MTR Corporation Ltd

Shatin to Central Link
Hung Hom Station
Holistic Study to Verify
As-constructed Condition

Expert Report
Dr. Mike Glover

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ARUP

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Mike Glover

My name is Mike Glover. I am a Fellow of the Institution of Structural Engineers and a Fellow of the Royal Academy of Engineering.

I was the Technical Director and Deputy Project Director for the £7bn high speed Channel Tunnel Rail Link (CTRL/HS1), a position I held throughout its design, procurement, construction, commissioning and ultimately its handover in 2007, to cost and programme. Subsequently, I was the Technical Director for the client for the £1.35bn Queensferry Crossing across the Forth in Scotland from project inception through Parliamentary Authorisation, design, procurement and construction and ultimately its successful opening in 2017, substantially under budget.

I was made an Arup Fellow in 2006, to ‘recognise the highest design and technical achievements of an Arup person’.

I was awarded the 2007 Sir Frank Whittle Medal by the Royal Academy of Engineering, the first civil engineer to receive the Award, and the Gold Medal of the Institution of Structural Engineers in 2008 'for ... outstanding contributions to the design and construction of major multi-disciplinary projects ...’.

In 2009, I was awarded an OBE in the New Year's honours list for Services to Engineering.

I have been appointed by MTRCL to present expert evidence to the Commission on Structural Engineering matters

My CV is appended to this Report.

1. Scope of Instructions

1.1. The scope of my instructions is to provide independent expert structural engineering advice in connection with the holistic review of the structural integrity of the as-constructed details of critical elements of the SCL Hung Hom Station (HUH).

2. Studies Undertaken within the Scope of Instructions

2.1. Arup has produced a number of Reports reviewing documents from MTRCL, their Designer, Atkins, and the Contractor's (LCAL) as-constructed drawings, and assessing the structural integrity of the Hung Hom Station Box structure on a fitness for purpose basis. I have led the study work that these Reports summarise.

2.2. The Reports have considered the structures in, amongst others, Areas B and C. A central aspect of these reports has been a review of the design principles and the percentage strength utilisations of the structural elements, both as advised by the MTRCL Designer Atkins and by way of independent assessments made by Arup.

2.3. In addition, a Report has been issued focused on the predicted behaviour of the Diaphragm Wall compared with inclinometer records taken during construction. Inclinometers are devices for measuring changes in wall deflection over the height of the wall, and are very widely used internationally.

2.4. The relevant Reports are listed below and I will refer to these Reports as necessary in this Expert Report:

a) Shatin to Central Link Hung Hom Station Holistic Study to Verify As-constructed Condition - Assessment Report – REP/0002 (Rev A) dated 9 November 2018[1]; and

b) Shatin to Central Link Hung Hom Station Holistic Study to Verify As-constructed Condition Assessment Report - Design Spot Checks for Diaphragm Walls – Plaxis Analysis (Rev B) dated 27 November 2018[2].

3. The Structure of this Report

3.1. The principal objective of this Report is to present to the Commission my opinion on the overall structural integrity and safety of the station box structure and its fitness for purpose for its intended use.

[1] **[B19/B25114-25156]**
[2] **[B20/B26004-26048]**

3.2. Additionally, I have commented on the proposal to carry out load testing and the scope of the opening-up works that are currently in progress.

3.3. Accordingly, I have structured my evidence under a number of headings dealing with: Ductility and Mechanical Couplers; Rebar Detailing and Engineering Judgement; Percentage Strength Utilisation; Coupler Strength Characteristics; Structural Adequacy; Load Test and Opening-Up; and, finally some Miscellaneous Matters.

4. Ductility and Mechanical Couplers

4.1. Ductility in a structural element is described in Section 2.4 of the Hong Kong Manual for Design and Detailing of Reinforced Concrete to the Code of Practice for Structural Use of Concrete 2013 (see Appendix B), as "the ability of a structure to undergo "plastic deformation", which is often significantly larger than the "elastic" deformation prior to failure".

