EXPERT REPORT

PREPARED BY

PROFESSOR DON McQUILLAN

Expert Witness appointed by the 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 January 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	:	The 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 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.
Diagrams integral to this Report prepared by the Author	:	See **Appendix II**
Photographs taken during inspection of the diaphragm walls and platform slabs at the Hung Hom Station Extension on Monday 5 November 2018 by the Author	:	See **Appendix III**
Photographs taken during visit to the factory of BOSA Technology (Hong Kong) Limited ("**BOSA**") on Tuesday 6 November 2018 by the Author	:	See **Appendix IV**

Thread Strength Calculation Table for a Seisplice coupler system prepared by BOSA	:	See **Appendix V**
Photographs taken during visit to the Construction Industry Council ("**CIC**") on Thursday 8 November 2018 by the Author	:	See **Appendix VI**
Dates of meetings with relevant persons	:	See **Appendix VII**
D-wall deflection analysis carried out by RPS as directed by the Author	:	See **Appendix VIII**
Enlarged annotated copy of a Photograph supplied by China Technology Corporation Ltd ("**Chinat**") **[D1/228]**	:	See **Appendix IX**
Photographs of the first two inspections of opened-up locations	:	See **Appendix X**
Agreed Expert Memorandum signed on 18 December 2018 (transcript plus original)	:	See **Appendix XI**

<u>The Terms of Reference of the Commission are as follows:</u>

In respect of the diaphragm wall and platform slab construction works at the Hung Hom Station Extension under the MTR Corporation Limited (**MTRCL**)'s Contract No. 1112 of the Shatin to Central Link Project,

(a) (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 (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 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.

<u>Instructions</u>

I have been instructed to give my opinion on the matters under paragraph (a) of its Terms of Reference.

In providing my opinion, I have also been instructed to consider the following areas and undertake the following tasks:

(a) Defective Steel Works

(i) Consider and assess allegations in relation to Defective Steel Works and the evidence received by the Commission thereon (including factual and expert evidence) and ascertain and provide my expert opinion on whether, based on the evidence available to me, Defective Steel Works exist in the diaphragm walls and platform slabs, and if so, the extent thereof.

(ii) For the purpose of arriving at my conclusions, analyse and provide my opinion on feasible and effective methods to ascertain whether and if so, the extent of such Defective Steel Works, given the existing state of the diaphragm walls and platform slabs.

(iii) Assess and provide my expert opinion on whether and if so, how and to what extent the diaphragm walls and platform slabs or parts thereof should be opened up for examination and inspection.

(iv) With reference to the terms of Contract 1112, sub-contract(s), approved plans, drawings, laws and regulations, practice notes, handbooks, guidelines, circulars, industry standards, practice and requirements (“Requirements, Standards and Practice”), identify whether and to what extent the steel fixing works in the diaphragm walls and platform slabs were in breach of Requirements, Standards and Practice.

(v) On the basis of my opinion and assessment in the foregoing sub-paragraph, provide my expert opinion on the causes of the Defective Steel Works (if any) and reasons for the non-compliance of the Requirements, Standards and Practice. Advise the Commission on whether such Defective Steel Works were as a result of poor workmanship, inferior quality of materials supplied and/or due to other factors.

(vi) Based on my findings under this heading, assess whether any rectification works have been carried out or any attempt has ever been made by any of the Involved Parties or other persons to rectify such Defective Steel Works to ensure the safety and integrity of the diaphragm walls and platform slabs. Provide my expert opinion on feasible and effective methods to rectify such Defective Steel Works and/or strengthen the structure of the diaphragm walls and platform slabs to ensure the safety and integrity thereof.

(b) Load Test

(vii) In relation to the Load Test Proposal read the witness statement of Li Tsz Wai, Ralph provided by THB **[G3/2088+** and §§30-31,**G3/2097]**. According to the Summary of Key Events annexed to that witness statement **[G3/2101]**, the expert engaged by MTRCL to provide the Load Test Proposal is C M Wong & Associates Limited.

(viii) Review and evaluate the Load Test Proposal of the expert appointed by MTRCL and advise the Commission on whether and to what extent I agree with the Load Test Proposal. Explain and identify any concerns and issues relating thereto. If I disagree with any aspects of the proposal, provide my views on how the load test should be conducted and what variations should be made.

(ix) If I have any alternative views on how to test the quality, safety and integrity of the diaphragm walls and platform slabs to ensure that they are structurally sound, provide my opinion and advise the Commission.

(c) Expert Team

(x) As regards the work of the Expert Advisory Team, while it appears to overlap significantly with the work of the Commission, review and comment on any interim and final report(s) of the Expert Team when such reports become available.

