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Specialist Field[1]: Bridge structures; vehicle-bridge interaction; concrete structures; structural fire engineering; and reverse problems

Instructions

I have been instructed to provide my opinion in respect of the following issues:-

1. Is the present Holistic Proposal [B20/26098-B20/26136] appropriate for ascertaining the as-built condition of the diaphragm wall and platform slab works at Hung Hom Station Extension ("the Works")?

2. MTR's explanation that so long as only 24 mm (as opposed to 40 mm) of the threaded rebar has been screwed in, the structural integrity will not be compromised, due to BD's requirement has taken into account "additional buffer" (as alleged by MTR). In this regard, Paulino Lim's evidence [H25/44824-H25/44856] regarding BOSA's requirement may be referred to.

3. Given paragraph 6 of the Joint Expert Memorandum [B20/26424] (in respect of which I have provided some additional comments) [G20/15046-G20/15048]), if opening up lower layers of the EWL slab is considered not practical, what should be done to ensure that sufficient tests (with equivalent statistical value) are conducted?

4. Would screwing out the threaded rebars for testing cause any material structural damage?

5. Were the Works executed in accordance with the requirement of Contract 1112?

6. If not, do the Works (as constructed) give rise to safety concerns?

7. Are remedial or strengthening works required for the Works? If so, what?

8. If no conclusive view can be formed at this stage or under the present scheme of investigation, what are the further steps to be taken?

[1] A copy of my CV is attached as Appendix A.

Declaration

I declare that:-

(a) I have read the code of conduct set out in Appendix D of the Rules of the High Court (Cap. 4A) and agree to be bound by it;

(b) I understand my duty to the Commission; and

(c) I have complied with and will continue to comply with that duty.

Francis Au

Professor Francis T.K. Au

Date: 7 January 2019

Opinion

1. Appropriateness of Holistic Proposal for Ascertaining the As-Built Condition

1.1 Rationale of Holistic Proposal

1.1.1 MTRCL submitted a Holistic Proposal to the Government to address various concerns raised after allegations of malpractice and sub-standard quality of Works at Hung Hom Station Extension for the Shatin to Central Link were widely reported in the media.

1.1.2 It is indeed quite rare to open up a reinforced concrete structure to verify the as-built details. However, in view of various problems with the site records, timely update of design drawings, etc. and in the absence of any better method, opening up parts of the structure using a random sampling approach appears to be a necessary and pragmatic follow-up action for confirmation of site record and restoration of public confidence.

1.2 Sampling Method as stated in Holistic Proposal

1.2.1 The comments in this section can be regarded as those of an engineer without professional training in statistics.

1.2.2 The sampling method using binomial statistics proposed in the Holistic Proposal of MTRCL is based on certain simplifying assumptions, e.g. classifying the outcome of each investigation simply as success (i.e. compliance) or failure (i.e. non-compliance). In view of the complexity of various issues involved, this methodology is considered reasonable.

1.2.3 When non-compliant cases are discovered during the investigation, it is necessary to further assess the effects on the strength and other properties of the coupler assembly by applying appropriate reduction factors or making similar adjustments. These reduction factors can be determined on the basis of laboratory tests. For example, if the measured engaged length of a non-compliant coupler is 28 mm instead of the specified 40 mm, reduction factors for strength, stiffness, etc. can be determined experimentally. From site measurements of the random samples, the mean value and standard deviation for each parameter can be worked out.

1.2.4 After completion of the investigation, further calculations should be carried out to assess the structural adequacy and safety of the Works. It should be noted that defects may not occur uniformly in the Works. Each part of the structure to be checked should also be based on statistical approach in respect of possible clustering of defects, e.g. by assuming a suitable defect rate equal to a certain number of standard deviations above the average value.

1.3 Implementation of Holistic Proposal

1.3.1 The opening up strategy has been devised for two purposes, namely (i) to verify the as-built conditions due to lack of proper site records; and (ii) to assess the workmanship of coupler installations and reinforcement fixing. Available results of verification to date (Highways Department 2019) [*see Bundle OU1*] indicate that the concerns are not unfounded. It is noted from Section 6.2 of the Holistic

Proposal (second paragraph on page 22 of 27) that "Depending on the initial investigation results and if defective coupler connections are found, a greater sample size may be considered". It is therefore prudent to review the results after completion of the opening up operations according to the Holistic Proposal to see if and how the sample size should be increased. This review may also be carried out earlier for better planning.

1.3.2 According to the Holistic Proposal, after the reinforcing bar and coupler assembly are exposed, phased array ultrasonic testing will be carried out on the coupler to ascertain the engaged threaded length of reinforcing bar. Phased array ultrasonic testing is an advanced method of ultrasonic testing that has applications in industrial non-destructive testing, e.g. inspection of welds. The phased array probe comprises many small ultrasonic transducers, each of which can be pulsed independently as prescribed. When groups of phased array elements are pulsed, the ultrasound waves combine constructively and destructively to give configurable wave fronts that provide versatile inspections. If one end of a reinforcing bar coupler is exposed, it allows the use of the phased array ultrasonic testing to evaluate the engaged length inside. This is essentially the application of a well-established technique to a new problem. However, the ribs on the reinforcing bar near the coupler should be ground off (and hence resulting in slight loss of cross-sectional area of the bar and its corresponding bond strength with surrounding concrete upon future reinstatement), thereby allowing good contact between the phased array probe and the reinforcing bar to be established for emission of beams of ultrasound in an oblique manner towards the end of reinforcing bar. Proper calibration should also be carried out for accuracy and repeatability of measurements. Provided that calibration and validation are done properly, this technique for determining the engaged threaded length of reinforcing bar is reasonably reliable.

