension and compression test (GCE) [**OU2/ 907.44-45, 907.46-61**]

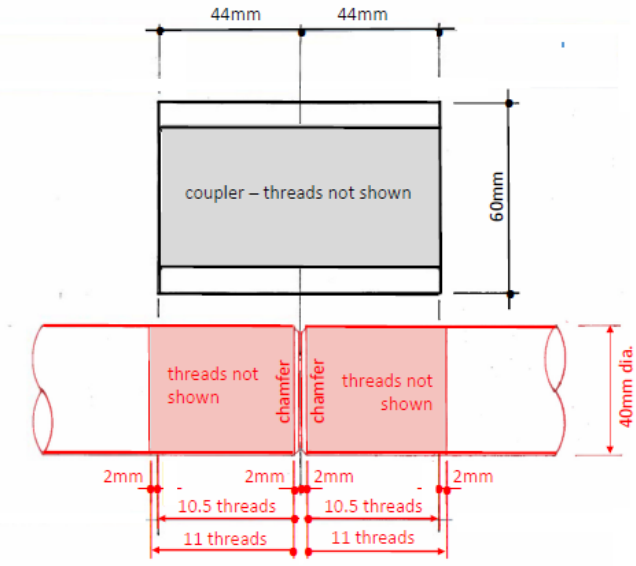
- Static compression test (GCE) [**OU2/ 907.45**]

The formal test reports from MTR Lab and GCE may be found at **OW1/91-119, OW1/239-268**.

11. By way of recap, BOSA[5][**A1/556-684**] show the total threaded length of a T40 bar end [**A1/595**] as 44mm ("t") plus a tolerance of 4mm ("TOL") i.e. a total of 48mm. This appears to be at variance with the statement at 3 of their SUMMARY [**A1/594**] where they state "*Under normal circumstances, we provide a positive tolerance of half a thread.*" BOSA's diagram [**A1/595**] shows a flat end on the threaded section of the bar and it appears that this is schematic only and that, in effect, as will be demonstrated next that the threaded length shown includes the bar end chamfer.

12. It is important, in the context of the enhanced PAUT testing methodology, to understand the terminology used and the relationship between the term "engagement" and the number of threads "effectively" engaged inside the coupler as now used consistently in the various test reports[6] [**OU2/907.7-907.10**]. The following sketch diagram which I have prepared shows the presence of a positive chamfer at the end of the threaded steel bars. It is not detailed enough to show a half-thread at the bar end which is discounted (and therefore not "effective") in terms of the structural contribution of the threads. By way of explanation, when the PAUT signal picks up the first thread (next to the bar end chamfer) the reading (from the start of the chamfer) can be between 0-4mm (1 thread) depending on the bar rotation and the thread start point. My understanding, therefore, is that the average part-first thread is taken i.e.2mm (0.5 thread).

[5] BOSA Technical and Quality Assurance Manual
[6] MTRCL's letter to HyD dated 15 February 2019



BOSA SEISPLICE TYPE 2 COUPLER ASSEMBLY [T40]

13. In an equal "butt-to-butt" connection, 10 "effective" threads inside a coupler equates to a "44mm engagement" i.e. [(10 threads)(4mm) + 2mm internal bar chamfer + 2mm bar end ineffective half-thread]. Similarly:

- 9 threads = 40mm engagement
- 8 threads = 36mm engagement
- 7 threads = 32mm engagement
- 6 threads = 28mm engagement

14. Both laboratories tested nine samples (a total of 18 no.) for each of:

- 6 thread engagement
- 7 thread engagement
- 8 thread engagement

15. The GCE results [OU907.34-39][OU907.45][OU907.47-50][OU907.55-62] and the MTR Lab results [OU907.7-10][OU907.30-32] taken together, demonstrate that a 6

thread engagement (28mm) is an unreliable acceptance criterion because the full tensile strength is not achieved in any of the samples tested.

16. A 7 thread engagement very nearly passes with 12 out of the 18 achieving the full tensile strength. There were three coupler failures and three thread failures.

17. An 8 thread engagement, however, consistently achieved the full tensile capacity of the grade 460 rebar with the fracture occurring every time in the bar as intended.

18. By interpolation it is therefore reasonable to assume that a 7.5 thread engagement (34mm) also achieves full tensile capacity. This was, in fact the acceptance threshold which I used originally. It was based on a least engagement of 32mm in the safety critical EWL top slab-to-D-wall connection as identified by the erroneous original PAUT results. It is not unreasonable then to review the enhanced PAUT results based on a conservative minimum engagement of 34mm (7.5 threads).

19. Only one MTR Lab sample out of 27 and, similarly, one GCE sample out of 27 passed the permanent elongation test criterion of 0.1mm. This test measures the non-recoverable stretch of the bar at 0.6 times the characteristic strength/load. I suspect that rather than elongation of the bar occurring, the recorded movement is due to initial thread slippage of the laboratory-generated samples. In a fully engaged "butt-to-butt" laboratory situation, the bar threads lock into the coupler threads imparting a small amount of pre-load. On site, if a partially engaged bar is screwed into the coupler until resistance is met, the threads still lock and, in my opinion, prevent initial slippage.

20. In the partially engaged laboratory situation, however, the initial very small amount of "slack" is taken-up as the tensile load is applied. Because utilization is generally low, the EWL top rebars are operating at low stress levels so, from a structural performance and safety perspective, the permanent elongation test is irrelevant.

21. If any such slippage occurred, it would only be regarded as significant if hairline-minor cracking was evident in the top of the EWL slab at the D-wall connection running parallel with the underlying-wall in those 10 panels where top coupled connections were used. The inspections carried out to-date have yielded no evidence of any such cracking. Even if it had occurred the cracking would not progress and is

in an internal environment so as not to allow water penetration into the underlying rebar and compromise durability. It should also be emphasised that coupler assemblies tested "free" in the laboratory will behave differently and give more conservative results than couplers which are encapsulated in concrete.

22. In respect of the GCE cyclic tension and compression tests, the results [**OU2/907.45-62**] show that a 6-thread (28mm) engagement was unacceptable whereas an 8-thread (36mm) engagement passed. For a 7-thread (32mm) engagement, seven out of the nine tests passed with two samples exhibiting screw thread and/or coupler failure. The cyclic test is one based on metal fatigue. In other words fatigue failure of a steel rebar can occur if operating at high stress levels over a high number of tension and compression load reversals. This test is therefore not relevant to the EWL or NSL slabs because stress reversal will never be experienced. Fatigue failure is therefore not an issue.

23. The GCE static compression test results were satisfactory for the 6-thread (28mm), 7-thread (32mm) and 8-thread (36mm) engagements. Likewise, however, a static compression test is irrelevant because the concrete itself at the bottom of the EWL and the top of the NSL slabs, where connected to the D-walls, is always in compression and no couplers are required except to satisfy code ductility requirements.

<u>LCAL partially engaged coupler testing</u>

24. LCAL also carried out comprehensive independent testing of partially engaged coupler assemblies using CEEK[7] [**OU7/9752-9803**]. Trial reinforced concrete slab panels were cast with T40 coupler assemblies in the top and bottom at similar depths to what was actually constructed in the works. Opening-up was carried out to expose the embedded couplers which were then PAUT tested using similar equipment to AES. The couplers were then cut open and the engagement was physically measured.

[7] CEEK Report on Technical Review of Coupler Testing Rev B dated 13 June 2019

25. In respect of the elongation tests, CEEK even part-filled couplers with grit to simulate site conditions where couplers might not be entirely debris-free but the operators screw in the couplers until resistance is met.

26. The key results are summarized at Sections 4.3.2 and 4.3.3 [**OU7/9764-9765**] as:

- Couplers partially filled with grit, with the bars screwed in tightly, passed the elongation test.
- Otherwise, only fully engaged "butt-to-butt" (44mm) coupler assemblies passed the elongation test.
- For partially engaged couplers under test, the permanent elongation always occurred at low load thus proving that the threads were "taking up the slack" initially.
- Couplers with a minimum 50% bar engagement i.e. 5 threads (25mm) exceed the required ultimate tensile strength.

27. CEEK also express their opinion at Section 4.3.1 [**OU7/9763**] that the couplers in the EWL and NSL slab-to-D-wall connections are not required to satisfy ductility criteria.

<u>THE HOLISTIC REPORT [OU5/3229-3350]</u>

28. The Holistic Report describes [**OU5/3243-3246**] the various stages of review and assessment as outlined in the Holistic Proposal:

- Stage 1 – a desktop exercise to identify gaps in the as-built record information [**OU5/3233**].
- Stage 2a – opening-up to verify the as-constructed EWL slab-D-wall connections [**OU5/3234**].
- Stage 2b – opening-up to expose randomly selected coupler assemblies for non-destructive PAUT of engaged length [**OU5/3234**].
- Stage 2c – a review of the as-constructed D-wall records [**OU5/3235**].

- Stage 2d – investigation of miscellaneous workmanship defects e.g. shear link misplacement, honeycombing, gaps at the top of columns and walls etc [**OU5/3236**].

- Stage 3 – structural assessment [**OU5/3239**].

29. The summary points arising out of the Holistic Report are:

- The structural assessment was based on the "as-built" geometry and actual construction methodology referred to as the "Updated Design" [**OU5/3279-3283/§§4.3.1-4.3.9**].

- Any couplers assemblies with less than 37mm engagement i.e. any partially engaged coupler assemblies are disregarded as having no contribution whatever to structural performance [**OU5/3276/§4.2.4**].

- This has resulted in the application of strength reduction factors when assessing structural performance of 36.6% for the EWL slab and 33.2% for the NSL slab [**OU5/3235/§10** and also **OU5/3255-3256/§3.3.24**].

- All steel shear reinforcement has been disregarded in the EWL and NSL slabs [**OU5/3278/§§4.2.17** and **§§4.2.18**].

- Notwithstanding these significant reductions, and apart from localized issues at air ducts etc., Areas B and C are structurally adequate. It is stated, however, that defective workmanship identified at sections of the embedded horizontal construction joint at the EWL slab-to east D-wall connection requires compensatory remedial work [**OU5/3266/§3.5.36**] [**OU5/3278/§4.2.20**] [**OU5/3283-3284/§4.3.9, §4.4.5**] [**OU5/3286/§5.3**].

