EXPERT REPORT

EXTENDED INQUIRY (COI 2)

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

Professor Don McQuillan

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.
Meetings, visits and inspections	See **Appendix II**

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 [AA1/277-278]

(Relevant extracts only)

"4. A soft copy of the Southward COI 2 Report (as defined in Messrs Lo & Lo's letter of 9 August 2019 [**AA1/231**] and limited to the list of issues submitted under Messrs O'Melveny's letter of 16 August 2019 [**AA1/239**]("**Issues 2**")) shall be produced by Leighton to the Commission's solicitors by **5:00 pm on Monday, 30 September 2019**.

5. Leave shall be given to MTRCL and the Government to file an SE expert report respectively in ... COI 2 on ... Issues 2, and in response to the ... Southward COI 2 Report adduced under ... paragraph 4 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 Verification 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 [AA1/349]

"1. A short extension of time is allowed for Leighton to file the ... Southward COI 2 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 [AA2/472-473]

1. The Commission's directions on the filing of SE expert evidence in ... COI2 as set out in Messrs Lo & Lo's emails dated 29 August 2019 ... [**AA1/277-278**] and 20 September 2019... [**AA1/347-349**] 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 ... Verification 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 ... Verification Report, or subsequently) are required for statutory or code compliance.

INSTRUCTIONS

In relation to the Extended 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 in the Original Inquiry on 6 January 2019 (my "**First Report**") [**see Commission's website**]. In addition, I have been asked to review and comment on the Verification Report[1] [**BB16/9952-10000**] issued by MTRCL dated 18 July 2019.

I am asked to:

(1) respond to the Southward COI 2 Report [**ER1/Item 10**] and the 2 principal topics therein, namely:
- Coupler connections/coupler engagement; and
- Shear link reinforcement and partial utilization of shear; and

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

[1] Final Verification Study Report on As-constructed Conditions of the North Approach Tunnels, South Approach Tunnels & Hung Hom Stable Sidings

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

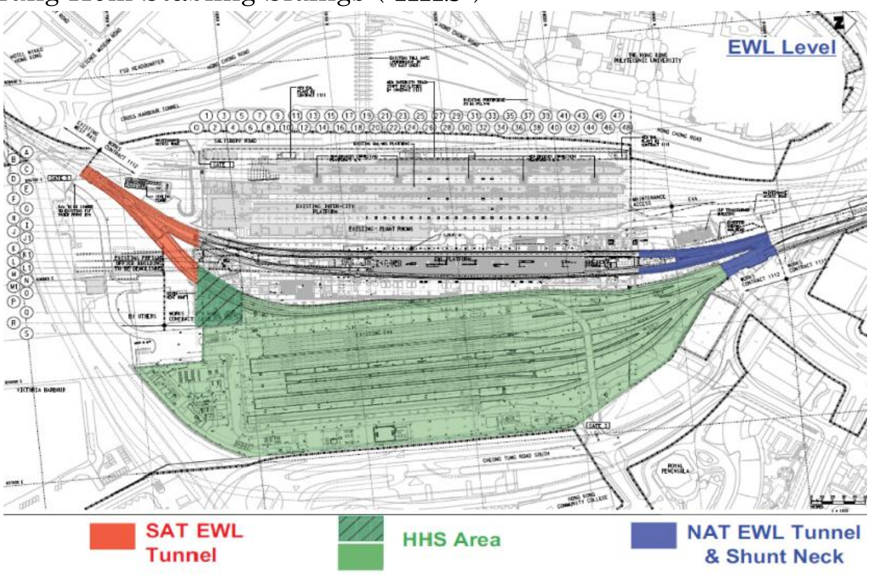
INTRODUCTION

1. I, Professor Don McQuillan, have been appointed as the Commission's independent Expert Engineer, to assist the Commission under the Extended Terms of Reference in determining the matters relating to the North Approach Tunnels, the South Approach Tunnels and the Hung Hom Stabling Sidings construction works at Hung Hom Station. I am a Chartered Engineer specialising in the disciplines of Structural, Civil, Bridge & Marine Engineering, and am a Director of RPS. My opinion and the conclusions reached, as set out in this Report, are formed on the basis of the evidence that I have seen.

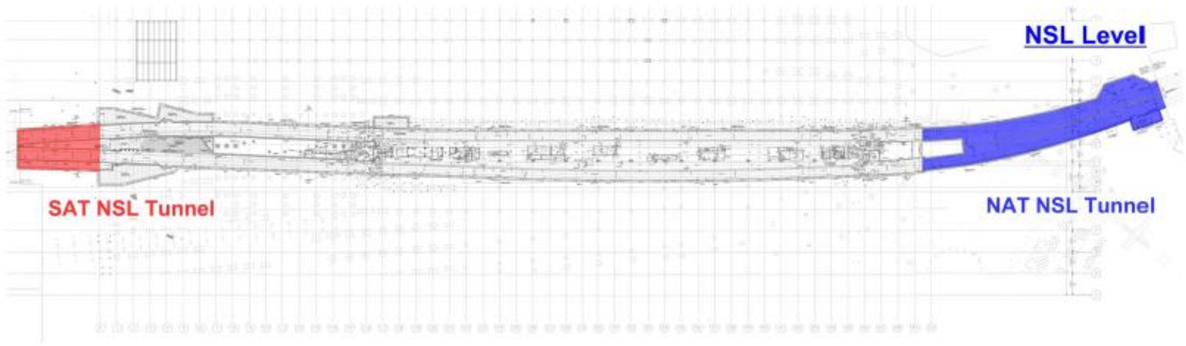
BACKGROUND

2. Following the hearing of the Original Inquiry (COI 1) in January 2019, which focused on the diaphragm wall and platform slabs of the Hung Hom Station ("**HUH**"), the terms of reference of the Commission of Inquiry were extended on 19 February 2019 to cover the construction works at other areas of HUH, located as illustrated on two layout plans courtesy of the MTR Corporation Limited [**BB16/9983-9984**] below [Drawing (1) and Drawing (2)], namely the:

- North Approach Tunnels ("**NAT**"),
- South Approach Tunnels ("**SAT**"); and
- Hung Hom Stabling Sidings ("**HHS**")