2. Acceptance Criteria and Performance of Reinforcing Bar Couplers

2.1 Reinforcing bar couplers are proprietary products designed and manufactured to comply with the relevant design code or an alternative standard accepted by the Building Authority. Apart from satisfying certain strength requirements, the coupled bar assembly should also comply with certain requirements in respect of deformation characteristics. It is often expected that the structural performance of a concrete member with coupled bar assemblies is not inferior to that with the equivalent continuous bars in all aspects.

2.2 According to the BOSA document "Visual Inspection – Acceptable Thread Tolerance" [C10/7013], "After connection has been fully tightened, one should see a maximum of <u>TWO FULL THREADS</u> to ensure a proper installation." The fact that "two full threads" have been both capitalised and underlined in the document emphasises its importance to ensure proper installation. This should be considered as the acceptance criterion for installation.

2.3 In Appendix 1 [H25/44833, English translation at H25/44527.1] of Paulino Lim's witness statement regarding BOSA's requirement, the equivalent strengths of coupled bar assemblies of the BOSA Seisplice System for 40 mm reinforcing bars having different engaged lengths are calculated and presented in Chinese. Based on BOSA's calculations, a splicing assembly having 6 threads engaged (or 4 mm × 6 = 24 mm engaged length as opposed to 40 mm of full engagement) will be sufficient to develop the axial strength of reinforcement. It is however noted that strength is just one of the aspects of structural performance.

2.4 Test results of equivalent tensile strength of coupled bar assemblies for 40 mm reinforcing bars having different engaged lengths [H25/44521-H25/44525] as summarized in Table 2.4.1 are generally above those shown in Appendix 1 [H25/44833, English translation at H25/44527.1] of Paulino Lim's witness statement.

Table 2.4.1. Test Results of Coupled Bar Assemblies [H25/44521-H25/44525]

Percentage engagement	Tensile load (kN)	Tensile strength (MPa)	Mode of Failure
30%	526.11	419	Slipping out of coupler
50%	791.54	630	Fracture at connector
60%	886.22	705	Fracture at bar
70%	870.96	693	Fracture at bar
100%	833.46	663	Fracture at bar

2.5 However, the unusual trend observed (e.g. the maximum value occurring at 60% engagement) suggests that variations of results can be quite large. In addition, there is no information on the load-displacement relationship of the test specimens. To assess the performance of non-compliant reinforcing bar-coupler assemblies having different engaged lengths, it is necessary to carry out the standard acceptance tests required by the Building Authority for each percentage engagement to gather further information including, but not limited to, the variations of strength, load-displacement relationship, etc., which, as I understand, have not been carried out by BOSA. In addition, the load-displacement relationship of a bar-coupler assembly with inadequate engaged length provides useful information on how the ductility of a concrete structure containing it is affected. In case many non-compliant couplers are found upon opening up which is based on random sampling, such information will be helpful to determine if and how extensive subsequent strengthening works or other remedial works should be done.

3. Opening Up Strategy for Assessment of EWL Slab and NSL Slab

3.1 Assessment of EWL Slab

3.1.1 Whilst the provision of flexural strength for hogging moment at the EWL slab adjacent to the connection between the EWL slab and the east diaphragm wall (the slab-wall joint) does not necessarily require bottom reinforcement, provision of bottom reinforcement is a mandatory requirement under the *Code of Practice for Structural Use of Concrete 2004, Second Edition* (Buildings Department 2004) (the Concrete Code) [H8/2818-H8/3015] and it still helps to ensure ductility, serviceability, etc. Therefore, the proper connection of the bottom reinforcement of the EWL slab to the diaphragm wall by way of mechanical couplers was required and would also serve useful purposes.

3.1.2 The need for opening up should be considered in the light of the incomplete and/or inconsistent site records in order to restore public confidence. The allegations of malpractice and poor workmanship in installation of couplers also call for some measures for assessment. The Holistic Proposal submitted by MTRCL has outlined an opening up regime with random samples to achieve a certain confidence level. If the outcomes are satisfactory, further opening up beyond the quantity proposed in the Holistic Proposal may not be necessary.

3.1.3 The sequence of opening up of concrete for testing of the EWL slab may be reviewed to allow the opening up of concrete for testing couplers for the top reinforcement to proceed first. It is noted from the site visit on 17 December 2018 [*photos in Bundle A1 Item 50*] that the working conditions inside the OTE duct for opening up concrete at the soffit of the EWL slab are quite poor and may cause safety concerns. However, such safety concerns are mainly related to those chosen sample locations inside the OTE duct only. Opening up at locations outside the OTE duct should be comparatively manageable. This is another reason why the opening up process would need to be reviewed and prioritised. It is understood that MTRCL has agreed to ensure the safety of the workplace for the opening up process.