4.2. Figure 2.4 in the same Manual gives both a graphic description and definition of ductility. In simple terms, ductility is a measure of how much energy a structural element can absorb by behaving plastically before breaking. Because the behaviour is plastic, the deformation is non-recoverable and hence permanent. For this reason, designs use ductile behaviour only to resist very extreme events.

4.3. Successive versions of the Hong Kong Code of Practice for Structural Use of Concrete[3] have included increased requirements for Ductility in Beams, Columns and Walls. The requirements are generally applicable in areas referred to as "Critical Zones" located immediately adjacent to connections/joints between elements. The intention is that under extreme loading, and particularly strong seismic ground motion, these zones would develop an energy absorbing ductile "plastic hinge", which also allows the concentration of forces to be redistributed safely to other elements of the construction.

4.4. The Commission has considered the distinction between Types I and II mechanical couplers. An important point is that both are capable of achieving the full ductility of a connection under normal loading

[3] **[C13/C8348-8554; H8/H2818-3015]**

conditions: see BOSA's Product Catalogue[4] which confirms that both its Type I and Type II couplers are designed with some ductility. So, in that respect, they are both "ductile" couplers.

4.5. A Type II coupler has been designed for more extreme loading conditions where the connection is subjected to stress reversal (i.e. tension to compression) through a number of cycles of such stress reversals, as would be the case in very strong ground motions caused by large earthquakes. However, the Hung Hom station box would not be subjected to such very strong ground motions under the low to moderate earthquake seismicity classification which it is predicted that Hong Kong might be subjected to[5].

4.6. The reasons for this are:

- Information Note 08/2015 Seismicity in Hong Kong[6] published by the Geotechnical Engineering Office (GEO) states that the seismicity of Hong Kong is low to moderate.
- Underground box structures have performed exceedingly well in very strong earthquakes which is reflected in the way these structures are designed internationally. In the USA Standard ASCE 7-16 such structures would be considered as Non-Building Structures and for Seismic Design Categories equivalent to Hong Kong they would be designed to be non-yielding.
- Hong Kong reference documents also reflect the low seismic risk associated with such structures. Information Note 08/2015 Seismicity in Hong Kong[7] states in its Key Messages "c) The possibility of significant earthquake damage to man-made slopes, retaining walls and reclamations in Hong Kong is low."
- Due to the disproportionately stiffer and stronger EWL slab (3000mm deep) relative to the Diaphragm Walls (1200mm thick), it would be impossible to develop ductile behaviour in the slab or its connection to the walls since the wall would have failed structurally under ultimate load conditions long before the rebar in the slab would have reached its yield stress, i.e. the slab

[4] **[H9/H4070]**
[5] **[A1/A698]**
[6] **[A1/A695-699]**
[7] **[A1/A695]**

connection would remain in the elastic range. This is clearly demonstrated by my illustrative calculation as set out in the calculation sheet appended to this report as Appendix C.

4.7. For the reasons cited above, the specification of Type II couplers is an unnecessary requirement for this structure.

5. Rebar Detailing and Engineering Judgement

5.1. There are a number of features of the rebar detailing which are surprisingly excessive in terms of both quantity and concentration, but perhaps the most prominent of all these is the quantity of rebar and number of coupler connections in the soffit of the EWL slab adjacent to the Diaphragm Walls.

5.2. It is not clear why the provision of rebar and associated coupler connections in the soffit of the EWL slab at this location should be practically the same as that provided in the top, because the slab is not subject to loading which would create tension in the slab soffit at this location; this is principally because of the great weight of the 3m thick slab.

5.3. Part of the explanation could be a desire to comply with the Hong Kong Code of Practice for Structural Use of Concrete 2004 (Second Edition) Cl 9.9.1.1(a)[8] (see Appendix D) that at any section of a beam within a critical zone the compression rebar should not be less than one-half of the tension rebar at the same section. However, and as explained above in Section 4 Ductility and Mechanical Couplers, the concept of a critical zone existing in this slab is incorrect.