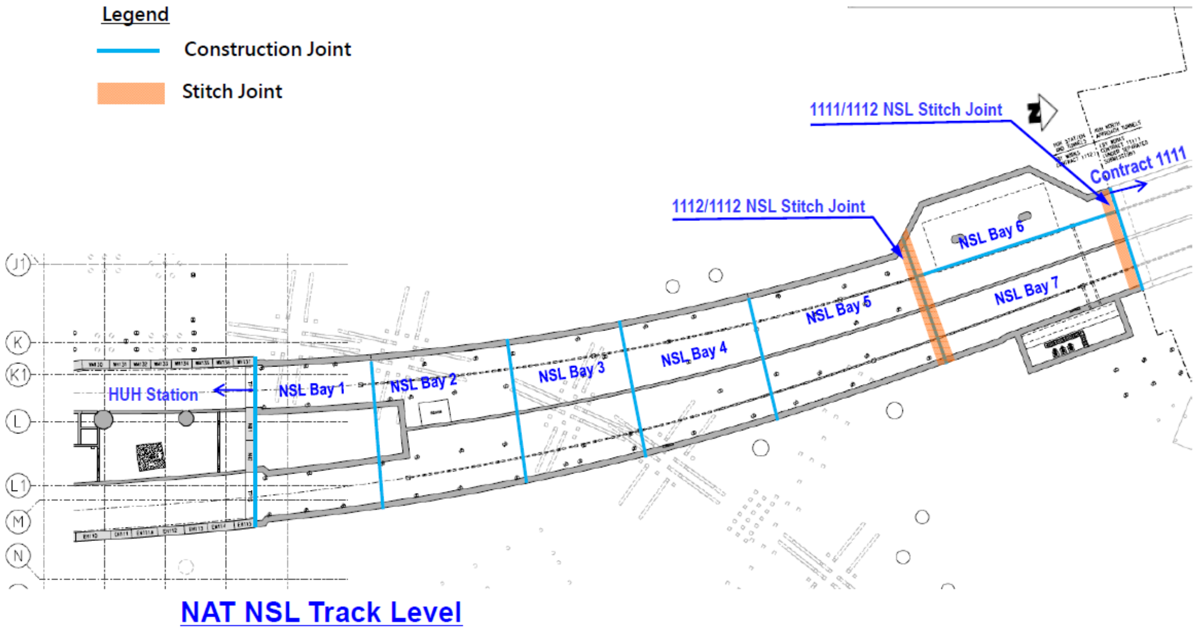


(1) Layout Plan at the EWL level

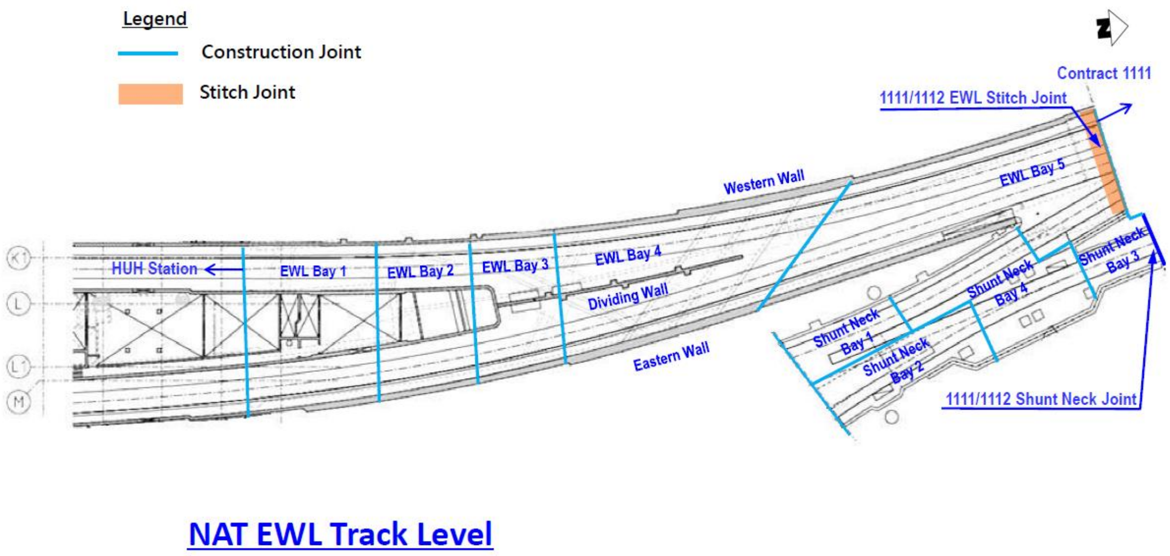


(2) Layout Plan at the NSL level

3. **Issue 1** of the Extended Inquiry is concerned with three stitch joints in the NAT (the "**3 Stitch Joints**") as illustrated on the following 2 layout plans [**BB1/89-90**]. **Issue 2** is concerned with a construction joint in the Shunt Neck as shown in the NAT EWL Track Level layout plan below. Water seepage problems and/or improper rebar connections in these joints led to invasive investigation and the identification of defective workmanship and interface issues.



1111/1112 NSL Stitch Joint ("Joint 1")
1112/1112 NSL Stitch Joint ("Joint 2")



<u>1111/1112 EWL Stitch Joint ("Joint 3")</u>
<u>1111/1112 Shunt Neck Joint</u>

4. Although Issue 1 and Issue 2 were largely closed-out by the main contractor, investigation into those 2 Issues resulted in allegations of the lack of inspection and supervisory records, including RISC Forms, unauthorised design changes (from lapped rebar to the use of mechanical couplers), and incomplete testing records of materials at the NAT, SAT and HHS areas ("**Issue 3**").

<u>Parties involved</u>

5. There are five main parties involved in the Extended Inquiry, the principal three being:

- The Government comprising:
  - Transport and Housing Bureau ("**THB**"), Highways Department ("**HyD**") including the Railways Development Office ("**RDO**"); and
  - Development Bureau ("**DEVB**") and the Buildings Department ("**BD**")
- MTR Corporation Limited ("**MTRCL**") who was appointed by the Government to procure and project manage the design, construction, commissioning and delivery of the SCL Project.

- Leighton Contractors (Asia) Ltd ("**LCAL**") who was the registered Main Contractor appointed to construct the relevant NAT, SAT and HHS works.

These were involved in the Original Inquiry as was PYPUN-KD & Associates Limited ("**PYPUN**") who were appointed by the Government as the Monitoring and Verification (M&V) Consultant to assist HyD with monitoring and verification of certain aspects of the project.

The only party not involved in the Original Inquiry is Wing & Kwong Steel Engineering Co., Limited ("**Wing & Kwong**") who was LCAL's steel fixing sub-contractor for the NAT and HHS areas.

VERIFICATION REPORT [BB16/9952-10000]

6. The Verification Report describes the various stages of review and assessment outlined in the Verification Proposal[2] [**AA1/141-160**] which was approved by the Government on 15 May 2019:

- Part 1a – consolidating and verifying available construction records to identify any gaps in site inspection records, material testing records and design change records.
- Part 1b – formulating and implementing a proposal for reviewing and ascertaining the as-constructed conditions of the structures.
- Part 2 – conducting structural review and devising schematic remedial works and long-term monitoring scheme of structural performance

[**AA1/146-147**].

7. The following consultants were appointed by MTRCL to assist:

- Siu Yin Wai & Associates Ltd ("**SYW**") – to verify available project records and information.
- Atkins China Limited ("**Atkins**") – to carry out a structural review of the as-constructed NAT, SAT and the small green hatched area of HHS shown in the "Layout Plan at the EWL level" above [Drawing (1)].
- AECOM Asia Company Limited ("**AECOM**") – to carry out a structural review of the as-constructed HHS area except for the small green hatched area shown in Drawing (1) above.
- Ove Arup & Partners Hong Kong Limited ("**ARUP**") – to carry out an independent structural review of the NAT, SAT and HHS areas.

8. The summary points arising out of the Verification Report are:

[2] Verification Proposal of As-constructed Conditions of NAT, SAT and HHS (Rev E) dated 15 May 2019

- The structural assessment for the NAT and SAT areas were based on the actual construction methodology and on more favourable updated design assumptions. [**BB16/9975/§§4.2.2-4.2.3**]

- The structural assessment for the HHS area, however, was based on the original design criteria. [**BB16/9975/§4.2.4**]

- Based on the statistical analyses carried out on coupler assemblies, and with no opening-up at all in any of the areas in question (i.e. areas where coupler connections have replaced lapped bars), a blanket 35% strength reduction factor has been applied in all areas for the purpose of the structural assessments [**BB16/9976/§4.2.6**].

- Consequently couplers at kicker level in the HHS trough upstand walls are deemed to have inadequate strength and require remedial strengthening work [**BB16/9978/§4.5.2**].

- Notwithstanding, all other areas i.e. NAT and SAT where coupler assemblies have been used are deemed to have adequate structural capacity [**BB16/9978/§4.5.1**].

- Because of alleged gaps in rebar testing records, the tensile strength of all rebar has been downgraded by the following strength reduction factors:

    - 4% for rebar with diameter equal to or greater than 16mm

    - 13% for rebar with diameter equal to or less than 12mm

- Notwithstanding these reductions, the bending capacity of the NAT, SAT and HHS structures is deemed to be adequate [**BB16/9979/§§4.5.5-4.5.6**].

- For the shear capacity assessment the rebar strength reduction factors (in tension) have also been applied i.e. 4% and 13% [**BB16/9978/§§4.5.3-4.5.4**].

- On this basis of the downgrade, and also because record site photographs fail to show the anchorage of the bottom hook, it is alleged there are shear deficiencies in part of the NSL slab box tunnel area of the SAT which requires remedial strengthening work [**BB16/9970/§§3.2.6-3.2.9**][**BB9980/§5.2**]. This is despite the fact that:

- The rebar was not congested and steel fixing was therefore easier than for the EWL slab
- The slabs and walls were generally a maximum of 1m thick
- The shear links were of a small diameter i.e. 10mm to 16mm

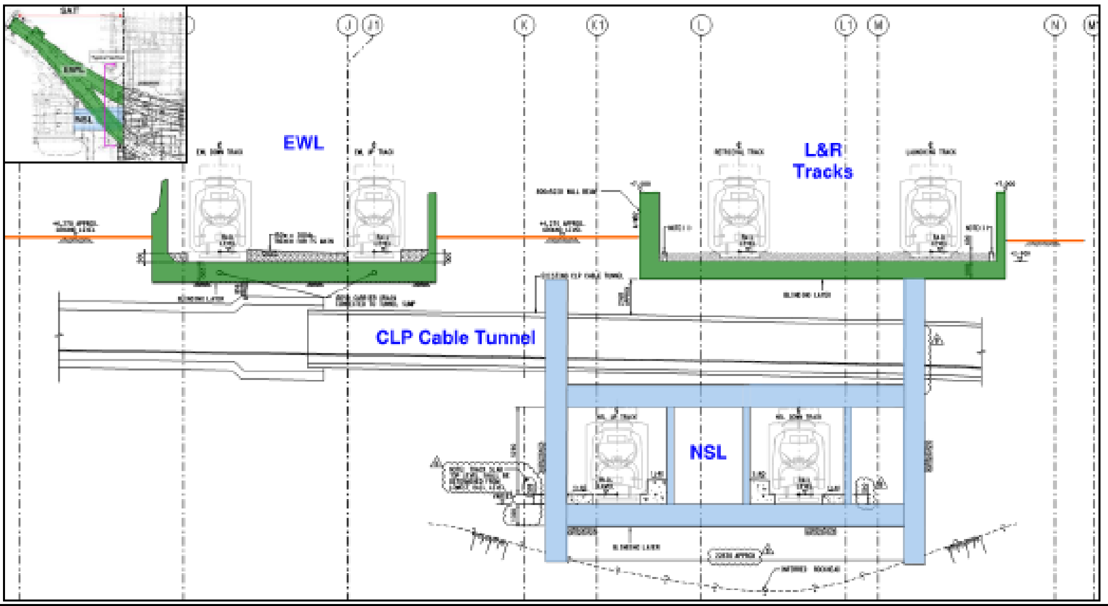
[**BB9970/§3.2.7**]

- As with the EWL and NSL slabs, the inference is that as-constructed shear link contribution has been totally disregarded in the evaluation of shear capacity.
- In addition to the remedial strengthening work proposed it is also recommended that long-term monitoring is carried out [**BB9980-9981/§§5.4** and **6.3**].