3.1.4 To ensure the trustworthiness of the outcomes from random sampling for opening up, reinforcing bars in the third or fourth layer have been chosen. Should the circumstances render the exercise infeasible, the original chosen samples may be replaced by other samples in the nearest layer to ensure that the sample size is sufficient and meaningful.

3.2 Assessment of NSL Slab

3.2.1 In view of the impracticality of access to the couplers for the bottom reinforcement of the NSL slab, opening up of these couplers has not been included in MTRCL's Holistic Proposal. However, to enable a proper assessment or verification of the quality of workmanship of the coupler installations in the NSL slab, the Holistic Proposal, which had taken into account views of the relevant Government Departments and Government's experts, has included opening up of concrete to expose the random samples of couplers for the top reinforcement of the NSL slab. In this aspect, the inclusion of samples from the top reinforcement of the NSL slab is considered essential for serving the purpose.

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

4. Unscrewing of Threaded Reinforcing Bars for Testing

4.1 It is noted that, before the application of phased array ultrasonic test to determine the engaged lengths of the threaded reinforcing bars inside couplers, calibrations of the test method for different engaged lengths have been carried out both in laboratory and on another construction site. A tolerance of around 2 mm to 3 mm has been obtained from well-established laboratory procedures. Provided that the standard procedures are followed, the measurements should be quite reliable.

4.2 Nevertheless, there are suggestions of unscrewing some of the threaded reinforcing bars for further testing and verification. As the structure has been largely completed and self-standing, the reinforcing bars in the structure are probably stressed and taking up certain loading. To be able to unscrew a threaded reinforcing bar, one would need to either cut off the bar and coupler assembly in question or hack off more concrete so as to expose the whole reinforcing bar. This will render the reinforcing bar largely useless afterwards as the force originally taken up by it will be redistributed to other parts of the structure and further weaken the same. This is considered unnecessary and therefore not recommended.

4.3 If the unscrewing of some of the threaded reinforcing bars is required for further testing and verification, it has to be of a reasonable sampling size in order to have reliable results. However, this would mean that the structure will be damaged further.

5. Compliance of Works with Requirements of Contract 1112

5.1 According to the design submissions for Hung Hom Station Extension, MTR New Works Design Standards Manual (NWDSM) - Civil Engineering (Rev A4) (Apr 2009) has been adopted for the design.

5.2 Regarding seismic loading in particular, it has been specified in the design reports that:

"All new structures shall be designed to resist seismic loadings. For buried structures, the horizontal ground acceleration behind retaining walls shall be taken as 0.07g in accordance with GEO Report 45 Gravity Retaining Walls Subject to Seismic Loading. It is not necessary to consider the vertical seismic for underground structures. (NWDSM clause 4.4.13.7)

For the assessment or modification of existing structures, seismic shall not be considered."

5.3 Regarding ductility in reinforcement detailing, Clause 4.8.9.3 in Section 4 of MTRCL's NWDSM provides detailing requirements for reinforced and prestressed concrete. Specifically, requirement (x) stipulates that:

"x) Reinforcement detailing and provision to ensure adequate ductility shall be provided in beam and column frame members in accordance with the details set out in Fig. 4.8.9.F1 and Fig. 4.8.9.F2. Particular attention shall be paid to the provision of shear links in the following areas:
a) the top and bottom of columns over the plastic hinge zone;
b) both ends of beams over the plastic hinge zone; and
c) around column vertical reinforcement where the column passes through beams/slabs."

5.4 It is noted that, pursuant to the requirements of NWDSM, Atkins as MTRCL's design consultant has taken into account seismic loading in the structural design of the Works. Accordingly, ductility couplers were specified in contract drawings in certain ductility zones for the connection between the EWL/NSL slabs and the diaphragm walls. The design was formally submitted by MTRCL and accepted by the Building Authority. The EWL/NSL slabs and the diaphragm walls are primary structural components having sizes comparable to or even exceeding those commonly found in beams and columns of buildings. The reinforcement detailing and provision to ensure adequate ductility for beam and column frame members as described in paragraph 5.3 above ought to be applied.

5.5 Available results from the opening up exercise to date indicate that the actual engaged threaded lengths of reinforcing bars in some of the couplers do not comply with the requirement of the manufacturer's specifications, which have raised doubts about the effectiveness of these couplers.

5.6 The introduction of additional construction joints to the slab-wall joint and the lack of control of their proper formation are another major concern. The descriptions of inadequate control of how concrete at the top of the eastern diaphragm wall was broken out (pages 143-144 of transcript of Day 7 of hearing) are consistent with some of the photographs (e.g. Photo 13 at [B17/25581] and Photo 18 at [B19/25587]). In view of the congested working space and the presence of vertical reinforcing bars in the diaphragm wall, it is not surprising at all for the construction joints to have significant variations in shape and position.

5.7 Further, the existence of honeycombing in certain areas will affect not only the concrete strength but also the efficiency of the lapping reinforcing bars in force transfer.

5.8 In addition, upon removal of the defective concrete at certain locations, it is also discovered that the shear links provided do not comply with the contract drawings, thereby casting doubts on the adequacy of the shear strength of the platform slabs.