5.4. A further explanation could be compliance with Cl 9.3.1.3 of the Hong Kong Code of Practice for Structural Use of Concrete 2004 (Second Edition)[9]: Reinforcement at supports which requires half of the calculated span rebar to be taken through to the support. But again, in the circumstances of this structure that seems to be rather excessive.

[8] **[H8/H2969]**
[9] **[H8/H2964-2965]**

5.5. In my opinion, the sheer scale of the slab in terms of its thickness and extent, and the low level of seismicity should have merited an engineering review and assessment of the actual requirements, rather than the apparent unquestioning application and acceptance of the codified requirements. Such a review would have demonstrated that at this connection the quantity of rebar required is nominal only.

5.6. Guidance that has been set down in the context of slabs of more normal proportions, say 300mm thick, is not necessarily appropriate for a 3000mm thick slab.

5.7. The Foreword to the Hong Kong Code of Practice for Structural Use of Concrete 2004 (Second Edition)[10] recognises this and clearly states that **"***This Code of Practice provides guidelines for professionals and practitioners on design, analysis and construction of concrete structures"* and that *"Although this Code of Practice is not a statutory document, the compliance with the requirements of this Code of Practice is deemed to satisfy the relevant provisions of the Buildings Ordinance and related regulations"*. Importantly, the guidance given by the Code is self-evidently not mandatory, and the design parameters set out in the Code are sufficient, but not imperative, conditions to achieve a safe and robust structure. As such, in the unusual situation of the Hung Hom Station Box which is structurally different in many material respects to the conventional types of buildings contemplated by and provided for in the Code, I would not have expected the guidelines therein to have been applied without question.

5.8. Therefore, for whatever reason, the quantity of rebar provided in the soffit of the EWL slab is substantially over-provided. A similar situation exists with the rebar provision in the top of the NSL slab, where the dominant loading is not gravity loading, but hydrostatic uplift, i.e. the slab is deflected upwards. A large reduction in the quantity of rebar installed and the number and size of couplers would have resulted had an engineering review of the kind referred to above been implemented.

5.9. The conclusion I have gained from my review of this project is that this is just one example, and unfortunately there are others, of a failure to

[10] **[H8/H2821]**

use engineering judgement and, instead, to rely on rigorous compliance with the wording of the Code 'requirements', which more properly should be regarded as guidance only. In fact, on closer scrutiny, that guidance can be and should have been determined as inappropriate for the particular issues posed by a structure like the Hung Hom station box.