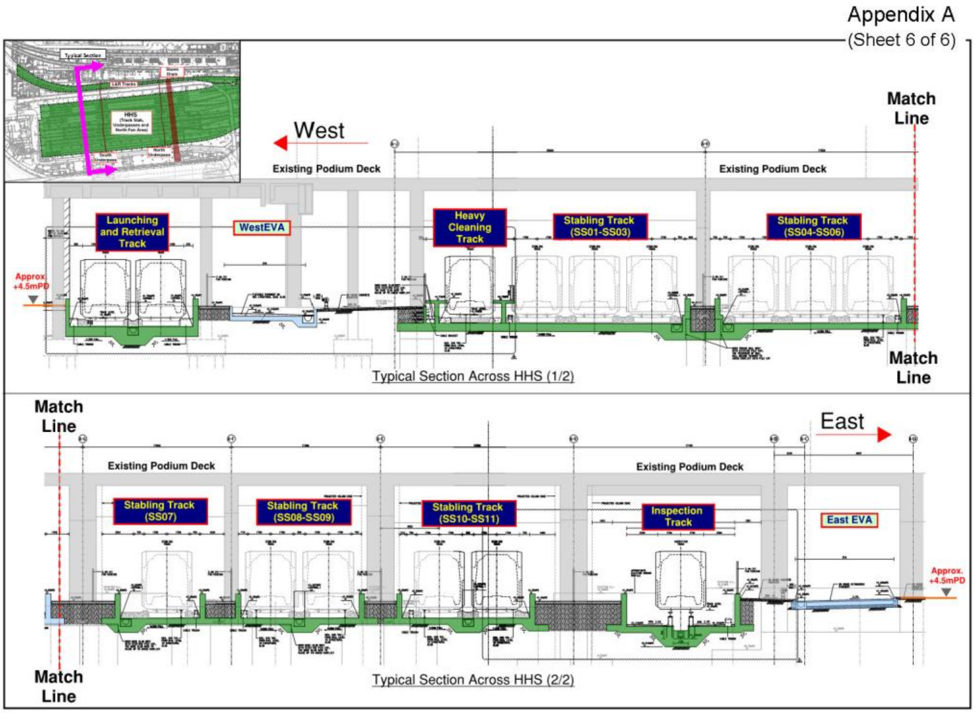
9. The Verification Report concluded at §6.4 [**BB9980-9981**] that the NAT, SAT and HHS works were structurally safe only "*…for the purpose of the ongoing construction activities*" thus inferring that the works would be unsafe if put into public use. This is contradictory, at least in part, in that the NAT is said not to require any remedial or strengthening work. What the Verification Report does, in essence, as does the Holistic Report, is to conflate the prime issues of "safety" and "contractual compliance" under the umbrella of "code compliance". As I have explained in my Supplemental Report adduced in the Original Inquiry ("**my COI 1 Supplemental Report**"), elements of a structure, or even an entire structure, can be "safe" even though not 100% "code compliant".

SAT AND HHS STRUCTURES

10. Cross-section drawings, again courtesy of the MTRCL [**BB16/9987-9988**], are copied below for ease of reference to show the structural arrangement at each of the two areas of the works which are alleged as being structurally deficient.



Typical SAT Section (showing NSL box tunnel area in blue)



Typical Section of Hung Hom Stabling Sidings (HHS)

Typical HHS Section

OPINION AND RESPONSE

11. I will now deal in turn with each of the two main issues raised by Mr Southward.

1 – COUPLER CONNECTIONS/COUPLER ENGAGEMENT

HyD acceptance criteria

12. By way of repetition, HyD's enhanced PAUT acceptance criteria, which by inference meet the permanent elongation test criterion as required by the HKCOP, are a) a maximum of 2 exposed threads; and b) a minimum of 37mm engagement [**OU1/584**].

13. For ease of reference, I have copied my Coupler Engagement Calculator below, which is contained in my COI 1 Supplemental Report. This relates to a "standard" 11 thread T40 bar (i.e. 10 threads plus one thread for positive tolerance) and shows the HyD acceptance criteria. There can be minor variations in the threaded lengths.

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	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

14. Only a 44mm engagement, with 10 "effective" threads, can provide a full "butt-to-butt" connection. Anything less always produces a gap between the two bars and,

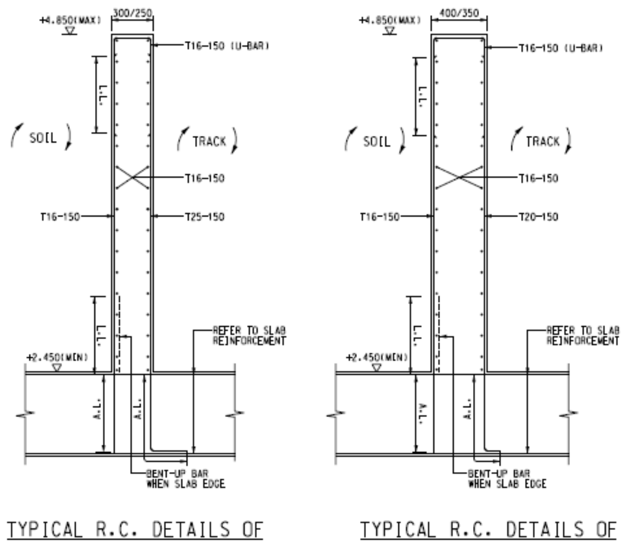
based on the knowledge derived from testing to-date, will always fail the permanent elongation test and therefore fail to be code-complaint.

Function of stabling sidings

15. As the term implies, stabling sidings are used to park trains not in use. The trains enter at very low speed and there is no public access to the platforms. The only safety-critical feature at HHS is that columns which support the main station podium structure are located in between trough walls as illustrated in the MTRCL "Typical HHS Section" drawing at paragraph 10 above. Provided the columns cannot be damaged by a train derailing and hitting a trough wall there is no structural safety issue. The columns could, for example, be surrounded by a compressible layer so that any lateral soil movement caused by impact to the trough wall has no effect on the column. In any case, it is my opinion that an impact due to train derailment is likely to cause serious damage to any trough wall such as to require localized reconstruction.

HHS trough wall kicker coupler assemblies

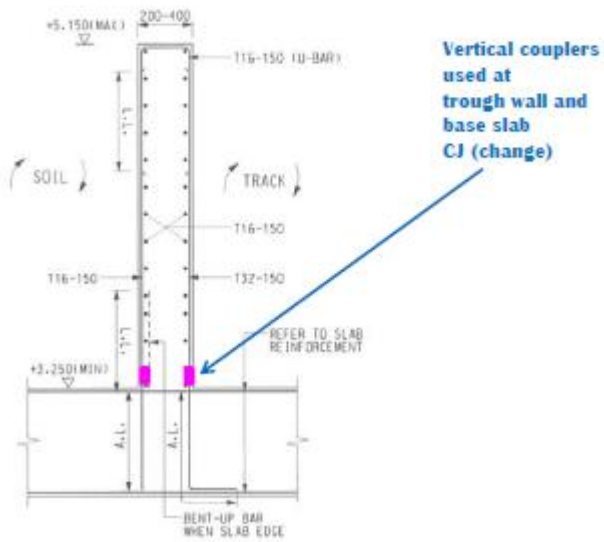
16. An extract from LCAL's Drawing No. 1112/B/HHS/ACM/C12/701 Rev C dated 27 May 2016 [**DD8/11310**] is copied below to show the typical HHS trough wall height, thicknesses and rebar details as intended in the original design.



TYPICAL R.C. DETAILS OF 300/250 THK. TROUGH WALL
SCALE 1 : 20

TYPICAL R.C. DETAILS OF 400/350 THK. TROUGH WALL
SCALE 1 : 20

17. Instead of using lapped bars at the kicker, i.e. the wall-top of slab interface, LCAL used mechanical couplers to provide a splice for the vertical rebar on each face of the wall which is acceptable in respect of the HKCOP. This is illustrated in the following diagram courtesy of AECOM [**BB17/10151**].



18. This obviously is an entirely different situation from what happened approx. 3m from ground level in respect of the bottom rebar of the main EWL slabs where heavy T40 starter bars had to be screwed into couplers which had been exposed in the D-walls. In this HHS situation a maximum diameter T25 bar is used, access to the couplers is unrestricted and gravity is working in favour of the main wall rebar being inserted and tightened into the couplers. Why would any shortcuts be taken when the connections were so easy to inspect?

19. The following are several examples, courtesy of ARUP[3][**BB18/11111 (Box 108), 11113 (Box 219) and 11103 (Box 284)**], taken during the construction of the HHS trough walls which demonstrate the more amenable working conditions, the lighter rebar and the open visibility to inspectors, including those of MTRCL.

[3] ARUP North Approach Tunnels, South Approach Tunnels and Hung Hom Stabling Sidings Volume 1 – Final Independent Report on Findings Rev B dated 8 July 2019 – Appendix F

Box 108 - Couplers at Trough Walls

Box 219 - Couplers at Trough Walls

Box 284 - Couplers at trough wall

20. The decision to impose a strength reduction factor of 35% [BB16/9976/§4.2.6 and §4.5.2] has not been justified because no invasive investigation at all has been

carried out to support the allegation of defective workmanship in the couplers. Instead, the same statistically-based failure rate, premised on partially-engaged coupler assemblies in the rest of the station which are alleged not to contribute structurally, has been applied to a much simpler form of construction. That the trough wall kickers are in any way deficient is therefore pure speculation.