5.9 Moreover, incomplete bearings (or gaps) have been discovered between the soffit of the EWL slab and the top of certain isolated NSL load-bearing walls and columns. This will affect the integrity and strength of the structure.

5.10 In view of the above, it is evident that parts of the Works are not in compliance with the requirements of Contract 1112.

6. Possible Safety Concerns in Case of Incompliance of Works

6.1 Ductility in Structural Design

6.1.1 In general, ductility is a desirable quality of all structures, irrespective of whether a structure is designed for seismic resistance or not. As mentioned above, in the design of the Hung Hom Station Extension, the structural design consultant of MTRCL, Atkins, designated several ductility zones in the Works within which ductility couplers have to be used, including some of the connections between the diaphragm walls and the EWL slab for both the top and bottom reinforcing bars. Under the consultation procedure, the Building Authority accepted the design and Leighton was required to build the Works accordingly.

6.1.2 It is explained in the *Concrete Code Handbook – An Explanatory Handbook to the Code of Practice for Structural Use of Concrete 2004* (Kwan 2006) that:

> "Ductility is needed not only for earthquake resistance, but also for general structural safety with respect to impact loads, cyclic loads and accidental loads. In actual fact, ductility helps to redistribute the loads from an overloaded and yielded member to the other parts of the structure so that even when a member has been overloaded, it would not collapse immediately. This applies to all kinds of loading and hence ductility helps to promote structural safety regardless of the type of loading applied; in other

words, ductility is crucial to the safety of a structure even when the structure is just subjected to ordinary dead, imposed and wind loads.

From the structural safety point of view, ductility is at least as important as strength, but most engineers tend to pay more attention to strength than to ductility. The inclusion of this section in the new code (i.e. Section 9.9 Detailing for Ductility of the Concrete Code [H/2969-H/2972]) to encourage more refined reinforcement detailing for better ductility of reinforced concrete members and eventually higher survivability and safety of the structure as a whole should be highly commended."

6.2 Role of Bottom Steel Reinforcement at the connection between the EWL Slab and the Diaphragm Wall

6.2.1 The Concrete Code specifies certain detailing requirements for various cases (e.g. clauses in Sections 9.2 [H8/2960-H8/2963], 9.3 [H8/2964-H8/2965] and 9.9 [H8/2969-H8/2972]). Even though the strength calculations show that no reinforcement is required at some locations based on certain simplifying assumptions, these detailing rules provide guidelines for good practice to ensure safety and serviceability of the structure. These detailing rules are mandatory requirements.

6.2.2 Specifically, under normal design load cases, no reinforcement was required for providing flexural and shear resistance at the bottom of the EWL slab at the interface with the diaphragm wall. However, apart from strength consideration, there are other design requirements set out in the Concrete Code, e.g. in respect of ductility and serviceability, which need to be complied with. Failure to comply with such requirements will also cause safety concerns as explained in Section 6.1.2 of this report.

6.3 Additional Construction Joints

6.3.1 According to *The New Penguin Dictionary of Civil Engineering* (Blockley 2005), a construction joint is defined as:

> "A joint in concrete due to a break in the construction... A structure is concreted in stages or sections and the concrete on one section will have hardened before the next section is cast. The joint must not compromise the water resistance and structural integrity of the structure and so the face of one section may require special preparation before constructing the adjacent section."

6.3.2 The formation of construction joints in the construction of reinforced concrete structures is governed by the Concrete Code adopted for the design of the Works. Clause 10.3.10 of the Concrete Code on construction joints [H8/2982] states that, "Their location should be *decided and agreed before concrete is placed*, and should normally run at right angles to the direction of the member".

6.4 Issues Related to Connection between the EWL Slab and the East Diaphragm Wall

6.4.1 *Importance of Slab-Wall Joint to Global Structural Behaviour*

6.4.1.1 In the structural design, it has been assumed that the EWL/NSL slabs are rigidly connected to the diaphragm walls. The diaphragm walls serve both as the foundation supporting the station as well as the retaining structure to support the soil outside the station structure. If the slab-wall joint cannot perform as a rigid joint as expected, the internal forces may be different from those predicted from structural analysis based on the rigid joint assumption. In particular, the mid-span bending moments in the slabs will increase. If the slab-wall joint is improperly detailed and/or poorly constructed, the stiffness of the slab-wall joint will be reduced. In this unfortunate case, not only will the mid-span bending moments in the slab increase, but the structural stability of the station may also be affected, e.g. excessive side-sway of station structure.

6.4.2 *Additional Construction Joints inside the Slab-Wall Joint*

6.4.2.1 The Joint Statement of MTRCL and Leighton (the Joint Statement) [B19/25480-B19/25484] mentioned various changes made to the slab-wall joint. Annex B of the Joint Statement [B19/25487-B19/25493] has provided some further information on the additional construction joints inside the slab-wall joint along the east diaphragm wall.

6.4.2.2 The revised details in the drawings at B19/25487 to B19/25489 and B19/25491 show the formation of additional construction joints at the connection. As the additional construction joints introduced by breaking out part of the completed diaphragm wall may create potential surfaces of weakness, they should be checked by proper structural calculations to ensure that the internal stresses generated at these joints would not be excessive. Apparently, no such calculations have been provided by MTRCL to the Building Authority so far. If the stresses at the actual locations of construction joint are excessive, remedial works may be needed.