- On the basis of partially-connected coupler assemblies discovered "accidently" in HKC (which is of different construction to Areas B and C), and with no testing whatever carried out in Area A, sections of the EWL slab connections in Area A (which is of similar construction to Area HKC) at the western D-wall capping beam are stated to be structurally inadequate with strengthening required [**OU5/3235/§13**][**OU5/3253/§3.3.19**][**OU5/3256/§3.3.26**] [**OU5/3276/§4.2.3**] [**OU5/3283-3284/§4.3.7, §4.4.2, §4.4.4**].

- In respect of this particular aspect of recommended strengthening it is to be highlighted that the strength reduction factor used in the assessment is approx. double the value used elsewhere i.e. 68.3% [**ER1/Item 12/§4.2.6**] [**ER1/Item 10/§4.3.8**]. In other words the strength of that connection has been downgraded by 68.3%.
- On the basis of zero contribution from the existing slab shear reinforcement, it is stated that strengthening is required in parts of Areas A, HKC, B and C [**OU5/3282-3283/§§4.3.6-4.3.7**] [**OU5/3284/§§4.4.2-4.4.5**].
- In addition to the enhancement measures proposed it is also recommended that long-term monitoring is carried out [**OU5/3241/§§42-43**][**OU5/3269/§3.6.18**][**OU5/3274-3275/§§4.1.8-4.1.9**][**OU5/3285/§§4.4.8-4.4.9**][**OU5/3286/§5.4**].

30. The Holistic Report concluded that the works were safe only "*…for the purpose of the ongoing construction activities*" thus inferring that the works would be unsafe if put into public use [**OU5/3240/§40**][**OU5/3241/§46**][**OU5/3286/§5.2**]. What the Report does, in essence, is to conflate the prime issues of "safety" and "contractual compliance" under the umbrella of "code compliance". As will be explained later, elements of a structure or even an entire structure can be "safe" even though not 100% "code compliant".

<u>AREA "A" AND AREA "HKC" STRUCTURE</u>

31. Unlike Areas B and C where the EWL slab is 3,000mm thick and the NSL slab is 2,000mm thick, Areas A (GL1 – GL7) and HKC (GL7 – GL15) have shorter spanning slabs, because of the internal wall arrangements which provide intermediate support, with the EWL slab and NSL slab only 1,000mm thick. Typical non-dimensioned cross-sections for Area A and Area HKC are shown in Appendices C1 and C2 [**OU5/3334 and 3336**] respectively of the Holistic Report. These two areas also feature mezzanine slabs.

32. It should be highlighted that, as for the EWL slab in Areas B and C, the top of the slab at the connection with the D-wall capping beam is in tension and the bottom is in compression, albeit the forces are much less.

33. To compensate for the lesser structural mass in respect of resisting flotation uplift forces, these two areas require ballast in the form of non-reinforced mass concrete fill in certain zones.

34. To-date none of the concrete mass fill in Areas A and HKC had been fully installed. The MTRCL addendum[8] [**OU8/10820-10835**] shows the extent of the fill to-date and what has yet to be completed. Unrestricted access to the underside of the EWL slab is therefore still possible.

EXPERT STATISTICAL REPORTS

35. My understanding of the two expert reports by Professor Yin Guosheng [9] [**ER1/Item 12**] and Dr Barrie Wells[10] [**ER1/Item 10**] and the respective evidence given to the Commission is that both experts reflect two opposing sets of instructions. Professor Yin is working off the premise that any coupler assemblies which do not comply with the HyD's acceptance criteria are rejected. Dr Wells challenges some of Professor Yin's statistical derivations and, by comparison, advocates the inclusion of partially-engaged coupler assemblies.

36. The diverging statistical analyses and expert reports do not provide me as a non-statistician with definitive failure rates and consequential strength reduction factors. In other words, they do not inform my structural engineering opinion. I need instead to consider matters from an engineering perspective based on extensive experience.

37. One issue, however, deserves comment i.e. the "doubled" strength reduction factor of 68.3% applied to the EWL slab in Area A. The rationale for this exceptionally high value is attributed to the fact that at the west D-wall capping beam, non-embedded couplers were used to connect horizontal starter bars from the capping beam to the

[8] HUH-Survey Report for existing fill within voids in Area A by MTRCL dated 06 August 2019
[9] Expert Report by Professor Yin Guosheng dated 16 September 2019
[10] Expert Report by Dr Barrie Wells dated 13 September 2019

starter bars for the EWL slab. MTRCL were of the opinion that the bars could be partially engaged on both sides of the coupler and therefore deduced that the bars on either or both sides could fail. The statistical analysis was carried out accordingly by Professor Yin [**ER1/Item 12/Section 4.2**]. When giving evidence to the Commission [**ComT/D5/150:9-154:3**], Professor Yin referred to a sensitivity analysis he had subsequently carried out which suggested the strength reduction factor could be reduced slightly from 68.3% to 56.1%.

38. Leaving aside the admitted higher probability of finding more partially threaded coupler assemblies in the AREA A EWL slab because of the "doubled-up" couplers, this statistical concept, however, is fundamentally flawed from an engineering perspective. If double coupler assemblies like these i.e. with partial bar engagement were tested to failure under tension in a laboratory, thread failure would only occur in the bar end with the least engagement i.e. the "weakest link". Therefore at any doubly engaged coupler assembly in the Area A EWL slab only one side can fail, if at all. It is also possible to conceive a situation in the laboratory where the partial engagement was different on each side but that both the threaded ends would hold and one bar would fail at ultimate load.

39. Leaving aside the validity or otherwise of Professor Yin's statistically-based strength reduction factors (which analyses are premised on only the fully engaged coupler assemblies taking load), it is my opinion that the maximum strength reduction factor MTRCL should apply to the Area A EWL slab should be the same as MTRCL applies to the rest of the EWL slab i.e. 36.6%. By way of emphasis, this is because, in the unlikely event of failure, only one side of a double partially engaged coupler assembly can fail.

ANALYSIS OF ENHANCED PAUT TEST REULTS

40. Appendix II contains the results of my own analysis of the test results contained in the Holistic Report. This is based on actual failure rates and initially makes no attempt to take account of a 95% statistical confidence level. The results are summarised in the table below. The 3mm error tolerance in the enhanced PAUT measurements has been applied equally to all the partially-threaded results.

	EWL slab - top			EWL slab - soffit			NSL slab - top			All locations		
	No. of tests	No of fails	%	No. of tests	No of fails	%	No. of tests	No of fails	%	Total tests	Total fails	%
10-thread engagement	29	5	17.2%	80	19	23.8%	93	23	24.7%	202	47	23.3%
9-thread engagement		4	13.8%		8	10.0%		4	4.3%		16	7.9%
8-thread engagement		4	13.8%		7	8.8%		4	4.3%		15	7.4%
7.5-thread engagement		4	13.8%		7	8.8%		3	3.2%		14	6.9%

41. Clearly there is not much difference between a 7.5 thread and an 8 thread engagement so a lower bound 6.9% overall failure rate is adopted. Indeed there is little difference between a 7.5 thread, 8 thread and 9 thread engagement in terms of failure rates.

42. The binomial statistical analyses carried out by Professor Yin, include an uplift on the actual failure rates, to account for a 95% confidence level, which produces the strength reduction factors MTRCL state in the Holistic Report. The NSL slab top failure numbers tally but the EWL slab soffit failure numbers are grouped differently compared with mine and show 25 failures from 90 tests as follows:

- 27.8% (EWL soffit) to 36.6%; an increase of 31.6%
- 24.7% (EWL soffit) to 33.2%; an increase of 0.344%

43. Although not statistically accurate, it is not therefore unreasonable to uplift my actual overall failure rate (and hence strength reduction factor) of 6.9% for a 7.5 thread engagement by say 35% (to achieve an approximate 95% confidence level). This yields a conservative strength reduction factor value of 9.3% i.e. (6.9)(1.35) and represents an engineer's approximate "sanity check" approach.

44. Dr Wells in his statistical report at §3.4 [**ER1/Item 10**] concludes that the strength reduction factors for the EWL and NSL slabs should be 14.5% and 6.5% respectively. He summarizes "*In my opinion the correct approach is to take the combined sample resulting in a strength reduction factor of 9.4%.*" This (i.e. 9.4%), coincidentally, is almost exactly the same value as that derived from my approximate "sanity check" analysis.

45. In respect of the strength reduction factor of 68.3% imposed on Area A EWL slab, Dr Wells at §3.5 [**ER1/Item 10**] opines that a more realistic value of 46.7% is appropriate. For reasons stated above at paragraphs 38 and 39, it is my opinion from an engineering perspective (although not in agreement with the strength reduction factors derived by Professor Yin) that 36.6% is the maximum value that should be applied (because only one side of the coupler assembly will fail, if at all).

46. Notwithstanding, and by way of emphasis, it is my considered opinion that 9.3% is the appropriate strength reduction factor to be applied throughout based on my analysis of the actual failure rates adjusted approximately to give a 95% confidence level.

OPINION

47. It is significant in respect of my original opinion on safety, and that expressed by the Commission in its Interim Report [**A2/711-879**], that in the wake of all the coupler investigation and testing that has been carried out, and all the statistical analysis which has been implemented, and the consequential very conservative structural assessment which is reflected in the Holistic Report, that Areas B and C which were the focus of the Original Inquiry have emerged with almost a "clean bill of health".

48. The spotlight now shines on Area A in particular which presents a conundrum in that no invasive investigation has been carried out by MTRCL to prove the presence of defective construction despite the fact that the soffit of the EWL slab at the D-wall connections is still accessible as a result of the non-completed mass concrete fill [**OU8/10818-10835**]. Rather very limited evidence found in Area HKC has been extrapolated to infer that Area A has similar defects. That might be a logical deduction of statisticians but it is totally unjustified and without merit, particularly when the evidence presented to the Commission confirmed that Area A was built circa May-July 2015, almost one year before Area HKC which was constructed circa July-August 2016, and by different personnel [**B5/2902**]. Definitive evidence of defects alleged to be latent in Area A can only be proved by carrying out specific investigation in Area A itself.