6. Percentage Strength Utilisation

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

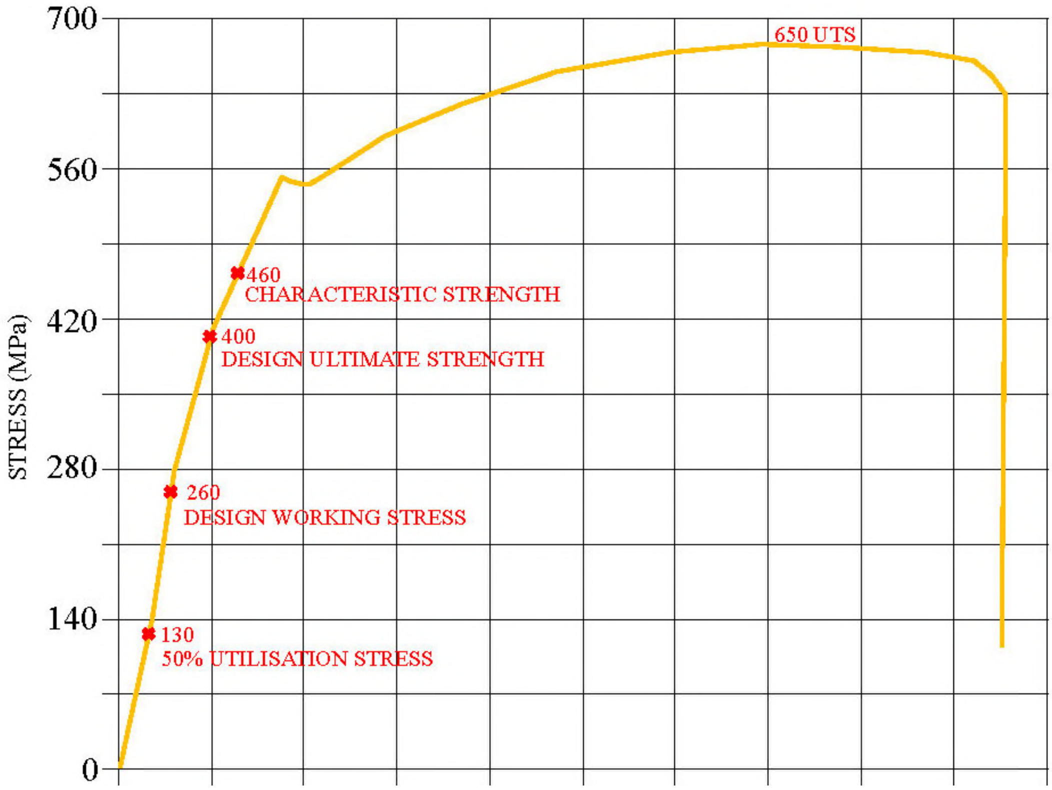

Figure 1

6.2. In simple terms, the structure is designed to two Limit States. The Ultimate Limit State (ULS) ensures safety and considers the fundamental issues of equilibrium and strength. The Serviceability Limit State (SLS) ensures functionality, principally by controlling cracking, displacement and vibration within accepted limiting values.

6.3. The rebar stress regime at ULS is defined as the Characteristic Strength, approximating to the elastic limit of the rebar, reduced by a safety factor to arrive at a Design Ultimate Strength. The respective values for each are 460MPa and 400MPa as specified by the Hong Kong Code of Practice for Structural Use of Concrete 2004 (Second Edition)[11].

6.4. It will be noted from Figure 1 that the Design Ultimate Strength is substantially less than the 650MPa Ultimate Tensile Strength (UTS), the maximum tensile stress that a material can withstand before breaking. The difference between the UTS and the Design Ultimate Strength represents a large margin of reserve strength and robustness.

6.5. The rebar stress at SLS will depend on the relative proportion of self-weight to live loading, but generally is assumed to be about 260MPa under full loading, when the structural element has been designed to the full design ultimate stress of 400MPa.

6.6. Most elements in a structure are not operating at 100% of their capacity under their full operational loadings. This can be a result of prudent design, standardisation or the fact that the critical loading conditions had now passed, for example because they occurred during construction, and were not to be realised in the future. The measure of this over-provision is commonly referred to as the percentage strength utilisation of an element; the SLS stress will be proportionately lower.

6.7. The percentage strength utilisation of an element can be described by the simple equation below:

$$= \frac{(Applied\ Force\ from\ the\ Design\ Ultimate\ Limit\ State\ Analysis)}{(Design\ Ultimate\ Strength\ of\ As - Constructed\ Element)}$$

6.8. The MTRCL's Designer, Atkins, has advised percentage strength utilisations for each element of the completed structure; for example, these are generally around 50% for the EWL and NSL slab to Diaphragm Wall connection, and are summarized in the Arup Reports listed in Section 2. Arup has carried out independent assessments of critical elements of the EWL and NSL slabs and confirmed these general levels of utilisation.

[11] **[H8/H2851-2852]**

6.9. For a percentage strength utilization of 50%, the stress level in the rebar will be about 130MPa when subjected to the full design loading, and generally even lower in most of the structure.

6.10. For this structure, these low levels of utilisation arise in great part from the phased nature of the construction. During construction, the EWL slab was free spanning between the Diaphragm Walls and subjected to severe construction loads; the slab was designed for these extreme conditions.