6.4.2.3 Some site photographs (e.g. Photo 13 at B19/25581 and Photo 18 at B19/25587) taken show that little control was exercised by the contractor in the breaking out of concrete and that the actual shapes of construction joint were quite different from what is now shown in the drawings attached to the Joint Statement.

6.4.2.4 Besides, the drawings show some arrangements of construction joints with complicated shape, e.g. the inclusion of a convex corner and a re-entrant corner at the same construction joint (see Type 4 on drawing at B19/25487, Type 4 on drawing at B19/25488 and numerous details on drawing at B19/25491). It is to be noted that in the presence of vertical reinforcing bars on both sides of the diaphragm wall, it would be extremely difficult for the contractor doing the trimming down work to ensure accuracy in the positioning of the joint and quality of construction joint preparation.

6.4.2.5 Similar problems described in the previous paragraph are also found in the drawings (i.e. B19/25515, B19/25516 and B19/25520) provided by Leighton under cover of its letter dated 19 September 2018 [B19/25494].

6.4.3 *Structural Design of the Slab-Wall Joint*

6.4.3.1 Just like a coupler that serves as the critical connection between reinforcing bars to enable the assembly to take or transfer forces, the slab-wall joint provides the essential connection between the slab and the wall to enable forces to be transferred through the joint. The slab-wall joint is considered a D-region (i.e. region of discontinuity, disturbance or detail) that behaves differently from the other members (Schlaich and Schäfer 1991). Stresses inside the slab-wall joint are complicated.

6.4.3.2 Theoretically, the through bar that replaces the short reinforcing bars with couplers in the diaphragm wall can only have an axial strength at most the same as that of an assembly comprising a few bars of the same material and cross-sectional area connected by couplers. It is not necessarily an improvement. Moreover, the need for breaking away part of the completed diaphragm wall also introduces additional construction joints of uncertain profiles within the slab-wall joint. It is necessary to check that the stresses at the construction joints are not excessive; otherwise the slab-wall joint should be properly strengthened. It is premature to jump to any conclusion regarding each slab-wall joint design without proper calculations for verification. Further, the slab-wall joint should be checked numerically to ensure that the forces carried by the concrete and main reinforcement outside the joint would be carried and transferred safely within the joint through proper strut-and-tie action.

6.4.3.3 Recently, Atkins has submitted design calculations about the performance of the revised slab-wall joint details (Atkins's Report) [J6/4557-J6/4562]. In Atkins's Report, only some calculations for a typical slab-wall joint are provided, it is certainly not enough. Calculations must be carried out on <u>all</u> design variations of slab-wall joint to ensure their safety. There are a lot of problems in the calculations in Atkins's Report. In view of the large number of variations of slab-wall joint details, a typical detail shown schematically in Figure 6.4.3.3.1 is used for discussion.

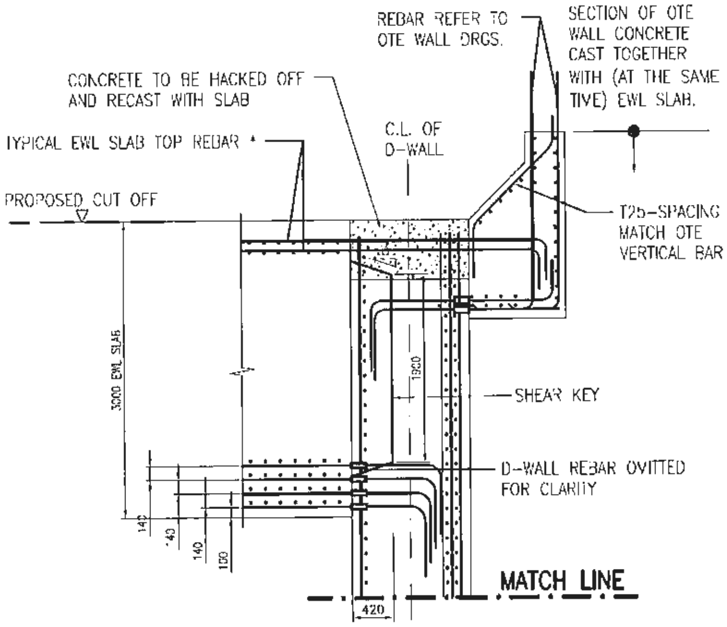

Figure 6.4.3.3.1 A typical slab-wall joint (from Detail 1 of [J6/4071])

6.4.3.4 In page 3 of 5 of Atkins's Report [J6/4559], Atkins attempts to calculate the horizontal shear stress at the additional construction joint inside the slab-wall joint. However, these calculations for the horizontal shear stress at the additional construction joint are unacceptable. The equation used (i.e. $v_h = 6\ F\ (D^2/4 - y^2)\ /\ (B\ D^3)$) is for evaluation of shear stresses in a homogeneous beam under shear due to flexure at elastic state. It does NOT give any estimate at all of the horizontal shear stress generated at the additional construction joint inside the slab-wall joint. It gives the shear stress within a beam at the elastic stage only. A reinforced concrete beam is not homogeneous as it comprises both concrete and steel reinforcement, and hence the equation used by Atkins is unsuitable. Equation 6.19 of the Concrete Code [H8/2875] is normally used to estimate the shear stress of beams and slabs at ultimate limit state and the behaviour is assumed to be inelastic.