49. I will now deal in turn with each of the three main issues raised by Mr Southward. Before doing so, however, I will address the inference contained in the Holistic Report that the structures are only safe if they comply with the Code of Practice for Structural Use of Concrete 2004 (Second Edition) ("**HKCOP**")[**H8/2818-3015**], the New Works Design Standards Manual ("**NWDSM**")[11] [**OU6/3753-3920**] and general best practice. It is noteworthy, for example, that §2.1.1 of the HKCOP [**H8/2837**] states "*The aim of design is to ensure that…a structure will…perform satisfactorily…*" and *"remain <u>fit for the purpose</u> of its intended use…"* (my underlining).

<u>"Safety" v "code-compliance"</u>

[11] New Works Design Standards Manual: Section 4 Civil Engineering, Version A4 (2009) by MTRCL

50. The term "safety" implies that there is no risk of collapse to a structure when subject to the full loading regime to which is specified. A structure can, however, be safe and not code-complaint and can therefore be "fit for purpose" provided that durability and consequential longevity are not compromised.

51. The HKCOP, like any other "limit state" code, arrives at its minimum standard safety by applying mark-up factors to both materials i.e. concrete and steel, and also to the applied loadings to cater for uncertainties and variations etc. This means that if one was to design a structure "to the bone" in compliance with code requirements, it would not only be safe but have a significant reserve capacity.

52. Because of climate change and the need to design more efficiently to reduce embodied carbon, there is, however, a growing trend towards "performance based" design. For example, an SEI (Structural Engineering Institute – a member body of the American Society of Civil Engineers) publication entitled "Advocating for Performance Based Design" published in April 2018 states that "*Performance-based design is a process that enables the development of structures that will have predictable performance when subjected to defined loading. Performance-based design (PBD) turns the traditional design paradigm upside down in the sense that the required performance is the starting point for the design. Considering the desired performance of the structure and selecting the scenarios that match the goals for structural function in the presence of a specific hazard, the designer works toward achieving that stated, desired goal. The performance of the design is demonstrated through analysis, simulation, prototype testing, or a combination thereo*f."

53. This approach already is well established in fire safety engineering where bespoke performance based solutions are derived from first principles instead of designing to prescriptive codes of practice. In fact, it is also mentioned in the HKCOP under §2.6 entitled "New and Alternative Methods" [**H8/2844-2845**] and in particular under §2.6.3 [**H8/2845**] entitled "Performance based approach".

54. In summary therefore it is acceptable to design from "first principles" and, for example, to apply loads to which the structure will actually be subjected instead of

"building-in" robust margins of safety by incorporating generically specified loads which will never be realised in practice.

1 – COUPLER CONNECTIONS/COUPLER ENGAGEMENT

55. This is the first core issue for consideration as per my brief.

HyD acceptance criteria

56. Firstly it is important to understand the implications of the HyD acceptance criteria. The intention appears to be for a "butt to butt" engagement of the two bars inside any coupler. This "locks" the bars against the threads and prevents any slippage as tensile load is applied. If subjected to a tensile load test, the coupler assembly is intended therefore to pass the permanent elongation test as required by, and in order to comply with, the HKCOP.

57. This requirement is confirmed in the response by MTRCL to Question 11 of RFI 2 ("**Request for Information**")[12] **[OU5/3730]** where it is stated "*…the test results indicated the partially engaged couplers failed to meet the elongation requirement as required by the Government…partially engaged couplers are not considered to be Code compliant.*"

58. The Coupler Engagement Calculator below, which I have prepared, shows the range of engagement for a "nominal" 11 thread T40 bar in relation to the HyD acceptance criteria ("nominal" meaning 10 threads plus one thread for positive roll-off tolerance).

[12] RFI issued to MTRCL on 8 August 2019

COUPLER ENGAGEMENT CALCULATOR

11 threads (T40) with thread pitch	4 mm
Bar end chamfer	2 mm
Coupler half length (T40)	44 mm

Length engaged (mm)	Threads engaged (No.)	Length exposed (mm.)	Threads exposed (No.)	Length inside (mm.)	Threads inside (No.)	Bar end to C/L (mm)	Bar ends gap (mm)	Acceptance criteria 2 threads external	Acceptance criteria 37mm internal
44	10	2	0.5	42	10.5	0	0	Yes	Yes
43	9.75	3	0.75	41	10.25	1	2	Yes	Yes
42	9.5	4	1	40	10	2	4	Yes	Yes
41	9.25	5	1.25	39	9.75	3	6	Yes	Yes
40	9	6	1.5	38	9.5	4	8	Yes	Yes
39	8.75	7	1.75	37	9.25	5	10	Yes	Yes
38	8.5	8	2	36	9	6	12	Yes	Yes
37	8.25	9	2.25	35	8.75	7	14	No	Yes
36	8	10	2.5	34	8.5	8	16	No	No
35	7.75	11	2.75	33	8.25	9	18	No	No
34	7.5	12	3	32	8	10	20	No	No
33	7.25	13	3.25	31	7.75	11	22	No	No
32	7	14	3.5	30	7.5	12	24	No	No
31	6.75	15	3.75	29	7.25	13	26	No	No
30	6.5	16	4	28	7	14	28	No	No
29	6.25	17	4.25	27	6.75	15	30	No	No
28	6	18	4.5	26	6.5	16	32	No	No

59. Assuming both bars have 11 threads (44mm) and are centred in the coupler, each bar has 10 actual threads "engaged" (40mm) because of the bar chamfer of 2mm at the centre of the coupler and discounting the inner bar end half-thread (2mm). This means only 0.5 thread (2mm) is exposed beyond the outer edge of the coupler. If the bar had 12 threads (48mm), which would be unusual, there would be 1.5 threads (6mm) exposed beyond the outer edge of the coupler.

60. So, if such a coupler assembly i.e. having bars with 11 threads has 2 threads (8mm) exposed, the bars cannot be "butt to butt" internally. Assuming again both bars have 11 threads (44mm) and are centred in the coupler, the gap dimension between the ends of the two bars, for 2 threads to be exposed externally, is 3 threads (12mm) i.e. 1.5 threads (6mm) each side of the coupler centre line. Similarly if the bar had 12 threads (48mm), for 2 threads to be exposed externally, the gap is 1 thread (4mm) i.e. 0.5 threads each side of the coupler centre line. A 12 thread bar fully engaged "butt to butt" requires 1.5 threads (6mm) to be exposed.

61. Then consider the situation where the enhanced PAUT engagement measurement is exactly 37mm for a bar with 11 threads (44mm) and there is no 3mm error i.e. the actual engagement is 37mm. This means that 9mm (just over 2 threads) would be exposed, which does not comply with the acceptance criteria and is possibly the reason why there is a slight overlap in the two criteria. Compliance would require 38mm engagement. The internal distance from the bar end to the coupler centre line is then 6mm and the gap between two equally centred bars is 12mm.

62. Looking at this another way, an "engagement" of 38mm means that only 34mm of thread is deemed to be engaged inside the coupler (34mm thread + 2mm bar chamfer + 2mm inner bar end half thread). 34mm equates to 8.5 threads.

63. The HKCOP at §3.2.8.2 [**H8/2853**] requires coupler assemblies in tension to achieve a permanent elongation not exceeding 0.1mm. The testing carried out by MTRCL has clearly demonstrated that partially engaged couplers i.e. not "butt to butt" cannot meet this criterion in laboratory conditions.

64. It therefore follows that the HyD criteria themselves are not code-compliant in respect of the permanent elongation requirement for a standard 11 thread rebar i.e. a maximum 2 thread exposure (8mm) or an actual 37mm engagement does not achieve "butt to butt" engagement and will inevitably fail any laboratory permanent elongation test and therefore cannot achieve code-compliance. It follows therefore that partially engaged couplers which pass the other requisite tests should not be structurally disregarded because they fail the permanent elongation test.

65. The acceptance criteria of 37mm, however, equates approx. to an 8 thread engagement. The testing carried out by MTRCL has clearly demonstrated that an 8 thread engagement more than achieves the requisite bar strength[13] [**OW1/258**].

66. There can therefore be no argument that partially engaged coupler assemblies of at least 8 threads, and probably 7 threads, are structurally adequate and meet the HyD's inferred acceptance criteria expectations.

---

[13] GCE Report on Tensile Test of Reinforcing Bar with Mechanical Coupler dated 4 May 2019

67. Another factor to be considered is the type of coupler used. As previously mentioned, there was no requirement to use ductility-grade couplers in the EWL and NSL slabs at their junctions with the D-walls. By comparison the D-walls have "ductile zones" outside of the slab connection interface where ductility-grade couplers were required to satisfy implicit seismic demands in the HKCOP.

68. Both ordinary-grade couplers and ductility-grade couplers are required to comply with the HKCOP permanent elongation criterion and only ductility-grade couplers are required to comply with the fatigue/cyclic loading test. However, nothing hinges on this point because all 8 thread and most 7 thread coupler tests met the cyclic load criteria [**OW1/244-245, 262-268**][14].

<u>**Atkins Stage 3 Assessment Report[15][OU6/4026+]**</u>

69. The Atkins analyses at Appendix B3 entitled "Capacity Considered Coupler Defects…" for the Area A, EWL slab **[OU6/4504-4514]** applies the strength reduction factor of 68.3% as per Professor Yin's statistical analysis. On this basis, where utilization exceeds 100%, i.e. where there is no reserve capacity, Atkins used 30% bending moment re-distribution as allowed by the HKCOP. Refer to §1.7 on page 3 [**OU6/4040**].

70. At §1.7.4 on page 4 [**OU6/4041**], Atkins conclude that, apart from Area A, "*All other slab connections are found to have sufficient capacity to sustain the design loads…*"

71. In respect of Area A, at §1.7.3 on page 3 [**OU6/4040**], Atkins conclude that enhancement i.e. strengthening is required at "*the locations in Area A…where the load capacity of the coupled bars is exceeded, even after moment redistribution, are subjected to 68.3% reduction in the number of effective couplers…*"

72. In my opinion, therefore, the Atkins analysis is unrealistic and hugely over-conservative in ignoring the structural contribution of partially engaged couplers.

[14] GCE Reports on Cyclic Tension & Compression Tests of Mechanical Connector Systems for Steel Reinforcing Bars dated 3 and 4 May 2019
[15] Atkins Stage 3 Assessment Report (Rev A) dated 20 August 2019

73. Section 16 in the Atkins Report entitled "Reliability and Conservatism of Assessment" [**OU6/4128-4139**] is of significance and reads almost apologetically in respect of disregarding partially engaged couplers. It demonstrates much more realistic and pragmatic engineering judgement and experience.