21. Even if it was considered justifiable to "mark down" the bending capacity of the trough wall, it must be remembered that impact due to a train derailment is very much an ultimate limit state event and not one of serviceability. The failure rate, and hence strength reduction factor, which is documented in my COI 1 Supplemental Report, which derives from an analysis of partially engaged coupler assemblies, suggests that 6.9% is an appropriate figure for a 7.5 thread engagement. There would, in my opinion, be justification for using 7 threads and perhaps even 6 threads as the ultimate limit state acceptance criterion, as will be discussed later.

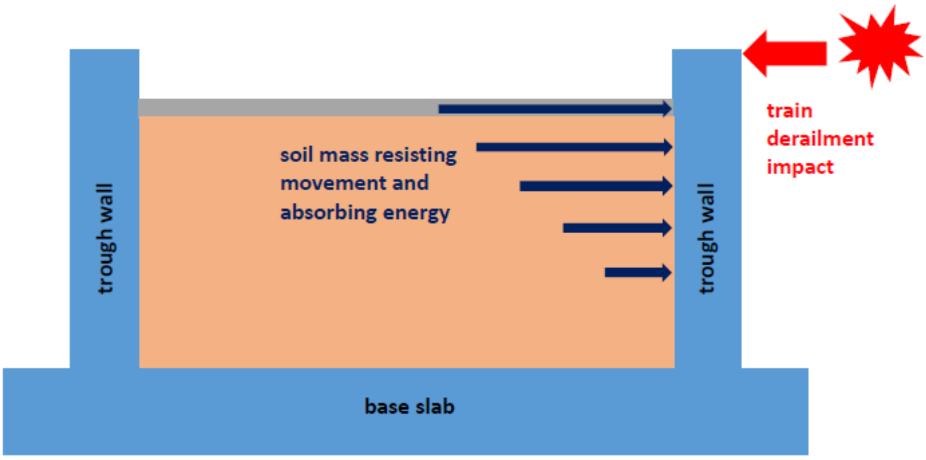
22. Having said that, it must be remembered that the PAUT testing was carried out on couplers which were more difficult to assemble i.e. T40 bars placed horizontally at depth. Coupler assembly would have been much easier on the HHS walls.

Tension steel strength reduction

23. The strength of rebar in tension, which impacts on both bending and shear capacity, has also been "marked down" by 4% and 13% depending on the bar size because not all rebar was tested once it arrived on site [**BB16/9977/§4.3.2**].

Energy dissipation into soil fill between the trough walls

24. In the event of a train impacting a trough wall the soil fill between the walls will absorb significant energy and restrict the deformation of the impacted wall section. The principle is illustrated in the diagram below but I have not attempted to quantify the contribution. To do so would require soil-structure interaction modelling and FEA analysis. Engineered fill is obviously better than randomly-placed fill from a uniform density perspective but the latter will still provide significant energy-dissipation in the event of a train impact load.



Typical HHS trough wall section

AECOM Draft Review Report[4] [BB17/10097-10943]

25. It is to be noted that this document, issued on 28 June 2019, pre-dates the Verification Report (18 July 2019) by approximately one month. AECOM's brief was to review the utilizations of the HHS structures for areas where mechanical couplers had been used.

26. Table 3 on page 5 [**BB17/10105**] summarizes the trough wall utilizations from train collision loads for three types, all adjacent top vertical movement joints, and gives a maximum value of 90%. This is corroborated under the Conclusions on page 7 [**BB17/10107**]. In other words if there was no alleged issue with the couplers there is reserve structural capacity.

AECOM BD Submission Document[5] [DD18/18482+]

27. AECOM's BD Submission was issued on 30 August 2019, over a month after the Verification Report was issued and some two months after their draft review report was issued.

[4] AECOM Draft Review Report for the Utilization of the Hung Hom Stabling Sidings Structures Rev 0 dated 28 June 2019
[5] AECOM BD (Buildings Department) Submission B3.13A1 – Structural A&A Works – Package 8 – Track Slabs + Underpass Corridor – Calculation Rev F Volume 1 of 2 dated 30 August 2019

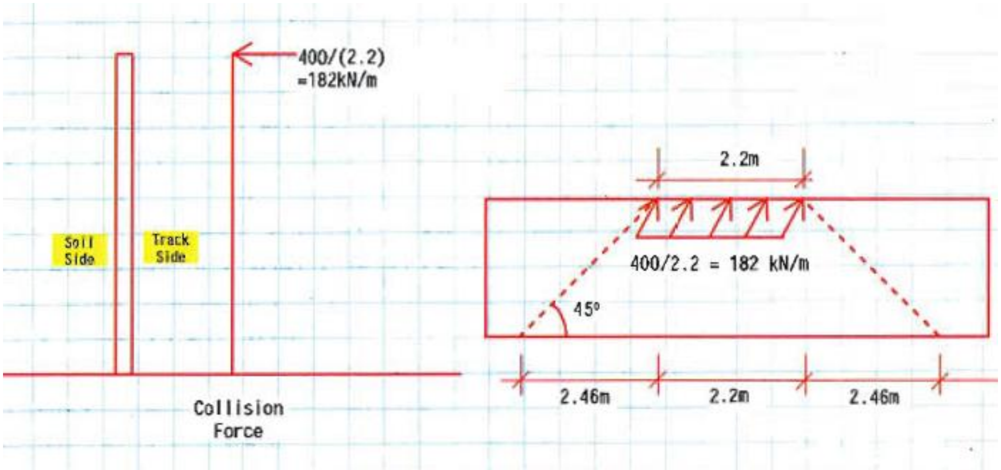
28. AECOM confirm at §1.2.2 on page 1 [**DD18/18493**] that trains in the sidings run at low speed and the chance of column collision is "*very low.*"

29. At §1.2.4 on page 1 [**DD18/18493**], they also confirm that the collision loading is only a ULS (Ultimate Limit State) consideration.

30. At §1.4.2 on page 2 [**DD18/18494**], AECOM state their design basis i.e. "*In order to enhance the confident level of the design…reduction of allowable stress to 299N/mm² (65% of fy 460N/mm² rebars) is adopted for the design of the Type I coupler splices…*" This corroborates the 35% strength reduction factor stated in the Verification Report (i.e. 100% less 35% equals 65%).

31. At 2.1.2 on page 5 [**DD18/18497**] AECOM state that the concrete grade used for the relevant trough walls is Grade 40D/20 i.e. a 28-day characteristic strength of 40N/mm².

32. The following diagram is copied courtesy of AECOM from Appendix A [**DD18/18512**]. It shows a very simplistic and conservative analysis approach whereby the impact load is assumed to spread down through the trough wall panel at 45^{o} as shown to the kicker level where the couplers are located. The true behaviour of the wall, in my opinion, is not reflected. If the impact load was to occur immediately adjacent to a vertical movement joint location, the load could only spread downwards in one direction. This gives the critical condition because a lesser length of wall at kicker level is considered as resisting the induced bending and shear force.



AECOM's assumed impact load distribution through HHS trough wall panel

33. The automated calculation output is not easy to follow but AECOM identify several "scenarios" where the "Safety Factor", i.e. another way of describing reserve capacity, is less than 1.35 (i.e. 35% reserve capacity). It follows that if the strength reduction factor is 35%, any situation where a wall panel has a Safety Factor less than 35% fails. AECOM therefore conclude that the trough walls panels in those situations are unsafe and that strengthening is required.

ARUP Report[6] [BB18/10944-11299]

34. At §8.3.2.1 on page 17 [**BB18/10963**] Arup, on the basis of a 45° load spread down the wall panel at a vertical movement joint, from the line of impact (similar to AECOM's approach as outlined above), state that the maximum utilization is 92%. ARUP, however, implemented a FEA using a 30° load spread and, at §8.3.2.2 on page 18 [**BB18/10964**], confirm that for the worst-case panel at a vertical movement joint, the maximum utilization reduces to 64% for bending.

35. At §8.3.2.3 on page 18 [**BB18/10964**], Arup discuss several mitigating factors which, if applied, would further reduce the trough wall utilizations. These include:

- Low train speeds in the stabling sidings with a low likelihood of derailment and collision.

[6] ARUP North Approach Tunnels, South Approach Tunnels and Hung Hom Stabling Sidings Volume 1 – Final Independent Report on Findings Rev B dated 8 July 2019

- Because the trains are empty the live load is, in effect, zero. Therefore the train loads specified by MTRCL are too conservative.
- The worst-case application of the load is at right angles to the trough walls whereas, in reality, that can never happen. A smaller angle of collision would significantly reduce the bending moments and shear forces on the trough walls.
- The train itself absorbs energy on impact.