(d) <u>Deviations from approved drawings and discrepancies between site records and design details</u>

(xi) In relation to the alleged deviations from approved drawings and discrepancies between site records and design details, review the drawings as approved by the BD and the as-built drawings on the diaphragm walls (if there is any, or otherwise, devise a method to ascertain the actual as-built details on site) and platform slabs and other relevant drawings and documents and ascertain the extent of any deviations and discrepancies between site records and the actual as-built design details.

(xii) In the light of my assessment provided under the preceding sub-paragraph, provide my expert opinion on the structural implications of such deviations and discrepancies and confirm whether the deviated design is structurally sound. Advise the Commission of any safety issues and concerns arising out of such deviations and discrepancies and provide my expert opinion on how to rectify and ensure the safety and integrity of the relevant structure(s).

(e) <u>“Honey comb” structures</u>

(xiii) Advise on the causes of the “honey comb” structures in the concrete at the soffit of EWL slab and the structural safety implication thereof. Advise whether there is any connection between the “honey comb” structures and the alleged Defective Steel Works and/or the alleged deviation from approved drawings mentioned hereinabove.

(xiv) Provide an opinion on possible measures to be taken to eliminate the formation of such structures and to rectify the repair the EWL slab to ensure that the EWL platform slab is structurally safe.

(f) Other works which raise concerns about public safety

(xv) The THB has identified one more matter which raises concerns about public safety (see §32 of the witness statement of Li Tze Wai, Ralph, **[G3/2097-2098]**). There was an allegation that Chinat refused to pour light weight mass concrete into a void or voids at the Hung Hom Extension. MTRCL has submitted on 2 August 2018 a "Form C" regarding "Costs Reduction for Using Broken Concrete in lieu of Part of the Mass Concrete to fill Voids in Hung Hom Station" **[G6/5041]**. HyD wrote to MTRCL on 8 August 2018 to seek further clarification and information on this subject **[G6/5157-5158]**.

(xvi) Advise the Commission how such allegation, if substantiated, may affect the safety and integrity of the diaphragm walls and platform slabs and how such defect may be rectified to ensure the safety and structural integrity thereof.

(xviii) Finally, on the basis of my review of the evidence (both factual and expert), drawings, documents and materials, advise whether, in respect of the diaphragm wall and platform slab construction works at the Hung Hom Station Extension under Contract 1112 of the SCL Project, there are any other works which raise concerns about public safety and if so, explain the facts and circumstances surrounding such works and advise on how such concerns about public safety may be addressed and eliminated.

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 my report and the hearing of the Commission, there is any change in circumstances which affect my opinion above.

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.