74. At §16.8.1 on page 99 [**OU6/4136**], Atkins state "*The criteria…in the Stage 3 Assessment…conservatively ignore the capacity of partially engaged couplers which can still contribute tensile capacity to the affected connections."*

75. §16.8.3 on page 100 [**OU6/4137**] confirms that the fatigue performance of the coupler assemblies was not investigated and is irrelevant because "*the loading…is…monotonic…*" i.e. experiencing little variation.

76. At §16.8.5.on page 100 [**OU6/4137**], Atkins state that there "*…is a strong case for accepting the ULS (Ultimate Limit State) capacity of all couplers that have 32mm or greater engagement i.e. 7 threads.*"

77. At §§16.8.8 – 16.8.10 on pages 100 and 101 [**OU6/4137-4138**], Atkins also express the opinion that the non-compliant permanent elongation recorded in the laboratory tests are attributable to the initial "slack" in the threads being mobilized when load is first applied.

78. At §16.8.15 on page 101 [**OU6/4138**], Atkins conclude "*…couplers with an engagement length of 28mm and above could be considered as effective at SLS"* (Serviceability Limit State), i.e. 6 threads or above.

79. Finally, in relation to the requirement or otherwise to use ductility-grade couplers, Atkins at §16.9.2 on page 102 [**OU6/4139**] explain that the ductility requirements of the HKCOP were derived for above-ground building structures. Atkins then state at §§16.9.3 – 16.9.5 on page 102 [**OU6/4139**] "*The behaviour of an underground structure such as HUH station, subjected to seismic excitation is different from that of an aboveground structure…the design of the slab/diaphragm wall joints is governed by static load combinations…and not due to seismic demand…the issue of whether Type 2 ductile couplers have been installed at the slab joint connections to the diaphragm walls is not of significant engineering concern since the detailing rules applied for the station design were written for a different type of structure…*"

<u>AECOM Assessment Report[16][OU6/9680 (soft copy only)]</u>

80. AECOM initially carried out an independent design review and structural assessment of the works based on the original design parameters. This was always going to be conservative by nature because it did not take into account "as-built" conditions.

81. The AECOM report states at §6.1 on page 6-3 "*…laboratory testing has demonstrated that with an engagement length of 32mm* (i.e. 7 threads) *or greater…the full design load can be transferred cross the coupled connection.*" The strength reduction factors quoted on the basis of a 32mm engagement are:

- EWL slab 7.1%
- NSL slab 1.2%
- Combined 5.1%

82. AECOM also state at §6.1 on page 6-3 "*A <u>conservative approach</u>* (my underlining) *to adopt the…reduction factors…with <u>37mm engagement length</u>* (my underlining) *were used to check the forces at the slab to D-wall connection. Based upon the analysis…the slab to D-wall connections…were within the design capacities and are considered safe despite the percentage of couplers which could not comply with the supplier's installation requirements.*"

83. Notwithstanding, AECOM further state at §6.1 on page 6-3 "*As advised by MTR, alternative reduction factors of 36.6% for EWL and 33.2% for NSL are to be adopted for the Project.*" In other words, AECOM's engineering judgement was over-ruled by MTRCL in the same way that Atkins were.

84. It appears that, at the time of the issue of their report, AECOM were unaware of, and had not been instructed, to use the stringent 68.3% reduction in the EWL slab to D-wall connection in Area A.

[16] AECOM Final Independent Structural Assessment Report dated 20 August 2019

85. Of interest, AECOM state at §6.3 on page 6-4 "*…there is no requirement …to use couplers to address any ductility requirements on the slabs to D-wall connection.*" This corroborates my opinion on the need or otherwise for ductility-grade couplers as expressed at 2e, 67 and 68 above. This view will be later seen to be supported by most, if not all, the assessment reports.

<u>AECOM Sensitivity Study Report[17][OU6/9681 (soft copy only)]</u>

86. AECOM were also instructed to carry out a separate sensitivity study which "*…covered the review of the as-constructed underground structures to the amended design assumptions/parameters…"* [p.1-1]

87. In their conclusions at Section 7 on page 7-1, the first paragraph states "*With consideration of the reduction factor…based on the coupler partial engagement investigation…the structural design capacity of HUH is adequate in the* (sic*) safety aspect.*"

<u>ARUP Stage 3 Assessment Reports: Volume 1: Design Basis Report [18] [OU6/8580-8752] and Volume 5: Area A[19][OU6/9257-9520]</u>

88. ARUP states at §8.2 on page 38 [**OU6/8620**] "*…the fitness for purpose acceptance criteria has been taken as 7 threads, or 32mm of engagement.*"

89. On this basis ARUP give strength reduction values for Area A, at §8.3.2 on page 39 [**OU6/8621**], and in Table 2 of Appendix C3 on page C-4 [**OU6/8637**] of:

- EWL slab 23%
- NSL slab 12%

In all other areas ARUP's strength reduction values are 12%. The 23% in Areas A and HKC of the EWL slab is because of the double partially engaged coupler issue and the higher statistical probability of not achieving the acceptance criteria.

[17] AECOM Final Independent Structural Assessment (Sensitivity Study) Report for Area A, HKC, Area B and Area C dated 20 August 2019
[18] ARUP Stage 3 Assessment Report: Volume 1: Design Basis Report, Rev F dated 23 August 2019
[19] ARUP Stage 3 Assessment Report: Volume 5: Area A, Rev F dated 23 August 2019

90. As an aide memoir, the comparative Holistic Report Stage 3 assessment strength reduction values for Area A are:

- EWL slab 68.3% (approx. 3 times the ARUP value)
- NSL slab 33.2% (approx. 2.75 times the ARUP value)

91. In conclusion, at §11 on page 43 [**OU6/8625**], it states "*Arup consider the station is fit for purpose and safe…*"

EIC Memorandum[20][OU7/9829-9836]

92. EIC Activities PTY Ltd (**EIC**) reviewed the CEEK report and concluded [**OU7/9831**] "*LCAL undertook testing which demonstrated the full tensile capacity of the coupler is achieved with only 25mm engaged…based on the inaccuracies of the PAUT test (up to 11mm) less than 2% are clearly shown to have an incorrect threaded length.*"

Southward Expert Report[21] [ER2/Item 14.1]

93. Section 6 of the Southward Report refers. I am in full agreement with his comments and conclusions with two minor exceptions.

94. Firstly, at §6.7 paragraph 1 on page 18, it is stated that "*…6 or more engaged threads are safe to be used in the Works…*" Although the CEEK test results support this, the MTRCL tests confirm that 6 threads cannot be relied on. 8 threads is a criterion no one can dispute and 7 threads could arguably be considered to be adequate. By interpolation, I consider 7.5 threads to be safe as per my table at paragraph 58 above.

95. Secondly, at §6.7 bullet 3, it is stated in the context of the cyclic tension and compression test for couplers that "*…the cyclic stress range is much smaller than the test loading regime.*" It should be clarified that the cyclic test has to do with potential fatigue failure of the rebar and is concerned not with fluctuating load but

[20] EIC Memorandum to LCAL entitled "Hung Hom Station – EIC Response to MTR Holistic Assessment – Couplers" dated 29 August 2019
[21] Tony Gee Structural Engineering Expert Report: Original Hearing dated 11 October 2019

rather load reversal i.e. tension to compression to tension etc. Neither the EWL nor NSL slabs will experience stress reversal.

96. Notwithstanding, these two minor variances in my opinion do not impact on the fact that partially engaged couplers do contribute significantly to structural integrity and performance and provide the requisite overall level of safety.

97. Based on the statistical report of Dr Wells, the Southward report at §6.10, last paragraph on page 23 states "*…the defect rate of couplers is no more than 10.2%*" and, on that basis concludes the structures are safe.

CONCLUSIONS ON COUPLER CONNECTIONS/COUPLER ENGAGEMENT

98. To pass a permanent elongation test and therefore be code-compliant, a rebar requires to have a full "butt to butt" engagement within a coupler.

99. If a standard 11 thread bar has 2 threads exposed, as per the HyD's acceptance criteria it is not "butt to butt", will fail the elongation test and be code non-compliant.

100. Similarly if a standard 11 thread bar has an actual engagement of 38mm, which is a 1mm improvement on the HyD's acceptance criteria, it is not "butt to butt", will fail the permanent elongation test and be code non-compliant.

101. In any event, the permanent elongation is not relevant. Likewise cyclic load reversal and compression tests are irrelevant.

102. I am in total agreement with the Atkins statements on the conservatism of their assessment. I am also in total agreement with the independent engineering experts i.e. AECOM, ARUP and Mr Southward that partially engaged coupler assemblies with at least 8 threads engaged, and probably 7 threads, pass the tensile load test and must therefore be included in the capacity assessment of the structures in the determination of safety.

103. The failure rates and corresponding strength reduction factors, particularly the 68.3% used by Atkins in Area A for the EWL to D-wall connections, although statistically derived, are, from an engineering standpoint, totally unjustified and conservative in the extreme.

104. The strength reduction factor for a "double" partially engaged coupler assembly as in Area A cannot, from an engineering theory perspective, be twice that of the other partially engaged couplers because, in the event of failure, only one side will break i.e. the weaker one.

105. The maximum strength reduction factor in Area A advocated by any of the independent experts is ARUP's 23%. AECOM suggest 7% but that may not take into account the double partial engagement issue. My own analysis, purely from an engineering perspective, shows a figure of no more than 9% to 10% is appropriate.

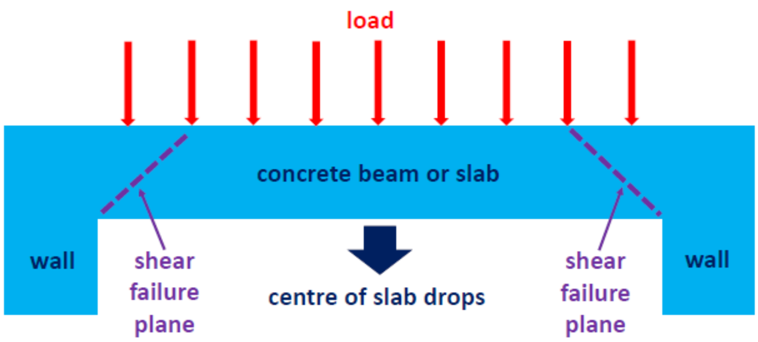
106. In summary, therefore, the independent experts, including me, all conclude the Area A structures are safe even though a low incidence of partially engaged coupler assemblies, which have not achieved a minimum of 7 to 8 threads, has been proven.