6.4.3.5 In the lower half of page 3 of 5 of Atkins's Report [J6/4559], Atkins refers to a cross sectional diagram of the revised joint (which is reproduced in Figure 6.4.3.5.1 below) claiming that the pink down-stand part of OTE wall can act as a shear key to help resist the horizontal shear stress at the additional construction joint. However, no calculation has been provided by Atkins to substantiate this claim. It is noted that the bearing force between the diaphragm wall (shown in white) and the down-stand part of OTE wall (shown in pink) acts at an eccentricity on the new concrete cast at a later date to replace the concrete at the top of the diaphragm wall hacked away (shown in pink), thereby also creating a bending moment. Apparently, the top reinforcement has not been designed to resist the combined tension and bending moment. It is uncertain if the horizontal shear resistance at the additional construction joint and the bearing resistance at the down-stand can indeed act together but not fail progressively. It should be demonstrated by calculations and/or experiments.

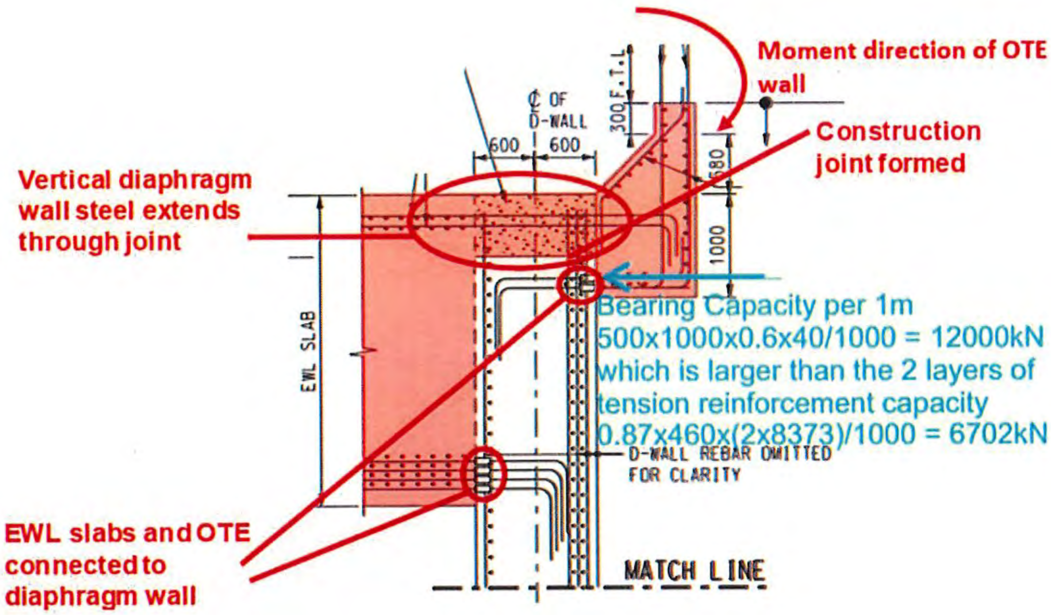

Figure 6.4.3.5.1 Assumption of additional shear key in slab-wall joint [J6/4559]

6.4.3.6 Further, in its Report, Atkins has not considered the stresses inside the slab-wall joint using the principles underpinning the design of common beam-column joints as described in Section 6.8 of the Concrete Code [H8/2924-H8/2926] on standard beam-column joints. Considering just the possible cracks outside the joint does not address the issue at all. If the same rationale adopted by Atkins were followed with the analysis of the standard beam-column joints in common buildings, there would not be any need for checking the joint shear stress (Clause 6.8.1.3 of the Concrete Code) and providing horizontal joint shear reinforcement (Clause 6.8.1.5 of the Concrete Code) <u>inside</u> the joint. A simple check of its available strength to satisfy equilibrium can give an initial indication of whether there is any problem.

6.4.3.7 Figure 6.4.3.7.1 below shows the slab-wall joint enlarged for clarity. From the structural analysis, it is possible to identify certain critical bending moment, axial force and shear force at the bottom of slab-wall joint (i.e. Section A-A) at the ultimate limit state. Based on these internal forces, the designed tension taken by the group of reinforcement near the outer face of the diaphragm wall at level A can be determined. The forces taken by these reinforcing bars at level D should be virtually zero. Therefore Section X-X from level A to level D is a critical shear plane as the shear stresses there can be very high and they should be checked numerically. Similar design checks for various critical sections within the joint should be performed for each and every type of cross-sectional details adopted along the east diaphragm wall.