6.11. Subsequently, these loads have reduced with associated reduction in the stresses in the EWL structure. In addition, extra supports have been constructed in the form of columns and walls from the NSL, which reduce the spans of the structure and the effects of subsequent operational loadings. It should be noted that the trackway lies virtually over, and loads directly onto, the Diaphragm Wall; the loading on the EWL slab is therefore less than would be expected.

6.12. The result of the above reductions in loading and span is that the utilisation levels have reduced to the low levels now reported by Atkins and confirmed by Arup.

6.13. These levels of utilisation confirm the structure has a comfortable level of robustness and redundancy.

7. Coupler Strength Characteristics

7.1. The opening-up works are at an early stage, but some early readings show the thread engagement of some coupler connections are less than the installation guidance provided by the coupler supplier, BOSA.

7.2. To provide context to the strength implications of lower levels of engagement, tension load tests on couplers with various degrees of engagement have been carried out by MTRCL. These tests demonstrate that the full ultimate tensile stress (UTS) of the rebar, 650MPa, is achieved with only 60% engagement, a fact which I understand has been confirmed by BD and the coupler supplier, BOSA: see the Report on tensile tests of couplers with different numbers of threads on connecting rebars carried out during site inspection by the Site

Monitoring Team of BD[12] and the BOSA Seisplice System Calculation Sheet for Couplers[13]. It should be noted that even at percentage engagements lower than 60% the strength achieved is still substantially above the Hong Kong Code of Practice for Structural Use of Concrete 2004 (Second Edition) defined design ultimate stress[14].

7.3. These results are not unusual, and are in line with results expected with threaded connections of many types and reflect the additional factors of safety built into construction products to accommodate the uncertainties and inevitable variations in workmanship that can and do occur in construction.

8. Structural Adequacy

8.1. There has been much discussion in the CoI hearing about workmanship and the alleged failure of various processes in the construction of Hung Hom Station. This has been necessary to allay the concerns that have been inflamed by the various statements and events of the past months as referred to in the media. However, that debate should not be confused with the inherent structural adequacy and, hence, safety of this construction.

8.2. The allegations of cutting of threaded bars had to be investigated to allay concerns about the extent of such malpractice, but that should not obscure the fact that such malpractice would have to have been on such an unimaginable industrial scale and, in addition, focused in specific areas, to have any effect whatsoever on the structural integrity of this construction, particularly in terms of making it unsafe – which it is not.

Inherent Reserve of Strength

8.3. A point that has been made in these discussions is that no construction is ever perfect, despite the diligence of the vast majority of the workforce. This has long been recognised by the construction industry and, hence, its designs and its products are subject to rigorous testing

[12] **[H25/H44521-44256]**
[13] **[H25/H44527.1]**
[14] **[H8/H2851-2852]**

and embody comfortable safety factors, as demonstrated by the coupler tests outlined above.

8.4. In addition, recognising that most elements of this structure have percentage strength utilisations generally less than 50%, the actual margins of safety are very large. The rebar in slabs at full working load is generally less than 25% of the rebars' design ULS stress in areas of tension, and in many areas of the alleged cutting of bars, such as in the soffit of the EWL slab, the slab is in compression with the load being taken by the concrete. It follows that the demands on the coupler connections are very much less than expected.

Diaphragm Wall to EWL Slab Connection

Further evidence of the large margin of safety in the construction lies in the Arup review of the readings of the inclinometers formed in the Diaphragm Walls, and summarized in the Arup Assessment Report - Design Spot Checks for Diaphragm Walls – Plaxis Analysis [15], referenced in Section 2 above.

8.5. The inclinometer records consistently show that the walls deflected substantially less than predicted and as a result the Diaphragm Walls and, in particular, their connection to the EWL slab, have been much more lightly loaded than the original design had estimated. Consequently, the rebar percentage utilization in the EWL slab to wall connection would be further reduced to be much less than 50%.