107. Had it not been for allegations of malpractice, it is my opinion that the extended Hung Hom Station would have opened as intended, without the detailed invasive investigation that has been carried out, and would be operating successfully.

2 – SHEAR LINK REINFORCEMENT AND PARTIAL UTILIZATION OF SHEAR

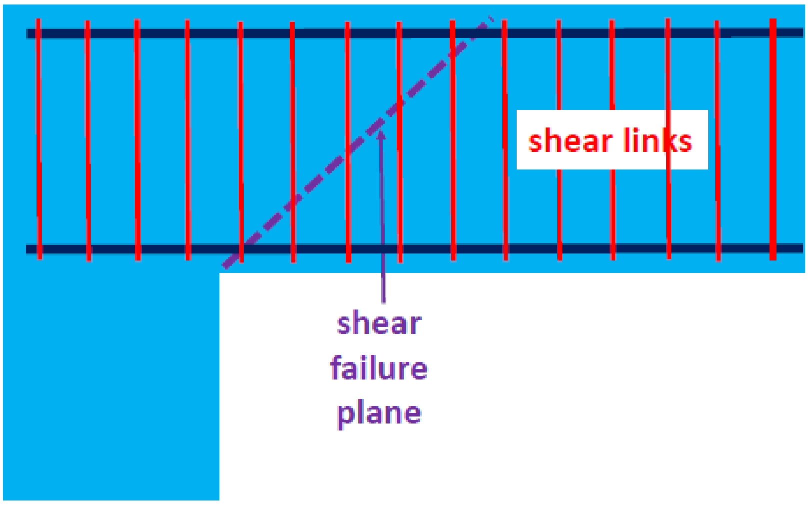
108. This is the second core issue for consideration as per my brief.

109. It may be helpful to initially explain why shear reinforcement is required in a concrete beam or slab element. The top and bottom rebar in a beam or slab, as has been previously discussed, for example in the EWL slab, provides the requisite flexural strength to resist applied load and prevent failure in bending. Another potential failure mode i.e. shear is shown in the simplified diagram below.



110. If the shear stress arising from the applied load exceeds the shear capacity of the concrete, and no shear reinforcement is provided, diagonal shear cracks will begin to form. Failure occurs suddenly when the centre of the slab drops.

111. To prevent the diagonal shear cracks from forming, it is necessary to provide vertical shear rebars, at calculated spacing, across the potential shear planes. These rebars which are normally of small diameter, are anchored to the top and bottom rebars and are commonly referred to as “shear links” and “stitch” the two concrete sections together across the failure plane. This is illustrated in the following diagram. In deeper concrete elements, the bond anchorage of the vertical rebar in the concrete above and below the failure plane also contributes.



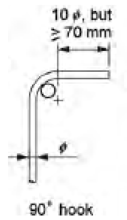
112. By way of recap, although shear links have been provided throughout the works in both the EWL and NSL slabs, minor code non-compliances have been alleged in the Holistic Report. The holistic Stage 3 assessment has completely disregarded the contribution of the shear links installed during the construction. This is confirmed by MTRCL's response to RFI 4[22] [**OU6/9686**] where it is stated "*…the capacity reduction factor of the installed shear links…is 100%.*" In effect the EWL and NSL slabs have been analysed as having no shear reinforcement. This is despite the fact that no opening up was done in Area A by MTRCL where it would have been relatively simple to investigate the presence and nature of shear links by opening-up on the top surface of the NSL slab. It is also in spite of the fact that it would have been much easier to install shear links in the thinner (1,000mm thick) and less congested slabs in Area A.

113. Three types of minor non-compliance have been identified:

- Where a right angled hook is provided at the top and/or bottom of a shear link, Clause 8.5 of the HKCOP [**H8/2945**] specifies the hook length as shown in the copied extract below (from Figure 8.2 – Anchorage of links) to be 10 times the bar diameter. So for a T12 rebar, the required minimum hook length is 120mm. The actual hook length provided is only 70mm. This is thought to be

[22] RFI 4 issued to MTRCL on 20 August 2019

a variation necessitated by the impossibility of being able to install the compliant shear links in the 3m deep EWL slab, from the top, particularly with multiple orthogonal layers of T40 rebar at 150mm centres (i.e. with 110mm clearance) in both the top and bottom of the slab.



- Where the bottom hook is not located on the bottom T40 rebar, but rather is anchored on the next orthogonal bar up i.e. second from the bottom.

- Where smaller diameter vertical bars than those specified have been used.

<u>Atkins Stage 3 Assessment Report [OU6/4026+]</u>

114. Atkins' approach was to treat all the slabs as non-shear reinforced and to identify localized areas where at §1.9 on page 4 of their Stage 3 Assessment Report [**OU6/4041**], "*the ultimate limit state shear stresses exceed the limits specified by the HKCOP, for reinforced concrete without links.*"

115. There is, however, a tacit acknowledgment at §15.3.2 on page 86 [**OU6/4123**] that "*The shear reinforcement demand indicated…is the maximum…with potential for further optimization.*" In other words, their assessment is conservative because they have totally discounted any contribution from the existing shear reinforcement.

116. Atkins have also assessed the shear using the concrete strength originally specified in the design. It should be emphasised that concrete very often is stronger than specified when poured and will also generally experience strength gain in the first few years post-construction. This can be very beneficial when retro-analysing a structure.

117. Section 16 in the Atkins Report entitled "Reliability and Conservatism of Assessment" [**OU6/4128-4139**] is again of significance.

118. At §16.6.1 on page 99 [**OU6/4136**], Atkins state "*The concrete cube strengths sampled…indicate that the actual concrete strengths are typically higher than that specified for design. Typical cube strengths of above 60MPa are common as compared to the specified 40MPa...*" That is a significant increase in strength.

119. At §16.7.2 on page 99 [**OU6/4136**], Atkins further state "*Consideration could be made for assessing the required in-situ concrete strength to eliminate the requirement for shear links as an enhancement proposal."*

120. Moreover, at §16.7.3 on page 99 [**OU6/4136**], Atkins state "*Where shear reinforcement has been installed, but without satisfying the detailing rules for the anchorage of links…the residual strength can be assessed, and the contribution of the partially installed links included in the capacity of the slab.*"

<u>AECOM Assessment Report [OU6/9680 (soft copy only)]</u>

121. AECOM, at §6.5 in their assessment report, on pages 6-4 and 6-5 in relation to concrete strengths for the slabs, state "*For the Grade 40 concrete used on the project, 8,640 cube tests results were available. The average strength for these cubes was approximately 73 MPa and the characteristic strength was approximately 59MPa.*"

122. AECOM conclude at §6.6 on page 6-5 "*Adopting the in-situ concrete strength for design could enhance the concrete shear capacity by about 10%..."*

123. §6.9.2 in AECOM's report deals briefly with the shear capacity and shear links issue. They state "*…only some areas required shear links. For areas requiring shear links, nominal shear links would suffice…except isolated locations where more…than nominal…is required.*"

<u>AECOM Sensitivity Study Report [OU6/9681 (soft copy only)]</u>

124. At §6.9.2 on pages 6-10 to 6-11 of their sensitivity assessment, AECOM then discuss how 18 locations of the EWL slab soffit were opened up (none in Area A however) to check the presence, condition and spacing of shear links. In 4 of the locations it was found by calculation that shear links were required. AECOM then explain that the

shear links in these locations did not comply with the detailing requirements of the HKCOP and recommend that a more sophisticated review be carried out.

125. Based on their sensitivity assessment using amended design assumptions and parameters, AECOM conclude at Section 7 on page 7-1 "*The shear check revealed that the shear links provided in accordance with the original design were sufficient for those areas which required shear links. Most of the areas that require shear links only required nominal shear links.*"

126. On page 7-2, AECOM suggest that the non-compliant shear links issue be addressed by MTRCL and Atkins. AECOM were not, however, instructed to carry out a definitive assessment based on these beneficial attributes.

ARUP Assessment Reports: Volume 1: Design Basis Report [OU6/8580-8752] and Volume 5: Area A [OU6/9257-9520] and Volume 7: Shear Strength Investigation of Slabs and Structural Safety Checks[23][OU6/9606-9663]

127. ARUP at §6.1 in their Design Basis Report on page 33 [**OU6/8615**] state "*Code compliance requires the bottom leg of the link to be anchored by being wrapped at right angles beneath the bottom layer of main span rebar. In a number of instances the legs are misaligned so as not to wrap beneath the lowest bottom rebar. However, because of the multi-layering of rebar, adequate anchorage of these links is provided.*" They further explain "*…in many respects the critical shear load cases have passed as these occurred during construction due to the excavation sequencing and subsequent construction of intermediate vertical loadbearing walls and columns between the NSL base slab and the EWL slab*". This is repeated in the Introduction of Volume 7 on page 1 [**OU6/9608**].

128. The ARUP investigation has been very thorough and detailed, and the key observations are as follows:

- The slabs, because of their geometry and span/depth ratio, act as shallow arches as confirmed by FEA (Finite Element Analysis). However, this

[23] ARUP Stage 3 Assessment Report: Volume 7: Shear Strength Investigation of Slabs and Structural Safety Checks Rev F dated 23 August 2019

substantial enhancement has not been taken into account by ARUP [**OU6/9609-9610/ §2.3**].

- Assessed on the specified concrete strength of 40 MPa, i.e. with no strength gain, code compliance requires a small number of areas to have nominal (i.e. minimum) shear reinforcement of T12 at 300mm centres each way [**OU6/9610/ §2.4**]. These are located in:
    - The Area A NSL mezzanine slab.
    - The Area A NSL main slab.
    - Bottom of the air duct in Area B between GL20 and GL21.
- Shear link non-compliance only therefore becomes a potential issue in these areas where nominal (minimum) reinforcement is required to achieve be code-compliance.
- Importantly, no distress has been observed in the structures [**OU6/9611/ §2.5**].