36. ARUP conclude at §11.3 on page 30 [**BB18/10976**]:

    v) "*For the HHS the general utilization levels in the structural members under consideration do not exceed 63% in bending…except for some trough walls due to a collision load that Arup considers is conservative and could be reduced to a much lower value.*

    vi) *From the…independent analysis Arup considers…the HHS elements where couplers have been used are fit for purpose and do not require remedial measures or enhancement works.*"

<u>EIC Memorandum[7] [CC12/7296-7297]</u>

37. EIC Activities PTY Ltd ("**EIC**") reviewed the Verification Report and on page 2 [**CC12/7297**] conclude "*…for a train impact, the ULS acceptance criteria should be 25mm instead of 37mm.*" The acceptance criterion of 25mm is in effect a 6 thread engagement (i.e. 25mm plus 3mm tolerance = 28mm engagement).

38. Also on page 2 [**CC12/7297**], EIC state that 16.2% is the appropriate statistically-derived coupler failure rate and hence the appropriate strength reduction factor for the HHS walls.

<u>SYW Part 1 Study Report[8][BB19/12114-BB20/12527]</u>

[7] EIC Memorandum to LCAL entitled "Hung Hom Station – EIC Response to MTR Final Verification Study Report for NAT, SAT and HHS" dated 23 August 2019

[8] SYW & Associates Ltd Part 1 Study Report: Verification Proposal of As-constructed Conditions of the North Approach Tunnels, South Approach Tunnels and Hung Hom Stabling Sidings dated 24 May 2019

39. It is noted that the SYW Part 1 report pre-dates the Verification Report by almost two months. This report deals with Part 1a (Consolidation and verification of available construction records) and Part 1b (Review and ascertain the as-constructed conditions of the structures) as defined in the Verification Report.

40. At §3.3.5.1 (ii) on page 15 [**BB19/12129**], SYW also confirm "*…empty trains will be running at very low speed.*" Also in respect of the HHS stabling sidings they also state at the end of §3.3.5.1 "*…which are not open to the public…can be classified as insignificant structures for assessment purpose.*"

41. At §4.3 on page 17 [**BB19/12131**] SYW make recommendations for MTRCL's consideration. §4.3 (v) states "*Though no follow up action is considered necessary for couplers installed in…HHS structures, it is worthwhile considering a short to medium term of monitoring of these couplers installed structures after opening of the SCL for complete assurance purpose.*" §4.3 (vi) states another option "*As an alternative to (v) above, the structures in question may be re-assessed with a reduced coupler strength/capacity according to the investigation result of the couplers installed in Areas B and C to justify their structural adequacy.*"

42. This, in my opinion, is tantamount to SYW on the one hand saying "The HHS structures are safe so do nothing except monitor" and on the other hand saying "However you could play safe and apply strength reduction factors". The two options are not equally weighted and not directly comparable.

<u>SYW Part 2A Study Report[9][BB20/12528-12652]</u>

43. The SYW Part 2A report was issued one month before the Verification Report and, as stated at §1.2 on page 3 [**BB20/12531**], includes visual inspection of the structures and checking of the design calculations.

44. SYW's conclusions are set out at §3 on page 5 [**BB20/12533**]. In particular, §3.1(ii) states "*The visual inspections…have revealed no major anomalies or any significant structural defects having cause for concern of safety.*"

[9] SYW & Associates Ltd Part 2A Study Report: Verification Proposal of As-constructed Conditions of the North Approach Tunnels, South Approach Tunnels and Hung Hom Stabling Sidings dated 14 June 2019

45. Moreover, §3.1(iv) states "*The effects of the couplers installed in the HHS trough-walls…are considered to be very insignificant in view of their being relatively simple on-grade structures and the very limited retained height.*"

46. The summary at §3.2 [**BB20/12533**] states "*…it may be concluded that the…HHS structures are safe and free of any significant structural defects.*"

47. This confirms that the HHS trough walls were deemed structurally adequate and safe until MTRCL imposed the 35% arbitrary strength reduction factor without any reasonable justification.

Southward Expert Report[10] [ER(COI2)1/Item 10.1]

48. Section 4 of the Southward Report refers. At §4.4, he identifies the critical trough wall sections as those each side of vertical movement joints as illustrated on page 8.

49. At §4.5.1 on page 10, Mr Southward opines that a strength reduction factor of 6.5% to 6.9% is appropriate if based on the failure rate of coupler assemblies in the NSL slab.

50. He makes the valid points at §4.5.2 on page 10 that:

- there is no similarity between the NSL slab and the HHS areas
- the type of construction is different in HHS, namely:
    - smaller size and weight of the rebar
    - vertical wall
- no investigation has been carried out in the HHS trough walls to confirm a problem with the coupler assemblies

51. At §4.6 on page 11, Mr Southward correctly highlights the fact that the actual concrete strength should be included in the assessment instead of the lower specified design value.

[10] Tony Gee Structural Engineering Expert Report: Extended Inquiry dated 18 October 2019

52. Mr Southward has carried out an alternative analysis of the critical trough wall panels adjacent to the movement joints using "yield line" theory which is permitted by Section 5.1.2 of the HKCOP [**H8/2864**]. In essence this mimics the way in which the wall section would deform and fail in the event of a train impact. At §4.7.3 on page 12, he explains "*The yield line theory…better represents the behaviour of slabs in ultimate load conditions. The assumed collapse mechanisms are defined by a pattern of yield lines along which the reinforcement has yielded and the location of which depends upon the loading and boundary conditions."* He mentions also the US AASHTO LRFD design code as used for analogous bridge deck parapet walls.

53. His diagram on page 13 explains his analysis and mode of deformation and then carries out two analyses, each for three wall sections of different height and thickness:

   a) He uses the MTRCL 35% strength reduction factor plus the MTRCL rebar tensile strength reduction factor of 4% for rebar larger than 16mm diameter. He concludes at §4.7.5 on page 14 "*…there is a minimum spare capacity of 7% for the tallest wall of 2.46m (i.e. Wall 2), which is the most critical situation of the three walls.*"

   b) He then explains why that calculation is too conservative because a) the 35% strength reduction is only applicable at the base of the wall where the couplers are located at kicker level; b) the remaining vertical rebar is only subject, at worst, to a 4% "mark down"; and c) the top-of-wall inverted "U"-shaped rebar needs to be taken into account. He therefore carries out further yield line analysis, splitting the critical wall section into three horizontal zones: bottom, middle and top. He concludes at §4.7.6 on page 15 "*…there is a minimum spare capacity of 11% for the 300mm thick wall (i.e. Wall 1), which is the most critical situation of the three walls.*"

54. Notwithstanding those results, Mr Southward considers the critical value of strength reduction factor at which the walls could withstand a train impact and concludes "*…the MTRCL proposed strength reduction factor of 35% could actually*

be further reduced to 58% and the walls would still be able to withstand the collision loads."

55. He concludes at §4.7.8 on page 15 "*...the trough wall upstands can withstand the train collision loads, even when a 35% reduction factor is applied to the strength of the coupled connections. In fact...the coupled connections could be reduced in strength to 58% and the walls would still have sufficient load capacity.*"

56. In other words, Mr Southward has hypothetically accepted MTRCL's hugely unjustified and conservative approach and has irrefutably proved, in spite of the very significant strength reduction factor, that the trough walls are safe and have significant reserve capacity. This removes any possible argument on the efficacy of partially engaged couplers.

<u>CONCLUSIONS ON COUPLER CONNECTIONS/COUPLER ENGAGEMENT</u>

57. The AECOM assessment, which informed the Verification Report, has been hugely conservative on two counts:

- It has obviously been influenced by statistical analyses of coupler assemblies which are based on questionable premises, in particular because no opening-up has been carried out in the HHS area; and

- The retro-analysis of the trough wall panel scenarios was done using very basic and overly cautious structural theory which does not represent the actual behaviour in the event of a train collision.

58. However, the structural contribution of partially engaged coupler assemblies can be set aside. Even assuming that MTRCL's hugely unjustified and conservative assumptions are correct, it has been demonstrated by Mr Southward, using "yield line" theory and analysis, that the HHS trough walls are not only safe but have a significant reserve capacity in the event of a train impact.

59. Were my method at paragraph 24 above i.e. soil-structure interaction modelling and FEA analysis to be implemented to include the contributory mitigating effect of the soil fill, I have no doubt that the safety reserve margin would be even greater.

60. My unreserved opinion corroborates that of the other independent experts i.e. ARUP, EIC, SYW and Mr Southward. The HHS trough walls are very safe and there is no doubt whatever that structural integrity and safety have not been compromised.

2 – SHEAR LINK REINFORCEMENT AND PARTIAL UTILIZATION OF SHEAR

61. This is the second core issue for consideration as per my brief. By way of repetition, part of the base slab of the NSL box tunnel area of the SAT, i.e. between D-wall panels EH1 and EH4, is alleged to be deficient in shear capacity in spite of the fact that no opening-up has been carried out on the top surface of the NSL base slab to investigate the presence or details of shear links. Being a buried structure, the base slab experiences significant upwards loading because of the significant water head. It appears, however, that Atkins have identified a load case whereby the NSL slab experiences net downward loading causing the base slab to try to shear downwards near the D-wall supports or where there are internal walls.

62. For the sake of emphasis, I again highlight my opinion that generally because of the lighter type of construction and more congenial working conditions, there was a much greater likelihood, in respect of the shear link installation, of "getting it right". This NSL slab in this part of SAT, however, is a deeper base slab and it may be that MTRCL were treating it the same as Area A in respect of potential shear link deficiencies. If so, this was unjustified in my opinion because, by way of repetition, no investigation was done in Area A to provide evidence. Likewise no investigation was carried out in the SAT to prove the shear links are deficient.