8.6. Also, as outlined in Section 5 Rebar Detailing above, the majority of the rebar and couplers at the Diaphragm Wall connection for the soffit of the EWL slab and the top of the NSL slab represent a large over-provision and are largely redundant in terms of their contribution to structural integrity.

Contractor's Alternative Detail

8.7. A further point that also needs to be fully understood is that the Contractor's Alternative Detail for the EWL slab to the east Diaphragm Wall connection is a superior detail to the accepted connection detail described by the consultation drawings, both in terms of performance and constructability. In structural terms, it is reasonable to view this as

[15] **[B20/B26004-26048]**

a change of a design detail and not a detailed design change; the force actions have not been changed, but the detail has been substantially improved.

8.8. The Joint Experts' Meeting held on 18 December 2018 considered concerns that have been raised related to the Contractor's Alternative Detail requiring the cutting down of the Diaphragm Wall, and latterly about the adequacy of the construction joint so formed. The Joint Experts' Memorandum summarises the experts' view of the acceptability of this work.

8.9. The background to the adoption of this view is:

- Cutting down of Diaphragm Walls is a normal part of box construction, both to reduce the level of the as-cast wall and the formation of the essential shear key. This 'key' is an important feature of underground box structures and extends the slab construction into the diaphragm wall beyond the inner layer of wall rebar, thereby ensuring the structural engagement of the slab and wall and avoids the workmanship risks of forming construction joints between two dissimilar elements of construction in a difficult underground environment.
- In this instance, because of the geometry of the EWL slab and the OTE slab beyond forming effectively a continuous slab locking in the top of the wall into a 'rebate' in the slab soffit, the quality of the construction joint has a minimal effect on the performance of the slab to wall connection.

Conclusion of Considerations of Structural Adequacy

8.10. Taking account of all the above evidence, namely:

- Substantial over-provision of rebar in some areas compared to the future demands of the structure;
- Low percentage strength utilisations generally throughout the structure based on the original design assumptions;
- Test evidence to demonstrate the large strength reserve in couplers with less than optimum thread engagement[16]; and

[16] **[H25/H44521-44527.1]**

- Evidence that stresses in the Diaphragm Wall and its connections to the EWL slab are much lower than predicted, based on site readings of the Diaphragm Wall movements during construction[17],

It is evident so far as I am concerned that the structure of the station box has large degrees of redundancy and robustness and, consequently, a comfortable margin of safety which supports my opinion that the structure is safe for its intended lifespan.

9. Load Test

9.1. The massive size and thickness of the EWL slab makes a load test of the structure to its full design load totally impractical.

9.2. A lesser test of the operational load has been suggested, but because of the enormous stiffness of the structure no meaningful conclusions could be drawn from such a test; the increases in deflections and stresses would be very small.

9.3. A more worthwhile approach would be to complete the re-analysis of the structure on the basis of the rebar detailing uncovered in the opening-up works to confirm its structural adequacy.

10. Opening-Up

10.1. The opening-up works of the structure, which are now in progress have two principal objectives/purposes, namely:

i. To verify the correctness of the LCAL's as-constructed details, principally to the top of the east side EWL Slab to Diaphragm Wall connection detail;

ii. To determine the extent of illegal cutting of rebar threads at coupler connections, if any, in the construction process.

10.2. The scope of the opening-up work was discussed in the Joint Experts' Meeting and the agreed conclusion summarized in the Joint Experts'

[17] **[B20/B26004-26048]**

Memorandum. I wish to emphasise my support for the views expressed in the Memorandum, and summarise below the reasons for that support.

10.3. The views expressed were that the locations of the opening-up for Purpose (i) are very comprehensive, but are often located in constrained locations in limited headroom ducts where the ability to create deep openings is verging on the impractical. I believe the opening-up should be limited to uncovering the first layer of rebar only to reduce the demolition of the construction at these structurally important connections, particularly since the detail repeats with depth. There is one exception requiring exposure of the top two layers where a single layer of through bars was laid on top of coupler connections in panel EH69.