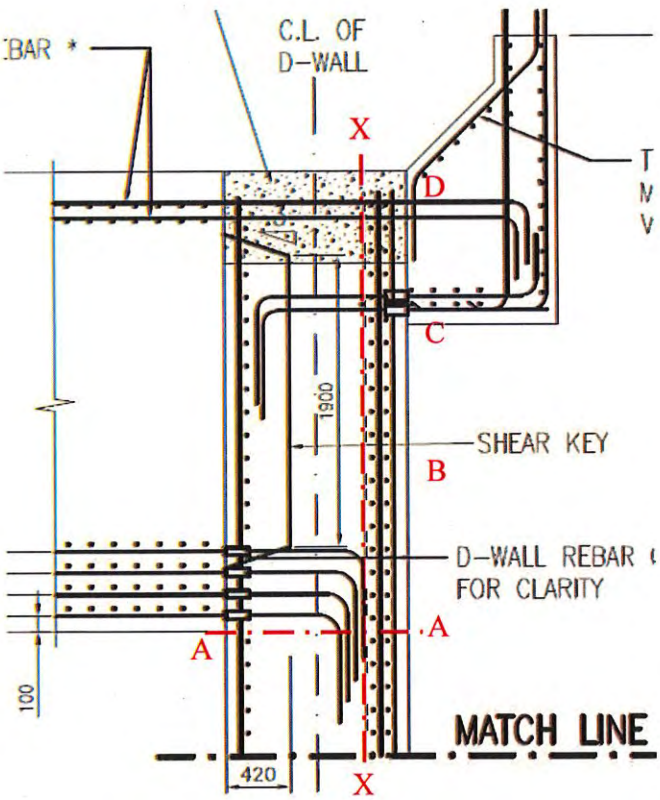

Figure 6.4.3.7.1 A typical slab-wall joint

6.4.3.8 Assessment Report Revision A of 9 November 2018 of Ove Arup & Partners Hong Kong Ltd. (OAP's Report) [B19/25114-B19/25156] has commented on the design changes of the slab-wall joint.

6.4.3.9 Section 2.7 on page 7 of OAP's Report describes the evolution of reinforcement in slab-wall joint qualitatively. To ensure structural safety, the effects of omission of L-bars and U-bars in the evolution of reinforcement in slab-wall joint on the internal forces within the joint should be checked numerically.

6.4.3.10 Section 2.8 on page 7 of OAP's Report describes modifications to the top of east diaphragm wall as an improvement. However, it does not mention the introduction of additional construction joints needed for such modifications, which are a cause for concern. The construction joints are interfaces of potential weakness. The stresses at the additional construction joints inside the highly-stressed connection should be checked numerically and verified to be acceptable; otherwise strengthening works will be necessary.

6.4.3.11 Certainly, unauthorised alteration of the slab-wall joint detail is an issue. The stress state in the slab-wall joint should be checked. The stresses at the additional construction joints should be checked for compliance. Without proper checking and verification, it appears premature to conclude in Section 5.2 on pages 12-13 of OAP's Report that "it represents a structural enhancement to the works" (5th paragraph). Even in a normal construction contract without unauthorised alteration of details, it is necessary to keep a full set of site records to demonstrate compliance. Without proper records as in this case, it is doubtful if one can conclude to "believe the Contractor's Alternative Proposal was generally implemented and constructed to an acceptable standard" (6th paragraph). Therefore there is a genuine need for getting additional evidence.

6.5 Strength Utilisation, Redundancy and Robustness

6.5.1 Section 4 on page 10 of OAP's Report has commented on the strength utilisation, redundancy and robustness of the structure. There are however some inaccuracies in this section as some of the technical terms are not used accurately, particularly the concepts of "redundancy" and "robustness" as described below.

6.5.2 The assumptions based on which the chosen values of internal forces (e.g. bending moment, shear force, etc.) for evaluation of strength utilisation factors are uncertain. For example, the modification to the slab-wall joint reinforcement may affect its ability to function as a monolithic connection. Such structural behaviour should be considered. Most likely the strength utilisation factors shown are based on the intact structure. The existence of honeycombing in certain areas will affect the lapping reinforcing bars thereby affecting their efficiency in force transfer even after the repairs. These issues and other defects will affect the actual strength utilisation factors and should be taken into account properly.

6.5.3 The description of the stresses in high yield deformed reinforcing bars is noted. They are typical of design according to the Concrete Code (i.e. $f_y / \gamma_m = 460/1.15 = 400$ MPa). If the strength utilisation factor is exactly 100%, the design just complies with the Concrete Code. The safety margin is an inherent requirement of the Concrete Code. If the strength utilisation factor exceeds 100% (e.g. 101%), it fails to comply with the Concrete Code.

6.5.4 According to The New Penguin Dictionary of Civil Engineering (Blockley 2005), the term "redundancy" as used in structural engineering is defined as: "The inclusion of extra components in a system over those that are absolutely essential – often to increase robustness. A redundant structure is one that contains extra members, or extra fixity in the joints, over a just-stiff frame which would become a mechanism *[e.g. a framework so constructed that substantial relative movements are possible, which in layman term means becoming unstable]* if just one member or degree of fixity were removed. If designed properly, redundancy can make a structure more robust since more damage is required to make the structure fail."