129. ARUP make the obvious point at §3.1 on page 5 of Volume 7 [**OU6/9612**] "*From simple observation of the density of the bottom mat of rebar, it is difficult to foresee how such a misaligned link could pull through the massive concentration of rebar. The link is effectively anchored without being mechanically attached to the bottom rebar…hence, although…not code compliant, the actual performance of such a misaligned link is not impaired …*"

130. In respect of the shorter shear link anchorage hooks, ARUP (who refer to them as "tabs") state at §3.2 on page 5 [**OU6/9612**] "*The reduction of the tab to 70mm …is…technically compliant with other international codes and standards, for example Eurocode EC2. Therefore, the use of 70mm tabs to links is a perfectly acceptable detail with no loss of strength.*"

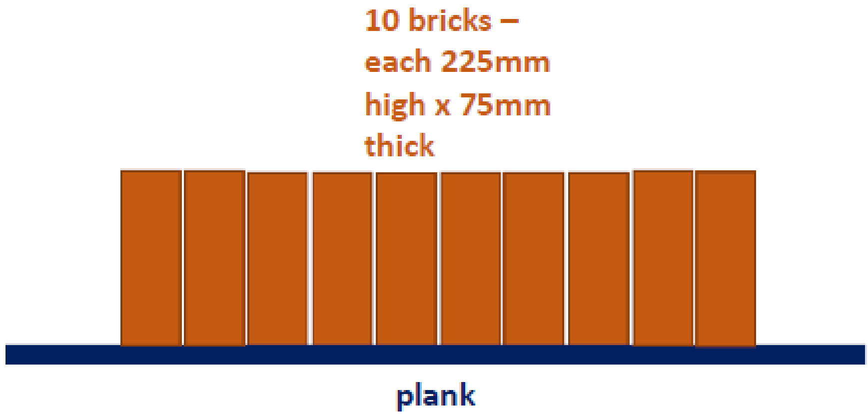
131. In some areas where opening-up was implemented, shear links were either not found at the bottom of the EWL slab or there were less than specified. ARUP opine that, because of the practical difficulties of threading the shear links down through

the multiple layers of heavy slab rebar that the hooks are probably engaging on another layer of rebar further up in the slab soffit [**OU6/9611/§2.6**]. That only becomes a potential problem, however, in those areas identified as requiring nominal shear links.

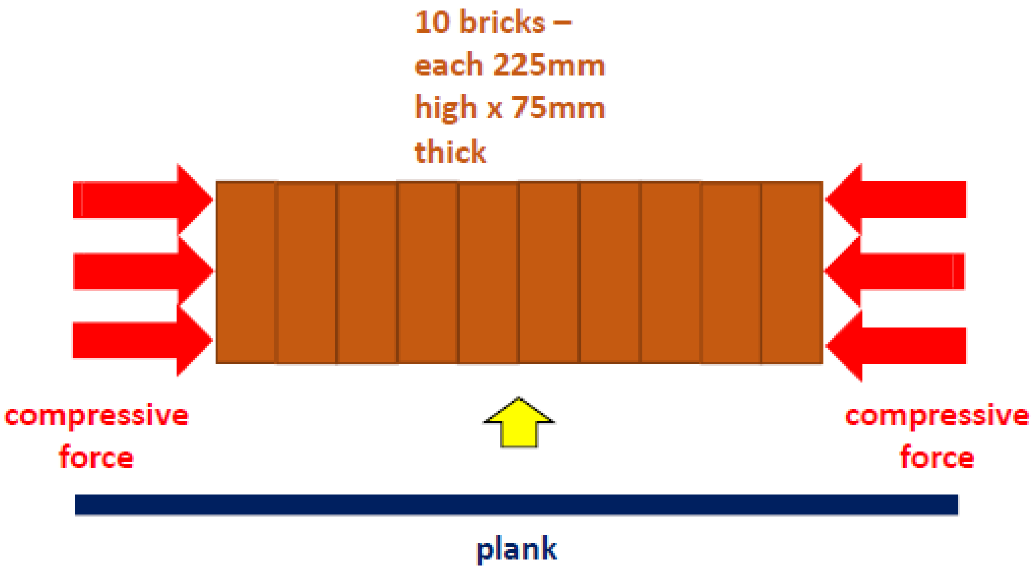
132. ARUP then suggested and assessed some possible mitigation measures to satisfy the HKCOP's minimum requirements. These include:

- Using the proven concrete strength at the time of construction i.e. 60MPa and also allowing a strength gain with time thereafter [**OU6/9614/§5(i)**][**OU6/9617-9618/Appendix A**].

- Using the "compressive membrane action" i.e. arching action of the slabs because they are restrained laterally by the external soil and water pressures [**OU6/9614/§5(ii)**]. This is simplistically illustrated as follows using the analogy of a short row of clay bricks standing upright on a plank where the shear planes are the vertical brick interfaces. Admittedly it better illustrates the concept of axial compression (discussed later) than that of compressive membrane action but to achieve arching action there must be lateral compression at each end.



The bricks can be lifted off the plank as a unit, without them sagging and falling, by squeezing in equally on the end bricks with each hand.



133. Appendix C3 of the Volume 7 report [**OU6/9632-9633**] gives interesting results when a detailed FEA was carried out which investigated different loading conditions and different concrete strengths but which disregarded all shear links and only accounted for the main top and bottom flexural reinforcement:

- Using the specified concrete strength of 40MPa the shear capacity of the slabs are almost double that when evaluated using the HKCOP methods, which are essentially derived from shallow beam theory and are not strictly appropriate for deep continuous slabs [**OU6/9632/ Section C3(i)**].

- Using the proven concrete strength value of 60MPa the slabs have almost three times the shear capacity [**OU6/9632/ Section C3(i)**].

- An approximately linear increase in shear strength occurs with increased concrete strength i.e. from 40MPa to 62.5MPa an increase of some 45% was demonstrated [**OU6/9633/ Section C3(iii)**].

- A modest increase in concrete strength from 40MPa to 45MPa increases the shear capacity by approximately 10% [**OU6/9633/ Section C3(iii)**].

134. In addition to these mitigation methods, ARUP also carried out a Structural Safety Check [**OU6/9639-9663/Appendix E**] in which lower more realistic load factors have been used [**OU6/9614/§5(iii)**].

135. In conclusion, ARUP state at Section 7 on page 9 of Volume 7 [**OU6/9616**] "*…the shear capacity of the construction can be shown to be adequate without shear links. For example, the Structural Safety Checks have shown that shear links are not required in the structure…the structure has a substantial designed reserve of strength and that the structure has for some time been subjected to its full dead loading, including train testing operations and has shown no signs of distress. In conclusion… the structure has adequate shear strength capacity.*"

<u>Southward Expert Report [ER2/Item 14.1]</u>

136. At §5.6.2 and §5.6.3 on pages 8 and 9 of the Southward report [**ER2/Item 14.1**], justifiable criticism is levelled at Atkin's conservatism and reference is made by way of summary to an independent assessment of shear capacity carried out by EIC for LCAL [**ER2/Item 14.1/Appendix D**][**OU8/10724-10732/Appendix B**][**OU8/10734-10739/Appendix C**][**OU8/10759-10770/Appendix F**][24]. It states "*The EIC calculations prove that only 2.5% of platform slab require minimum* (notional) *shear links, out of a total of 23,647m² of slab. This is 0.01% of the total. However…these areas have already been constructed with a satisfactory amount of minimum* (nominal) *shear links…*"

137. At §7.1 on page 25, Mr Southward takes issue with MTRCL's opening-up of the EWL slab soffit locations for the purposes of identifying shear links. It should be noted that no shear link investigation was carried out in Area A. The shallow openings to expose the bottom rebar and shear link hooks, were specified as 1m x 1m [**OU5/3264/§3.5.26**]. In all 18 locations, two sides of the 1m x 1m area were opened to a width of approx. 150mm. This equates to only some 28% of the full area. Not surprisingly this restricted investigation found few shear link hooks.

138. LCAL opened up a 1m x 1m patch in the Area A EWL slab soffit as shown in §7.2 on page 26 [**ER2/Item 14.1**][**OU7/9848**]. This is the only shear link investigation

[24] EIC Report on "Shear Analysis" dated 16 October 2019

carried out in Area A. Shear link end hooks are clearly visible. Superimposed on this 1m x 1m area is the 2-sided opening made by MTRCL. Within the "l"-shaped red lines no hooks are seen. Mr Southward justifiably argues that the failure to open up 1m x 1m areas as specified renders the shear link investigation unreliable.

139. More significantly Atkins, on the basis of this MTRCL shear investigation, decided to completely disregard all the shear link reinforcement throughout the EWL and NSL slabs with the result, as already noted, that Atkins have identified some areas where shear reinforcement is required, but still only the minimum requirement.

140. At §7.4 on pages 27 and 28, Mr Southward justifies the structural contribution of shear links which are anchored on an intermediate layer of rebar instead of the bottom (or top) layer. From a structural perspective the shear links still fulfil the intended function (refer to the diagrams in paragraphs 109 and 111 above) of "stitching" across the potential shear plane provided they anchor at or beyond the centroid of the layers of tension rebar.

141. At §7.5.4 on page 32, Mr Southward calculates the effect of the reduced shear link hook length and concludes "*…the as constructed shear links are adequate for the design and remain in compliance with the HKCOP.*"

142. He then, in §7.6 on pages 33 to 35, majors on the approach taken by alternative international codes, which are based on recent research, including Eurocode EC3, BS8110 and AASHTO LRFD and demonstrates that the as constructed shear links comply with these other codes.

143. Finally in §§7.7-7.9, on pages 36 to 39, Mr Southward describes two alternative methods of assessment of shear capacity. In particular, he relies on work done by Professor Stephen Foster, an internationally recognized expert in the shear behaviour of reinforced concrete [**OU7/9916-9940/Appendix D**]. This assessment was commissioned by EIC on behalf of LCAL [25] [**OU7/9838-9941**] and will be discussed next.