Atkins Revised Structural Assessment[11] [BB19/11611-12112]

63. The date of this report is of interest i.e. 19 July 2019 bearing-in-mind that the Verification was issued one day earlier i.e.18 July 2019. I will elaborate on this in due course.

64. At §4.1.14 [BB19/11668], Atkins state "*The peak utilizations…for the NSL slab are based on assessing the behaviour of the slab as one way spanning. This is conservative as the transverse wall at the southern end of SAT can support the slab, i.e. two way spanning…*"Atkins then state, at §4.6.1 [BB19/11681], that "*The results of the assessment …(NAT and SAT Revised Structural Assessment)*

[11] Atkins Detailed Design for Hung Hom Station and Associates Tunnels NAT and SAT Revised Structural Assessment dated 19 July 2019

indicate that the utilizations of all critical structural members within the SAT tunnel are within the structural capacity."

65. At §4.6.4 [**BB19/11683**] they confirm "*Shear utilisations are also modest…in the SAT tunnel.*"

66. Then at §4.6.13 [**BB19/11685**] they disclose "…*we have been advised by MTRCL to adopt a strength reduction factor of 35% in assessing the capacity of the structure. This is comparable to the NSL slab in the adjacent HUH station."*

67. This strength reduction factor, premised on partially engaged couplers, is admitted by Atkins to impact on the shear capacity of the structure because at §4.6.17 [**BB 19/11685**] they state "*…the shear links capacity would be decreased due to the reduction of the tension reinforcement provided at the coupler positions in the structure. The maximum utilization for shear reinforcement is 75%...where the area of tension reinforcement near supports has been reduced by 35%.*"

68. At §4.6.22 [**BB19/11688**] Atkins then state "*The moment and shear capacities after considering the coupler reduction factor and after moment distribution, show utilizations for…the shear links less than 77% for SAT tunnel…"*

69. The reason for the net downward shear load on the NSL slab is explained at §4.7.3 [**BB19/11689**] where Atkins state "*The shear reinforcement at the NSL base slab is required in the form of flexural shear due to load transfer from the fixed end moment at support and also the shear force due to the concentrated load from the structure above the NSL including the embedded soil."* The latter is a reference to the dead load imposed on the SAT tunnel as a result of the backfill following the bottom-up cut-and-cover method of construction used in this area of the works which is explained at §4.1.6 [**BB19/11659**].

70. §4.7.6 [**BB19/11690**] is significant. Atkins there confirm "…*based on the revised NAT and SAT Structural Assessment and consideration of shear enhancement and axial load, <u>shear reinforcement is not now required</u>.*" (My underlining for emphasis).

71. It is also explained by Atkins at §4.8.3 [**BB19/11690**] that "*For the shear utilization a 13% reduction factor for shear links (diameter no greater than 12mm) was assessed. Coupler defects were not considered concurrently with the 13% strength reduction factor.*"

72. In summary, at §4.8.5 [**BB19/11692**], Atkins state "*With consideration on the 13% reduction in shear reinforcement, the maximum shear utilization for SAT tunnel was found to be 80%."*

73. Atkins' "Conclusions" are also enlightening in that, at §5.1.2 [**BB19/11693**], they confirm that "*For the NAT NSL, a new combined 3-D shell model is set up to reflect a more realistic structural behaviour of the tunnel structure.*"

74. At §5.1.6 [**BB19/11693**], Atkins confirm "*The most critical section for shear force occurs at the SAT NSL slab, which is of a maximum utilization of 77%, with moment redistribution applied and 35% of coupler defects considered.*

75. In respect of the strength reduction factors of 4% and 13% for rebars greater or equal to 16mm and less or equal to 12mm respectively, Atkins state at §5.1.10 [**BB19/11693**] that they carried out a sensitivity analysis and conclude that "…*the utilization for…SAT structures are well within design capacity.*"

76. Significantly, there is no mention of the shear links being totally disregarded as is stated in the Verification Report which was issued one day earlier.

77. This Revised Structural Assessment is therefore very useful in confirming that the SAT NSL base slab has adequate reserve shear capacity, and therefore was safe, before MTRCL took the decision (which Atkins at the time appeared not to have been aware of) to totally disregard the structural contribution of the shear links.

Atkins BD Consultation Document[12] [AA2/490-754]

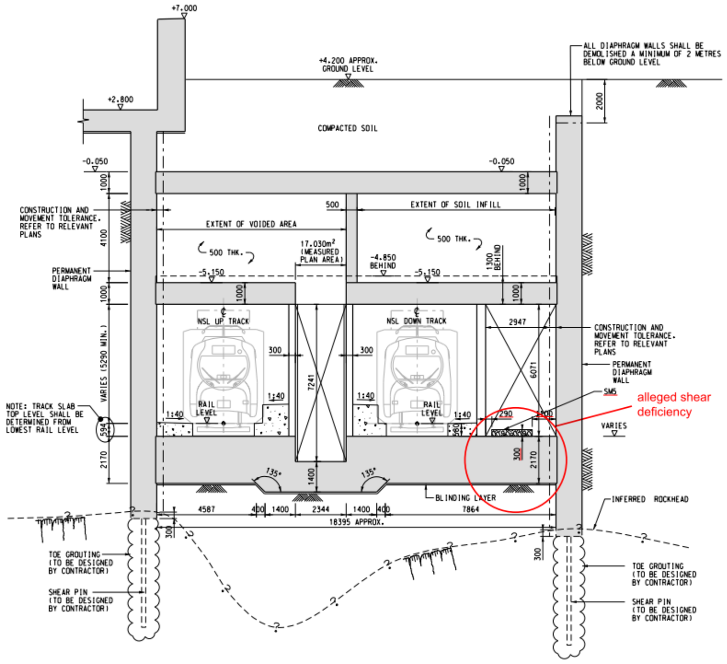
[12] Atkins BD (Buildings Department) Consultation Document HUH-1 – HUH Station Primary Structure and Excavation & Lateral Support Part 1 of 4: SAT, Area A and HK Coliseum (Rev AN) [Volume 1] dated 27 September 2019

78. Atkins produced this Consultation Document more than two months after issuing their Revised Structural Assessment. Despite the previous confirmation that there was reserve shear capacity, in their Executive Summary of this BD Consultation document, Atkins state their design assessment basis [**AA2/504-505**] i.e. "*To cater for the workmanship issue for the shear link, existing shear link is considered redundant in the structural assessment and indicated on the RC drawings for SAT (NSL slab, NSL mezzanine slab and slab @ -0.050m level), Area A and HKC.*" This is confirmed at §10.8 on page 56 [**AA2/563**].

79. They also confirm [**AA2/505**] that "*For SAT assessment, rebar strength reduction factors of 4% for rebars diameter greater than 12mm and 13% for rebars less than or equal to 12mm respectively are adopted.*" Of interest, this slightly contradicts the Verification Report which uses the 4% reduction for rebar greater or equal than 16mm diameter but the error favours the assessment if it was applied as stated by Atkins. This is confirmed at §10.8.1 on page 56 [**AA2/563**].

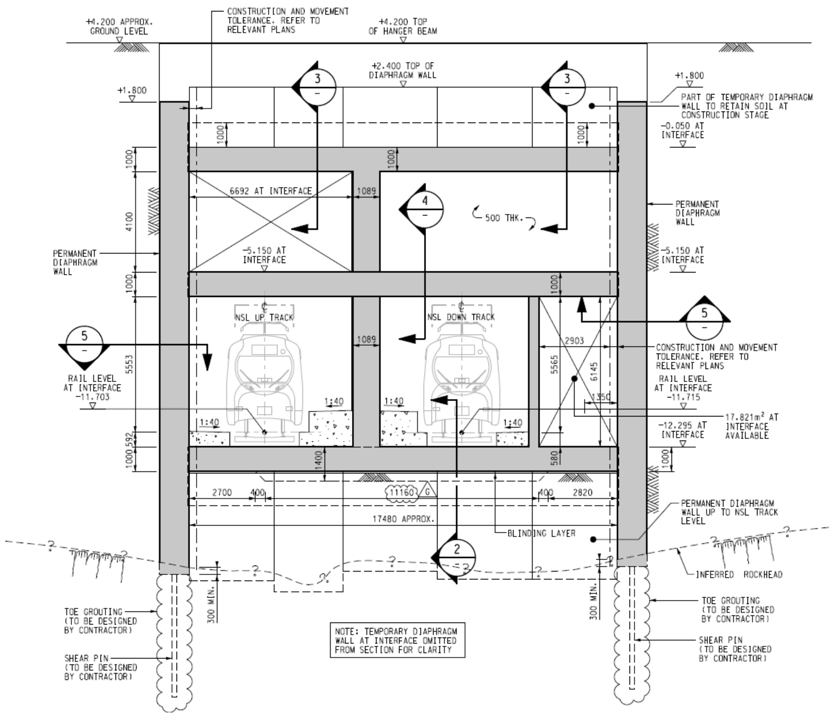
80. At §7.1.1 on page 28 [**AA2/535**], Atkins confirm the use of Grade 40D/20 concrete i.e. 40 MPa strength which is the specification for the original design. In other words, Atkins have not carried out their assessment using the actual, as-tested, value nor have they accounted for strength gain with time.