6.5.5 The low working stress in reinforcement mentioned in the 6th paragraph of Section 4 of OAP's Report does not imply "a high level of redundancy in the structure". As unfactored loads are used for design checking of the serviceability limit state (SLS), the stresses under the SLS are certainly lower than those at the ultimate limit state (ULS). This is hardly surprising and is in fact a requirement of the Concrete Code. For example, a simply supported beam (i.e. a beam supported by a hinge at one end and a roller at another end) designed to carry very low stresses does not have any redundancy at all. Adding more roller supports under the beam will introduce more redundancies. In summary, redundancies result from the structural configuration and support systems, but not the level of working stress.

6.5.6 Merely adopting the prescribed design stress-strain curves (Figures 3.8 and 3.9 of the Concrete Code [H8/2851-H8/2852]) for design does not automatically provide robustness to a structure. According to Pearson and Delatte (2005), building codes in many countries have adopted structural integrity or "robustness" provisions that may be directly traced to the Ronan Point collapse in East London on 16 May 1968, in which the explosion on the 18th floor brought down four flats above it, leading to a progressive collapse of one corner of the block. Clause 2.2.2.3 (a) of the Concrete Code provides for the requirement of robustness as: "Structures should be planned and designed so that they are not unreasonably susceptible to the effects of accidents. In particular, situations should be avoided where damage to small areas of a structure or failure of single elements may lead to collapse of major parts of the structure." Section 2.2.2.3 of the Concrete Code [H8/2838-H8/2839] describes various measures to ensure robustness, e.g. provision of ties, use of notional horizontal design ultimate load, etc. Therefore the following statement in the 3rd paragraph of Section 4 of OAP's Report is inaccurate: "there is therefore a substantial margin of reserve strength and robustness even if the strength utilisation is 100%".

6.6 Miscellaneous Workmanship Issues

6.6.1 MTRCL should provide calculations to assess the effects of honeycombing on the structural behaviour taking into account the construction sequence and the possible problems at the slab-wall joints as explained above. Section 5.3 on page 13 of OAP's Report mentions that "Atkins conclude that the structure has sufficient redundancy to accommodate the redistribution of stress associated with the honeycombing" (3rd paragraph). A report by Atkins (page [H13/7682]; Conclusion 3) states: "The worst case is checked using 400 mm deep concrete lost at these locations". Checking by assuming a loss of 400 mm due to honeycombing at the soffit in this 3000 mm thick slab is inappropriate if the part of slab is taking sagging moment. Honeycombing at the soffit is observed at certain locations where 40 mm reinforcing bars are lapping with each other. Obviously they are not taking the intended forces assumed in the calculations. Because of the extent of honeycombing, some reinforcing bars in certain layers will not be taking up the forces as designed, thereby affecting the strength of slab in the vicinity. Even after the honeycombing is repaired, the reinforcing bars affected will not contribute to the support of existing loading. They will only be effective in supporting future loading to be applied after the repairs. More rigorous calculations should be carried out to assess the impact of honeycombing on the structural behaviour and strength of the structure.

6.6.2 Little has been said in Section 5.5 on page 13 of OAP's Report on the possible problems associated with problematic installation of shear links. Sufficient calculations should be carried out to verify if the shear links provided are sufficient in providing the necessary shear strength; otherwise strengthening works should be carried out.

6.6.3 Section 5.5 on pages 13-14 of OAP's Report has mentioned the incomplete bearing between the EWL slab soffit and the top of isolated NSL loadbearing walls and columns. During the construction process, there was a stage at which the loads in some columns supporting the existing concourse were transferred to the EWL Slab. Any adverse effects due to incomplete bearing between the EWL slab soffit and the top of isolated NSL load-bearing walls and columns on the EWL slab should be investigated properly, e.g. possibly over-stressing the EWL slab in the vicinity and/or the vertical reinforcing bars connecting the EWL slab and columns.

7. Need for Remedial or Strengthening Works

7.1 If there are problems found in the slab-wall joint such that effective transfer of forces within it cannot be achieved, the slab-wall joint should be strengthened, if considered appropriate. Possible measures include the installation of steel bar anchors.

7.2 Non-compliant reinforcing bar couplers suffer in various aspects, including but not limited to strength and deformation characteristics. After completion of the investigation and obtaining a defect rate based on 95% confidence level, design checking should be carried out to determine if strengthening measures have to be taken, and if so what.

7.3 Should the couplers at the top of the slab-wall joint of the EWL slab be found to cause any issue, additional steel reinforcement may be provided either internally or externally depending on various restrictions. Should the couplers at the bottom of slab-wall joint of the EWL slab be found to cause any issue, further investigation may be necessary to identify the possible strengthening measures.

7.4 Honeycombing should be repaired with suitable material and by a suitable method so as to ensure proper bonding between the substrate and the reinforcement.

7.5 If strengthening of the shear resistance of the EWL slab is necessary, one of the possible measures would be by installing additional steel bar anchors.

7.6 The incomplete bearing (or gaps) between the soffit of EWL slab and the top of certain NSL load-bearing walls and columns should be remedied by grouting or other similar measures.

8. Possible Further Steps

8.1 It may be desirable to install a long-term structural health monitoring system to monitor the variations of displacements and deformations at key locations of the structure, which will help to assess the structural performance. However, it should be noted that a structural health monitoring system installed after completion of a structure can only monitor the subsequent responses but not the existing stresses and deformations.

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