[25] See EIC Report on "Shear" dated 30 August 2019

144. Mr Southward at the end of §§7.8 on page 38 concludes "*In both cases, the provided shear reinforcement is 25.6% more than required, so the reduction in effective anchorage does not compromise the shear strength of the slab and the shear design remains code compliant.*"

145. He elaborates at §7.9 on pages 38 to 39: "*…there is no justification for completely disregarding the shear links in the design calculations for the following reasons:*

- *The limited investigation measures of MTRCL do not prove that the shear links were not installed in the relevant parts of the Works.*

- *In Area A, where MTRCL require most of the suitable measures to be carried out, the only investigation done was by LCAL, one which categorically proves the presence of the shear links*

- *The evidence of the as constructed shear links show that links were used that, although do not comply with the detailing rules of the HKCOP, can carry the design loads due to their over-provision and are therefore compliant with the HKCOP.*

- *The detailing rule for shear links, in terms of the 10 x bar diameter distance required past the end of the bend, is applicable and required for shear links that carry the full ULS design load.*

- *The HKCOP allows modification of anchorage lengths dependent upon the design load in the bars.*

- *I [Mr Southward] have presented the justification for a reasonable extrapolation of the anchorage mechanism through which the shear link works in practice, which means that because of the over-provision of the shear links when compared to the minimum steel requirements, the straight length of the shear links can be reduced to 70mm without compromising the design strength of the structure.*"

In summary, Mr Southward concludes that the shear links are capable of taking the full loading.

<u>EIC Reports on Design Principles[26] and Shear[27]</u>

146. EIC have been equally, if not more thorough than ARUP in their assessment. In their Design Principles report [**OU7/9743-9810**], they set out the factors and methods, in a hierarchical list, by which they have assessed the shear capacities **[OU7/9747]**. Whereas the HKCOP is premised on simplified "fixed angle truss model" theory, they explain in particular how many international design codes have adopted the use of "Modified Compression Field Theory" **[OU7/9750/Section 10]** based on a better research-based understanding of the shear behaviour of slabs and beams. They also point out that "Arch/Compressive Membrane Action" has also been extensively researched and is documented in both UK and Ontario codes [**OU7/9750**].

147. EIC in their Shear report [**OU7/9837-9941**] point out that Clause 2.6 of the HKCOP in fact permits "*new and alternative methods.*" [**OU7/9842/Section 3/2nd paragraph**]

148. They also emphasise that their assessment of shear is premised on "*…the basis of design must provide a structure which can sustain all loads and deformations; remain fit for purpose; have adequate durability etc."* in compliance with Clause 2.1 of the HKCOP [**OU7/9842/Section 3/3rd paragraph**].

149. At the time of writing, EIC had not been provided with the MTRCL Updated Design loads [**OU7/9845/Section 3.4**] and correctly state that, if these are adopted in their assessment, the shear forces will reduce and the slab will have greater shear capacity.

150. In summary EIC concluded:

- <u>Shear Enhancement</u> [**OU7/9845/Section 3.2**] – The original design was conservative compared with the code allowance.

[26] EIC Memorandum to LCAL entitled "Hung Hom Station – EIC Response to MTR Holistic Assessment – Design Assessment Principles" dated 23 August 2019

[27] EIC Report entitled "Hung Hom Station – Response to MTR Proposed Suitable Measures – Shear" dated 30 August 2019

- Axial Compression [**OU7/9845/Section 3.3**] –This was not considered in the original design and an increase of concrete only shear capacity of 5% was confirmed applicable.

- Actual Concrete Strength [**OU7/9846/Section 3.5**] – Compared with the specified design value of 40MPa, EIC evaluated actual concrete strengths, based on test results at the time of construction, of generally 60MPa with 70MPa in the Area A EWL slab. The exception was the Area A NSL slab where, because of a limited number of test results a lower-bound of 48.8MPa was considered prudent [**OU7/9846/Footnote 5**].

- Actual Reinforcement Strength [**OU7/9847/Section 3.6**] – Because the test results confirmed the yield strength of the installed rebar to be higher than the specified 460MPa, EIC confirm an increase of the shear strength of approx. 3% in the concrete.

- Reduced Partial Safety Factors [**OU7/9847/Section 3.7**] **–** correctly highlight the fact that when assessing a structure, as compared with an initial design, some international codes permit a reduction in the partial shear safety factor. EIC chose not to do this because they relied on the higher actual concrete strengths.

- Reduced Anchorage Length [**OU7/9848/Section 3.8**] – EIC rely on a specialist report[28] from Professor Stephen J. Foster, Head of School of Civil and Environmental Engineering at The University of New South Wales, Sydney, Australia [**OU7/9917-9940**]. Professor Foster discusses the influence and analysis of low anchorage shear reinforcement and at Sections 7 and 8 [**OU7/9927-9930**] he provides a design approach with worked example for a 1m deep concrete element with shear links which have no bottom anchorage i.e. to simulate the condition alleged by MTRCL in Area A.

[28] Report by Professor Stephen J. Foster entitled "Mechanisms relating to shear strength of reinforced concrete thick one-way slabs in relation to Hom Hum (sic) Station and the influence of reduced anchorage of shear reinforcement." dated 2 September 2019

On this basis EIC have carried out their assessment on the following basis [**OU7/9848 (bullet points)**]:

a) "*If the anchorage is less than 5 x dia then the effect of the hook should be ignored"*

b) *"If the anchor is between 5 x dia and 10 x dia then the strength of the shear reinforcement should be reduced as a percentage of the anchorage length*"

- Partial Engagement of Shear Reinforcement [**OU7/9849-9850/Section 3.9**] – Again EIC rely on Professor Foster's specialist report and premised on it state "*Even if the end anchorage is ignored, a number of bars will still have an anchorage below the shear plane and hence will still be effective in resisting shear.*" This concurs with my statement at paragraph 111 above.

- Modified Compression Field Theory [**OU7/9850-9851/Section 3.10**] – The underpinning theory is again explained by Professor Foster in his specialist report. EIC state "*The results show a significant increase in the capacity for the 1m deep slab…using methods that incorporate Modified Field Compression Theory when compared to older methods as seen in the HK code…For the 3m deep slab the results are similar between the codes.*"

- Anchorage of web reinforcement in flange [**OU7/9851-9852/Section 3.11**] – This relates to duct areas between GL 30 and GL31. EIC state "…*the vertical reinforcement is T10@200 crs. In their Holistic Report MTR appear to have discounted this reinforcement along with the other shear reinforcement…Photographic evidence of the reinforcement being installed can be seen from the construction records."*

EIC summarize by stating "*EIC consider the…reinforcement effective.*" [**OU7/9852/Section 3.11**]

151. As an overall summary [**OU7/9858/Section 5**]:

- Using the actual concrete strengths and the original design loads (i.e. not the Updated Loads), EIC identify only two locations (SP37 and SP47) which require strengthening (compared with MTRCL's updated assessment of 13 locations).

- If partial shear engagement is also considered, as per Professor Foster's recommendations and proposed methodology, EIC identify only one location i.e. SP37, which requires strengthening

- However if Modified Compression Field Theory is also incorporated into the assessment, EIC find that no enhancement is required and the slabs are safe as-is.

CONCLUSIONS ON SHEAR LINK REINFORCEMENT AND PARTIAL UTILIZATION OF SHEAR

152. The Holistic assessment, because of minor shear link code non-compliances, has treated the slabs as having zero shear reinforcement even though shear links have been confirmed by the limited opening-up. The Report does concede however at §4.2.18 **[OU5/3278]** that "*the installed shear links will provide some strength and hence an additional safety margin to the slab.*" In my opinion that constitutes overt conservatism.

153. No opening-up for shear has been carried out for Area A yet, again, that is where the Holistic assessment alleges the greater deficiency in shear capacity.

154. My unreserved opinion, confirmed by the reports of the independent experts, particularly ARUP, EIC and Mr Southward, is that the shear capacity of the slabs is more than adequate and there is no doubt whatever that structural integrity and safety have not been compromised by the minor code non-compliances.

3 – THE HORIZONTAL CONSTRUCTION JOINT (CJ) IN THE EWL SLAB-TO-D-WALL CONNECTION

155. This is the third core issue for consideration as per my brief.

Background

156. At the hearing of the Original Inquiry in January 2019, three out of four expert engineers, including me, confirmed the horizontal construction joint (CJ) embedded in the EWL slab-to-D-wall connection was of no structural consequence. The Government's expert, however, raised doubts about the structural adequacy and safety of the connection in panels with high utilization to the extent that he advocated the retro-installation of vertical steel dowel bars to bridge the horizontal construction joint to prevent shearing across the joint [**Transcript D40/145:16-21**][**Transcript D44/117:15-24**]. During my oral evidence given to the Commission, I recommended that a FEA (Finite Element Analysis) be carried out to prove the issue once and for all [**Transcript D44/122:7-18**].

157. By way of emphasis the CJ cannot be reviewed in isolation. It is an integral part of the EWL slab-to-east D-wall connection (except for the 10 panels which were constructed to the original detail) and the entire connection therefore needs to be assessed as an entity.

158. The Holistic assessment involved the removal of four 45mm diameter concrete cores, at two locations in the top of the east EWL slab-to-D-wall connection, as recorded in the Holistic Report at §3.5.33 [**OU5/3265**]. VR (Virtual Reality) Scoping was the method used to inspect the construction joint.

159. At one location a gap was observed but no detail is given as to the gap width etc. At another, a piece of hessian was discovered [**OU5/3266/§3.5.34**].

160. Notwithstanding, at §3.5.35 [**OU5/3266**], the conclusion was that "*…there was no signs of movement, slippage or distress of the horizontal joints at the two D-wall panels*" with the cause of the presence of the said gap and hessian material attributed to workmanship issues.