81. At §11.1 on page 59 [**AA2/566**], Atkins allege that there is a shear failure zone in the 2,170mm thick base slab adjacent to the D-wall which requires localized strengthening. There is no supporting analysis but, by inference, they re-ran the analysis models. The SAT cross section for the D-wall panels between EH1 and EH4 is copied below, courtesy of Atkins, for ease of reference [**AA2/580**].



<u>SAT cross section between EH1 and EH4 with 2,170mm thick base slab</u>

82. By comparison the next typical SAT cross section is copied, courtesy of Atkins, to show the 1m thick base slab which generally exists in that area [AA2/580].



<u>Typical SAT section with 1,000mm thick base slab</u>

ARUP Report[13] [BB18/10944-11299]

83. ARUP conclude at §11.3 on pages 29 and 30 [BB18/10975-10976]:

iv) "*For the SAT…the general utilization levels in the structural members under consideration do not exceed...65% in shear…*"

vi) *From the…independent analysis Arup considers…the SAT…elements…are fit for purpose and do not require remedial measures or enhancement works.*"

EIC Memorandum on Response to MTR Proposed Suitable Measures (Shear – SAT Area)[14] [CC12/7845-7864]

84. Firstly, I do not consider my mention and review of this EIC report to be in breach of the Commission's Directions dated 12 October 2019 for the following reason. The Verification Report shows that strengthening is considered necessary because of an alleged shear deficiency in the SAT [**BB16/9978/§4.5.4**]. EIC rightly need to ask "Why is the SAT deficient in shear capacity?" They then need to satisfy themselves, or otherwise, that the shear capacity is less than that required.

85. EIC again highlight the fact that no investigation has been carried out in SAT to identify shear links and their configuration and prove any defects [**CC12/7850/bullet point 2**]. The most similar location in terms of slab thickness and detailing is Area A where MTRCL have carried out no invasive investigation and where the LCAL opening-up confirmed the shear links to be present and adequately anchored.

86. EIC make reference to their report[15] [**CC12/7375-7479**] and rely again on the work of Professor Stephen J. Foster which confirms that partial utilization of shear links

[13] ARUP North Approach Tunnels, South Approach Tunnels and Hung Hom Stabling Sidings Volume 1 – Final Independent Report on Findings Rev B dated 8 July 2019
[14] EIC Memorandum to LCAL entitled "Hung Hom Station – EIC Response to MTR Proposed Suitable Measures Shear – SAT Area dated 11 October 2019
[15] EIC Report entitled "Hung Hom Station – Response to MTRC Recommended Suitable Measures – Shear" dated 30 August 2019

meets the minimum of the HKCOP and contributes to the overall shear capacity [CC12/7454-7478].

87. Having reviewed Atkin's analyses they point out that only a simplified 2-dimensional frame analysis method was used. They state on page 2 [CC12/7847] "*…this idealized analysis fails to account for the following*

- *The Northern Diaphragm wall* (i.e. a unique feature at the end of the SAT north boundary) *will attract a significant portion of load due to 2 way action. The section 1, 2d frame analysis would assume all load transfers to the east and west diaphragm walls…this would result in a significant over estimation of shear and bending moments in this sections [sic] analysis.*

- *The thick 2170mm slab will attract a larger load than the adjacent 1000 and 1300 thick slabs.*"

88. EIC conclude [CC12/7850]:

- "*An assessment incorporating 2 way action in the SAT slab is necessary…The 2d frame analysis…is simplistic and over estimates shear values at the suitable measure locations.*

- *Due to thickness of slab and proximity of the dividing transfer walls to the east and west diaphragm walls it is likely that arch and/or strut and tie analysis methodologies would show no requirement for shear ties.*"

89. It appears, however, that EIC had not seen the Atkins Revised Structural Assessment which, as previously noted, was issued several months earlier and confirmed that a 3-D analysis had been implemented.

<u>EIC Memorandum on Review of Suitable Measures Proposed for Southern Approach Tunnel (SAT)[16] [CC12/7866-7878]</u>

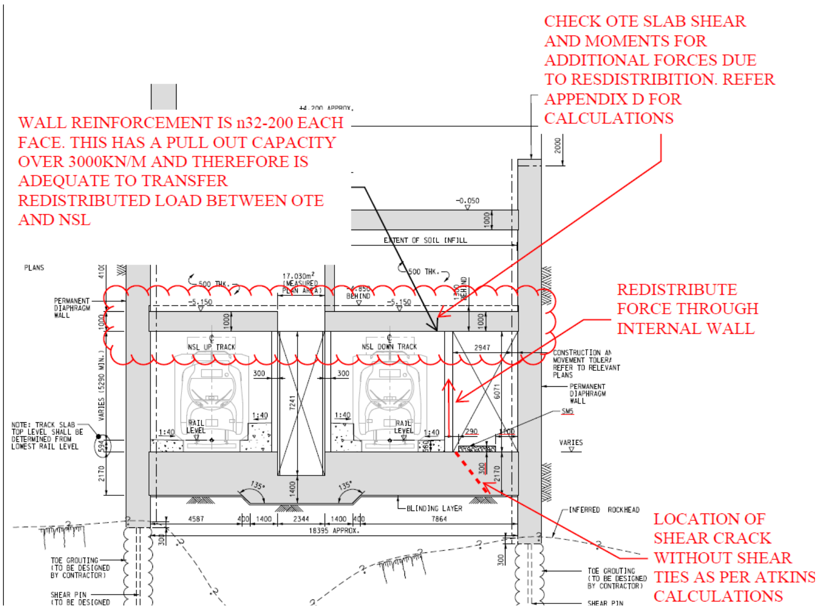
[16] EIC Memorandum to LCAL entitled "Hung Hom Station – EIC Review of Suitable Measures Proposed for Southern Approach Tunnel (SAT) dated 24 October 2019

90. Likewise, for reasons given above, I do not consider my mention and review of this EIC report to be in breach of the Commission's Directions dated 12 October 2019.

91. EIC recap on their hierarchical approach to assess the safety of the structures throughout the works. They confirm [CC12/7870] "*We have reviewed Atkins' calculation data where suitable measures are proposed at the SAT. We have recalculated the shear utilization taking into account the elements of the 'Hierarchy of Shear Assessment Methodology'...In summary Atkins' calculation of the shear capacity did not take into account:*

- *Correct tension steel area ...*
- *Consideration of the redistribution from the vertical supporting walls. At the ULS state, excess shear force in the base slab will be redistributed through the wall and upper slab, both of which have surplus capacity; and*
- *Representative 28-day strength of the concrete which was supplied to site ... , which for this area is 55MPa ...* "

92. The second bullet is best explained by copying EIC's appended diagram [CC12/7873]



93. EIC conclude [CC12/7871]:

- *…the analysis method used by Atkins is too simplistic and…a 3D analysis would provide more accurate and reduced shear loads at the areas where suitable measures are proposed* (this statement was not obviously informed by the earlier Atkins' Revised Structural Assessment).
- *We have…recalculated the shear utilisation…demonstrates that no shear links are required when considering the redistribution to the upper slab.*
- *In any event, if partial utilisation of the shear is used…the shear links meet the minimum shear requirements in the Hong Kong Code of Practice for the Structural Use of Concrete 2004 version and are capable of taking the loading. **We therefore consider that no suitable measures are required at SAT**.*"

94. In other words, EIC confirm the SAT structure to be safe as-is.

SYW Part 2 Study Report [BB20/12528-12652]

95. In respect of the SAT NSL slabs, Appendix D-2b to the SYW Part 1 Study Report [**BB20/12357-12374**] contains record photographs taken during construction. I have copied typical photographs below of a) the NSL mezzanine slab rebar [**BB20/12369**]; and b) the NSL roof slab rebar [**BB20/12370**]. These clearly show compliant shear links.

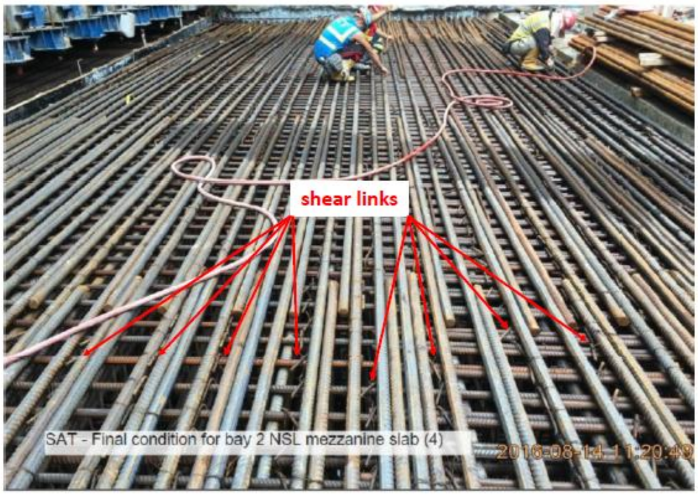


Photo Records Reference: SAT NSL B38-2

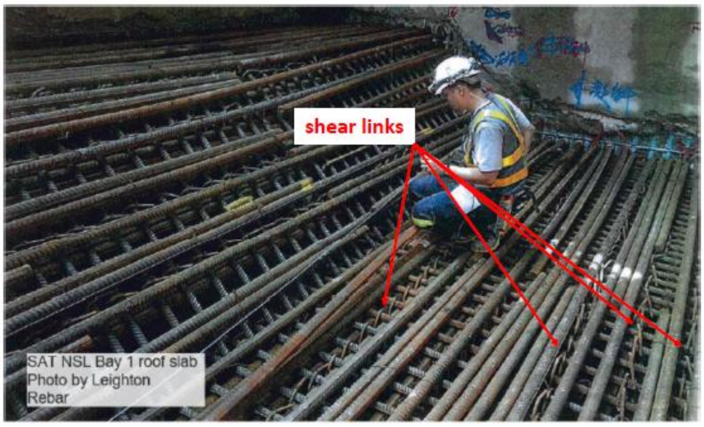


Photo Records Reference: SAT NSL B39-2

96. Although there are record photographs of the NSL base slab at various stages of rebar assembly, I have not, however, found any showing the completed rebar.