10.4. For Purpose (ii), the important objective is to establish a sufficient sample of coupler connections to establish statistically the potential extent of alleged illegal cutting of bar threads. The purpose is to establish the extent of malicious wrong-doing, if any.

10.5. On the basis of the evidence supporting the structural adequacy and safety of the construction, there is little case for opening-up the structure beyond obtaining sufficient samples to statistically gain confidence that such widespread/wholesale illegal cutting has not taken place.

10.6. I suggest that the uncovering should be limited to the first layer of rebar exposed in the soffit, particularly since the bottom layer of rebar in the EWL slab soffit would have been the most difficult to install, and hence most likely to have been subject to malpractice. Demolition and cutting out of rebar to get to deeper layers would achieve no additional benefit, requires substantial reinstatement and presents an increased Health and Safety risk to operatives working in very constrained positions and often in ducts with limited access and low headroom.

10.7. In my opinion, currently there is no case for opening-up the NSL slab or the Diaphragm Wall since there is no evidence to suggest that these structures were not built in accordance with the accepted design, there have been no allegations of illegally cut threaded bars in either structure and the structural utilisations are low. Any opening-up of these structures would require considerable demolition of the installed rail works and the structures and extend the delay to the project further for no obvious benefit.

10.8. Additionally, in my opinion, sufficient samples of coupler connections will be gathered from the current opening-up of the EWL slab to establish a statistical basis for dispelling the allegations of the widespread malpractice of cutting threaded bars. I do not see a difference between the NSL and EWL coupler connections with respect to the materials, workforce, supervision or methodology, and hence samples from either will represent the characteristics of both.

11. Miscellaneous Matters

11.1. For completeness, I note the reported defects of honeycombing in the concrete soffit of the EWL slab and installation of shear links.

11.2. These matters were noted at the Joint Experts' Meeting held on 18 December 2018 as recorded in the Joint Experts' Memorandum. The agreed opinion was that the concrete defects were not unusual in such a massive construction and were capable of repair.

11.3. To elaborate on the honeycombing issue: it is accepted that the lapped bars in the zone of honeycombing have become de-stressed. However, it was also recognised that the structure has a relatively low strength utilisation level, that the most critical loading and span situation had passed, and that the load has been redistributed to adjacent areas. I understand that a formal justification has been made by the Designers, Atkins, to demonstrate structural integrity.

11.4. The misaligned shear links were not considered to reduce the structural action of the links, because of the multiple layers of rebar in the soffit of the slab.

11.5. There have been reports of some misplacing of shear links either in terms of spacing or rebar diameter. In reviewing the design, I was initially surprised by the large quantity of shear links provided in this massive 3000mm slab; shear links are not usual in slabs of this depth. Subsequently, I have formed the view that in many cases the links were provided as an extra precaution because of concerns about the uncertainties associated with construction loadings, particularly large concentrated loads, and the numerous and large openings through the slab, which concentrate load in the narrow spaces left between them – these were prudent design decisions.

11.6. However, in its post-construction loading condition and shorter spanning arrangement, the stresses in the structure have now generally reduced. As a result, I would expect the required extent of shear links to be much reduced to the extent that shear reinforcement is not generally required, except in localised areas. I understand that a formal justification has been made by the Designers, Atkins, to that effect. Independent spot checks by Arup have also confirmed this to be the case.

11.7. The structure of the Hung Hom station box shows no signs of distress, cracking or distortion to indicate that it has been overstressed during the critical construction stage. I also note that for the EWL slab the construction stage represented the most critical loading condition related to the combination of high and variable construction loadings, long free-spans between Diaphragm Walls and the large, numerous openings formed therein. The future operation loads and the extra supports provided by the NSL loadbearing columns and walls represent a more benign loading environment, which provides yet further confidence in the safety of the existing construction.

12. Declaration

12.1. I declare that the contents of this Report are correct to the best of my knowledge, information and belief.

M Glover

Mike Glover

7 January 2019