161. This issue was further interrogated by RFI 1 issued to MTRCL on 7 August 2019 [**OU5/3411/Question 3**] in order to obtain confirmation that the issue was only workmanship-related and not, in any way, a strength issue. The MTRCL response to question 3 (c) [**OU5/3435-3437**] indeed confirms that failure of the EWL slab-to-D-wall connection will occur in the D-wall at the EWL slab soffit which, in turn, confirms the weight of evidence given to the Commission at the hearing of the Original Inquiry. The RFI 1 response further states "…*the stress levels at the CJ are low.*"

162. A follow-up request for clarification was issued to MTRCL under RFI 2 on 8 August 2019 [**OU5/3413-3414/Question 6**][**OU3728**] "*Explain* w*hy, if it was concluded that there were no signs of movement, slippage or distress of the horizontal joints…it is considered necessary to install vertical dowel bars as a remedial work solution.*" MTRCL's response confirms the issue is one of defective workmanship and cites non-conformance to the HKCOP, the NWDSM and MTRCL's Materials and Workmanship Specification for Civil Engineering Works [**OU5/3728-3729/Answer 6**]. The HKCOP at §10.3.10 [**H8/2982**], for example, requires that "*The joint surface must be clean and free from loose particles…*" The MTRCL Specification at §8.24(5) [**C5/3717-3718**] states that CJ surfaces "*shall be washed with clean potable water and all loose material removed.*"

163. The justification for insisting on the retro-installation of dowel bars is not therefore premised on structural integrity or safety. Rather, by inference, this is a precautionary measure blamed on D-wall panels which theoretically have high utilization. This is despite MTRCL's confirmation of low stress levels in the CJ and that the D-wall will potentially fail below the connection rather than at the CJ or anywhere in the EWL slab-to-D-wall connection.

164. I do not consider my reference to the non-justified and unnecessary remedial work, i.e. the retro-installation of vertical dowel bars, to be in breach of the Commission's Directions because, the implementation of such measures is likely to compromise the structural integrity and safety of the existing "safe" structures. This is because there is a high risk of causing structural damage during the coring process, including cutting of rebar. As an expert I have a duty of care to state my concern in this respect. The workmanship defects are not impacting on safety or serviceability

or durability and, in my opinion, are therefore best left alone without any intervention.

<u>Atkins Stage 3 Assessment Report [OU6/4026-8578] and EWL Slab / Diaphragm Wall Joint Assessment Report [OU6/3944-4025][29]</u>

165. Atkins carried out a FEA to study the shear capacity and behaviour of the horizontal CJ for the full range of load. In addition, they modelled various CJ arrangements in order to carry out a sensitivity check.

166. In their Joint Assessment Report, Atkins at §1.1.9 on page 2 [**OU6/3950**] and §6.1.8 on page 25 [**OU6/3973**] conclude "*This further study has confirmed that the EWL/Diaphragm wall joint provides a structural capacity at ultimate limit state that exceeds the maximum moments generated from the structural analysis, undertaken as part of the Stage 3 Assessment…"*

167. At §6.1.7(2) on page 25 [**OU6/3973**], they say "*The utilization of the main tension reinforcement within the diaphragm wall remains low."* This is in spite of the absence of inverted "U" bars above the CJ.

168. In the Executive Summary on page D5 of Appendix D to the Joint Assessment Report [**OU6/4004**], Atkins also state in respect of the D-walls "*Bond slip failure in the diaphragm wall outer bars was not apparent, the bar utilization is at very low levels and the bars are anchored properly.*"

169. An interesting and relevant observation is made by Atkins on page D23 of Appendix D to the Joint Assessment Report [**OU6/4022**] where they illustrate the analogy of a wide bridge abutment supporting a shallow deck slab (to the EWL slab-D-wall joint) and say "*Detailed assessment of such arrangements is not usually carried out but the detail occurs very frequently in all heavy civil engineering where irregular aspect ratio connections exist with construction joints.*"

170. In their Stage 3 Assessment Report Atkins, at §1.11.9 on page 6 [**OU6/4043**] state "*The VR scope findings showed that there were no signs of any 'shearing' or 'tearing'*

[29] Atkins Stage 3 Assessment Report (Rev A) dated15 August 2019 and Atkins EWL Slab/Diaphragm Wall Joint Assessment Report dated August 2019

across the faces of the construction joints, leading to the conclusion that no lateral movement of the D-wall had taken place."

<u>AECOM Assessment Report[30] [OU6/9680 (soft copy only)]</u>

171. AECOM did not carry out a FEA, rather, they checked the CJ using the Free Body Diagram method (See Appendix 12). At §6.8.3 on page 6-10 they state "*The horizontal shear stress at horizontal CJs for all panels…were checked within…capacity."*

<u>ARUP Assessment Report: Volume 6 – Integrity and Ductility of Slab / Diaphragm Wall Connections in Areas B and C[31] [OU6/9521-9605]</u>

172. ARUP also carried out a FEA more accurately described as a Non-Linear FEA (NLFEA)[**OU6/9531-9533/§3.4**]. They modelled a "contact surface" at the CJ between the D-wall and the EWL slab and applied a low coefficient of friction to mimic a "smooth" interface to facilitate slippage across the CJ were it to occur.

173. ARUP's analyses confirm that failure will occur in the D-wall and not the CJ or any element within the EWL-to-D-wall connection. They state at §3.4 on page 9 [**OU6/9531**] "*…the failure mechanism is flexural failure in the diaphragm wall at the soffit of the EWL slab…the weakest element in the joint arrangement is the diaphragm wall.*"

174. Bearing-in-mind that the retro-installed vertical dowels are proposed by MTRCL because of "high utilizations", ARUP also conclude from their NLFEA at §5.3 on page 13 [**OU6/9535**] "*…from the results, the utilization levels in all joints are not large, with average values generally less than 60%, and most areas less than 50%, with limited peak values of around 70%. On the basis of these results and the inherent conservatism in the calculation method, the integrity of the joint zone has been demonstrated.*"

<u>Southward Expert Report [ER2/Item 14.1]</u>

---

[30] AECOM's Final Independent Structural Assessment Report for Area A, HKC, Area B and Area C (Rev 1) dated 20 August 2019

[31] ARUP's Stage 3 Assessment Report: Volume 6 – Integrity and Ductility of Slab / Diaphragm Wall Connections in Areas B and C Rev F dated 23 August 2019

175. Mr Southward's company Tony Gee and Partners (Asia) Ltd. has also carried out a comparative FEA with a specific focus on the impact, if any, of a thin gap at the CJ interface as was found in one of the cores [**§8.4/pp.41-46**]. A physical gap between the EWL slab and the D-wall was modelled and compared behaviour-wise with a "no gap" situation.

176. At §8.4.2 on page 43, Mr Southward states "…*the stress distribution for the control* (i.e. "no gap") *and defective* (i.e. with gap) *joint models are almost identical. A slight variation in stress around the construction joint can be seen in the low stress range."*

177. He concludes at §8.4.3 on page 46 "*The presence of a completely unbonded construction joint that cannot transfer shear…is not significant in relation to overall material strength…no impact to the overall joint capacity is anticipated."* In addition at Section 9 on page 48, Mr Southward also states *"The condition of the top of the East D-wall at its junction with the EWL slab is of no structural concern, even if the worst possible assumption is made that there is a physical gap between these elements."*

CONCLUSIONS ON THE HORIZONTAL CONSTRUCTION JOINT (CJ) IN THE EWL SLAB-TO-D-WALL CONNECTION

178. Extensive FEA assessment has been carried out by Atkins, ARUP and Tony Gee with the CJ modelled as "smooth" and with a physical thin gap.

179. The stresses are confirmed by all parties to be low and the structural integrity and safety of the CJ is not therefore in dispute by any party.

180. The proposal to compensate for poor workmanship in my opinion could compromise the structural integrity and safety of the EWL slab-to-D-wall connections and I strongly recommend that this unjustified and unnecessary "token" work is not implemented.

SUMMARY OPINION ON STRUCTURAL INTEGRITY AND SAFETY

181. My summary opinion as expressed in paragraph 126 of my First Report [**ER1/Item 3/p.49**] in respect of Areas B and C was "*In conclusion, on the basis of all the evidence available, I am satisfied and in no doubt that the structural integrity of the EWL slab has not been compromised as a result of changes of detail and sub-standard workmanship incidents, and that there are no safety issues or concerns. I am also satisfied that code compliance has been achieved. The same opinion applies in respect of the D-walls and lower NSL slab. It is highly improbable that the results of further opening-up will alter this conclusion unless further defective top rebar couplers are discovered in the limited number of panels so constructed, in which case bespoke retro-analysis will be required to check the load capacity in those areas.*"

182. Since then Area A in particular has also come under scrutiny. MTRCL, based on their holistic structural assessment, which has been informed by statistical analysis of the coupler assemblies on the basis that partially engaged couplers must be disregarded, and limited opening-up of the EWL slab soffits in respect of shear reinforcement, have significantly "marked-down" the strength of the structures. MTRCL allege the station is only safe in its current state for on-going construction activities [**OU5/3375-3376/Ans to Question 11C**] but not for putting it into full operation and opening it for public use.

183. I have independently and extensively reviewed the assessments of MTRCL's consultant (Atkins) and conclude that they are unjustifiably conservative, on Atkin's own inferred admission, because they are premised on the "marked down" structural strength.

184. I have likewise independently and extensively reviewed the assessments of the other independent engineers i.e. AECOM, ARUP, Mr Southward and EIC who have taken into account the structural contribution of a) partially engaged coupler assemblies; and b) the as-constructed shear link reinforcement.

185. The EWL slab-to-east D-wall connection with its integral horizontal CJ is admitted by all parties, including MTRCL, not to be a structural issue.

186. I am therefore fully satisfied, and in no doubt, that the structures are safe "as-is" and there is no reason why the station cannot and should not be opened for public use.

187. It may be difficult, however, to convince the public that there are no structural integrity or safety issues particularly when there has been so much adverse publicity and because they have been informed that extensive remedial works are required with consequent delay to programme.

188. I therefore recommend, as I did in my First Report, that to allay public concerns, long-term monitoring of the structural performance of the works should be carried out, even though it is not anticipated this will yield any significant results.

189. However, should any enhancement work, about which the public have been informed, be carried out, there would then be no justification, in my opinion, for performance monitoring as well, because the structures would be even safer.

Commission of Inquiry into the Construction Works at and near the Hung Hom Station Extension under the Shatin to Central Link Project (formerly Commission of Inquiry into the Diaphragm Wall and Platform Slab Construction Works at the Hung Hom Station Extension under the Shatin to Central Link Project)

Supplemental Expert Report of Professor McQuillan in Original Inquiry (COI 1) dated 6 December 2019

Erratum

At paragraph 42 (page 23), the second bullet should read "24.7% (NSL top) to 33.2%; an increase of 34.4%"