97. It is assumed that the MTRCL inspectors were satisfied with the installation of the shear links in the NSL roof and mezzanine slabs and it can reasonably be assumed that they also found the installation of the shear links in the NSL base slab was in order. In my opinion, the MTRCL decision to completely disregard the shear links and their contribution to the structural shear capacity of the NSL base slab was unjustified in light of the above evidence and astonishingly conservative.

98. SYW's summary at §3.2 on page 5 [**BB20/12533**] of the Part 2A Study Report, written before MTRCL's decision to ignore the structural contribution of the shear links, states "*…it may be concluded that the…SAT structures are safe and free of any significant structural defects.*"

Southward Expert Report [ER1 (COI 2)/Item 10.1]

99. At §5.2 on page 18, Mr Southward comments:

- "*In Area A of the station structure, which is similar in slab thickness to that of the SAT structure, the only investigation done was by LCAL which categorically proves the presence of the shear links.*"
- *The concept of completely disregarding the as constructed shear links is analogous to the binomial approach adopted by MTRCL and Professor Yin*

in assessing the coupler connections…this approach would ascribe no loading capacity to coupler connections where the threads were still partially engaged. Similarly…a shear link with partial placement contributes to the strength of the structure. It is therefore wrong to completely disregard the shear links."

100. At §5.3.2 on page 18, Mr Southward explains in more detail the point highlighted by EIC at their second bullet point and in paragraph 91 above. Atkins' shear force diagrams, which inform their assessment, are appended to EIC's memorandum of 11 October 2019[17] [**CC12/7855-7864**]. The shear failure Atkins' predict in the NSL slab of SAT is caused by a net downward load.

101. In my opinion this failure mechanism cannot, however, occur. As described in the Atkins Revised Structural Assessment Report[18] at §4.1.9 [**BB19/11660**], the NSL base slab, although designed as a suspended slab with a hypothetical "*air gap*" between its soffit and the soil, was constructed using the excavated consolidated soil as formwork. In the ULS if the base slab tried to fail in shear, it would be prevented from so doing by the soil below it.

102. Notwithstanding, let us hypothetically accept the possibility of the base slab failing downwards in shear as predicted by Atkins. Mr Southward explains how this mode of failure cannot happen because the reinforced concrete wall which rises from the top of the NSL slab to the underside of the OTE slab acts in tension, to resist the downward movement of the NSL slab [**§5.3.2**]. In so doing, load is transferred into the OTE slab etc.

103. Mr Southward confirms that EIC have checked the capacities of the wall and the OTE structure are adequate to take the amount of load in excess of the load which would cause shear failure in the NSL slab. He concludes on page 21 "*…a shear failure could not occur in the NSL slab as predicted by Atkins.*"

[17] EIC Memorandum to LCAL entitled "Hung Hom Station – EIC Response to MTR Proposed Suitable Measures Shear – SAT Area" dated 11 October 2019
[18] Atkins Detailed Design for Hung Hom Station and Associates Tunnels NAT and SAT Revised Structural Assessment dated 19 July 2019

104. If the theoretical shear failure was caused by a loading case which resulted in a net upward load on the base slab, then the same principle is applicable i.e. the wall acts instead in compression, which is even easier to justify, transferring load into the OTE structure.

105. In respect of MTRCL's decision to "mark down" the steel tensile strength because some rebar was not tested when it arrived on site [**BB16/9965/§3.1.10**] [**BB16/9977/§4.3.2**], Mr Southward makes the point at §5.4.2 on page 21 by way of a reminder "*…the rebar testing conducted…satisfies the relevant quality assurance standards and the level of confidence in the rebar that was not tested after it was delivered to site exceeds that which is required under the relevant standards.*"

106. On this issue Mr Southward concludes at §5.4.3 on page 22 "*From a structural engineering perspective, I do not believe that it is appropriate to adopt any strength reduction factor because of the small percentage of rebar that was not re-tested after it was delivered on site. All of the evidence indicates that the rebar used in the Project was of satisfactory quality and that the re-testing conducted after the rebar was delivered to the site was sufficient to satisfy the relevant standards.*"

107. Notwithstanding this objection, all the assessments appear to have incorporated the imposed "mark down" values.

108. At §5.5 on page 22, Mr Southward restates the fact that both Atkins and EIC identified that only limited areas of the works in general required any shear reinforcement and in those limited areas, only minimum (nominal) shear link steel was required. He also explains that there is an over-provision of shear link steel, compared with the minimum code requirement. On this basis, he states "*Therefore, a reduction in effective anchorage length of the shear link is possible. In other words, the over-provision of the shear reinforcement is such that the as constructed links can still carry the ultimate loads and therefore the safety of the structure is ensured.*"

109. He then explains at §5.5 on page 22 "*The HKCOP allows modification of anchorage lengths dependent upon the design load in the bars…because of the over-provision*

of the shear links when compared to the minimum steel requirements, the straight length of the shear link can be reduced to 70mm without compromising the strength of the structure." In other words, because the shear links are not working to 100% capacity the anchorage length can be modified proportionately.

110. In summary, Mr Southward concluded at §5.7 on page 23 "*Only the shear strength of the SAT slabs has been questioned…based on the structural assessments carried out, the entire structure is safe and fit for purpose."*

<u>CONCLUSIONS ON SHEAR LINK REINFORCEMENT AND PARTIAL UTILIZATION OF SHEAR (SAT)</u>

111. The Atkins shear capacity assessment which informed the Verification Report on the SAT has also been hugely conservative because:

- No investigation has been carried out to confirm any problems with the as constructed shear link reinforcement in SAT.
- The contribution of the as constructed shear links has been completely discounted even though there is an acknowledgement that the type of construction was less prone to buildability issues than the rest of the station.
- No account therefore has been taken of the structural contribution of partially anchored shear links.
- Because some rebar was not tested on site before use, strength reduction factors have been applied to all tension reinforcement.
- This reduces both the bending and shear capacity of the SAT.
- The specified design concrete strength was used instead of the higher tested as-constructed values.
- The modelling and analysis method used was too simplistic and does not properly represent the structural behaviour of the SAT.

112. My unreserved opinion, confirmed by the reports of the independent experts, ARUP, EIC, SYW and Mr Southward is that the shear capacity of the SAT slabs is more than adequate and there is no doubt whatever that structural integrity and safety have not been compromised.

SUMMARY OPINION ON STRUCTURAL INTEGRITY AND SAFETY

113. My summary opinion as expressed in paragraph 125 of my First Report dated 6 January 2019, 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.*"

114. Since then the NAT, SAT and HHS areas have also come under scrutiny. MTRCL's verification assessment has been speculative in that it has not been informed by any opening-up in these areas.

115. Also, because a small percentage of the total rebar was not tested when delivered to site, MTRCL have applied a strength reduction factor to the tensile strength of the rebar.

116. This reduces both the bending and shear capacity of the structures.

117. It is assumed, without justification, that there are partially engaged coupler assemblies in the HHS trough walls and, as a consequence a strength reduction factor has been applied which is of the same magnitude as was applied to the assessment of the main station despite the fact that the HHS trough walls reflect a much easier form of construction than the main station box tunnel structures.

118. As a result, the HHS trough wall panels, adjacent to vertical movement joints, are alleged by MTRCL to be structurally inadequate and are therefore, by inference, unsafe.

119. It is also assumed, without justification, that the shear reinforcement in the NAT and SAT areas does not comply with the HKCOP and consequently the structural contribution of the as constructed shear links has been totally disregarded.

120. As a result, the NSL base slab of the SAT is alleged by MTRCL to be structurally inadequate and are therefore, by inference, unsafe.

121. I have independently and extensively reviewed the assessments of MTRCL's consultant, Atkins and conclude that, in addition to the above speculative strength reduction factors which presumably they have been instructed to use, Atkins are unjustifiably very conservative in respect of the methods of analysis and the material properties used.

122. I have likewise independently and extensively reviewed the assessments of the other independent engineers i.e. ARUP, EIC, SYW and Mr Southward who have taken into account the structural contribution of a) partially engaged coupler assemblies; and b) the as-constructed shear link reinforcement, and also used actual concrete strengths and more relevant analytical methods.

123. I am therefore fully satisfied, and in no doubt, that the HHS and SAT structures, in addition to the non-disputed NAT area, are safe "as-is" and there is no reason why these parts also of the station cannot and should not be opened for public use.

124. 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.

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

126. Monitoring of the HHS area is not relevant because the trough walls are in essence containment structures in the event of a train derailment collision.

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