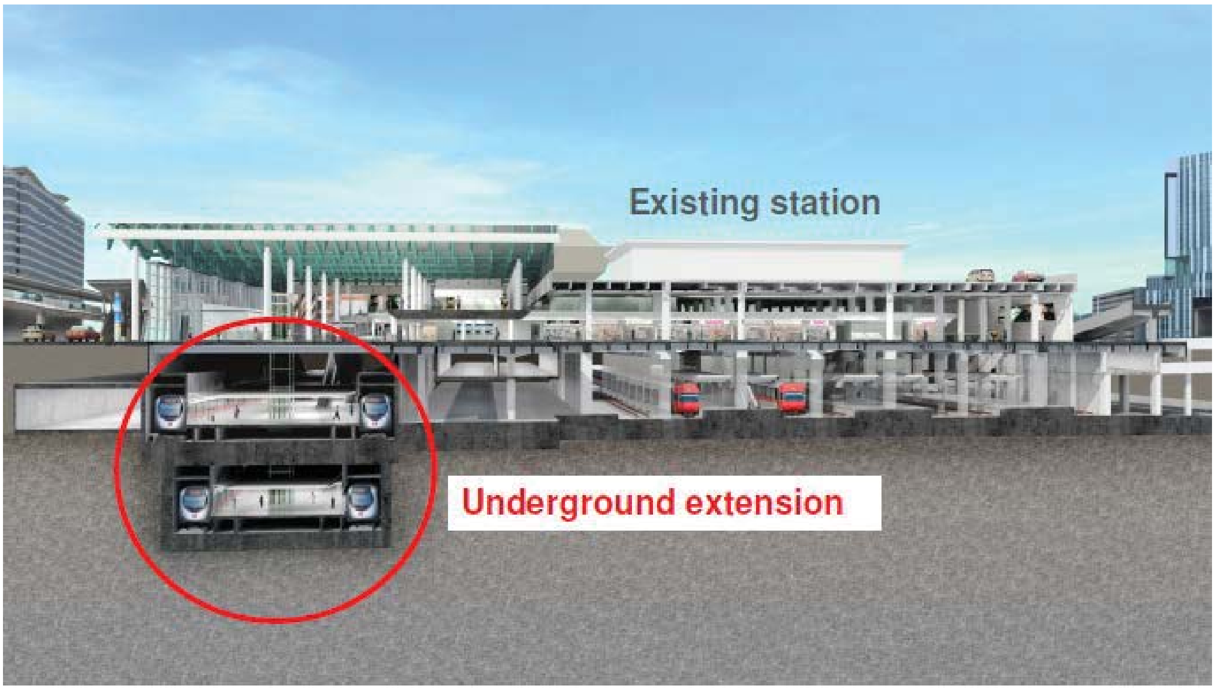
Professor Don McQuillan
6 January 2019

INTRODUCTION

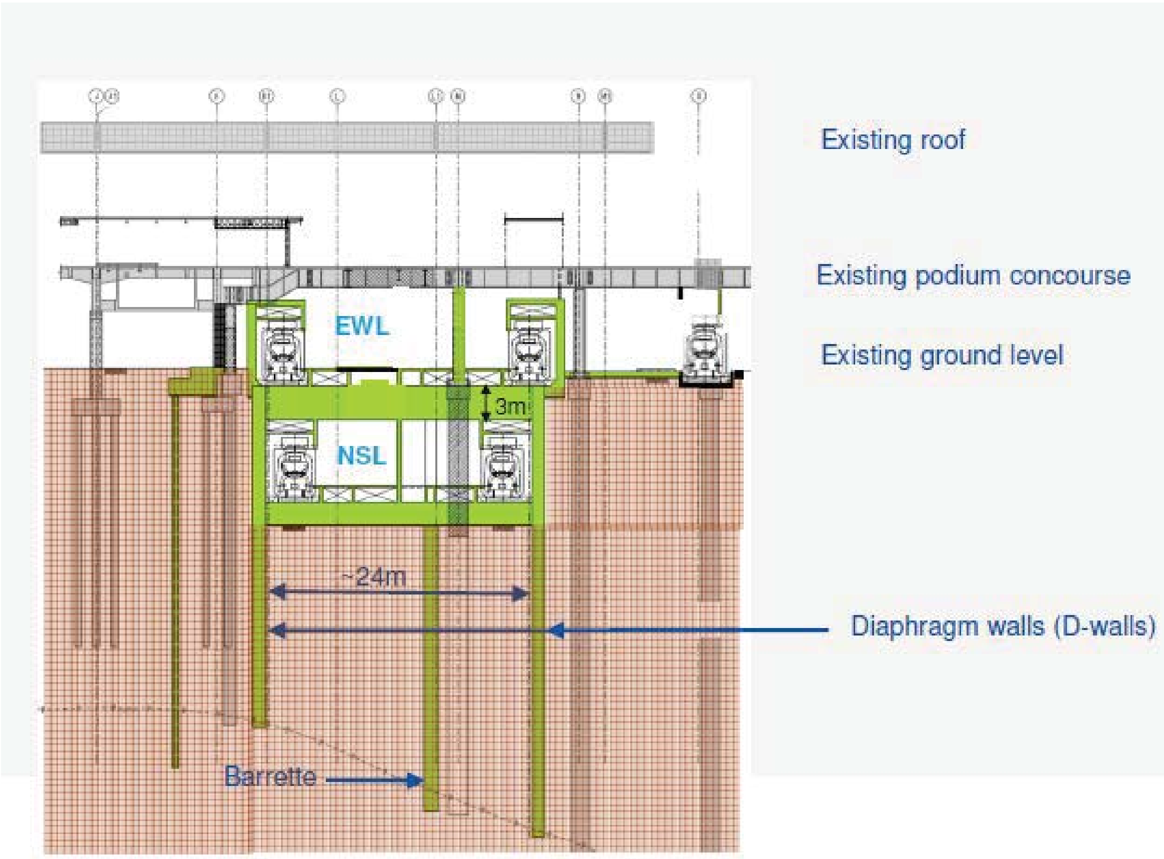
1. I, Professor Don McQuillan, have been appointed as the Commission's independent Expert Engineer, to assist the Commission under the Terms of Reference in determining the matters relating to the diaphragm wall and platform slab construction works at Hung Hom Station. I am a Chartered Engineer specialising 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. The background to the Inquiry is well documented. The strategic Shatin to Central rail infrastructure link (SCL), delivered under Contract 1112, focuses on the existing Hung Hom Station which required to be extended sideways (to the west) and underground to facilitate the inter-connection of the East – West Corridor (EWL) and the North – South Corridor (NSL) by forming two new platforms. The following diagram (courtesy of SNC-Lavalin Atkins) **[J4/3327]** illustrates by means of a cross-section the juxtaposition of the new underground extension to the existing station. The upper platform will be referred to in this report as the "EWL slab" and the lower one as the "NSL slab".



3. The following simplified diagram[1] **[J4/3330]** shows the new scheme more clearly.



4. The project layout[2] **[A1/22 and A1/250]** shows the overall site, which is over 400m long, subdivided in terms of gridlines (GL) as follows:

- Area A: GL0 – GL7
- Area HKC (Coliseum) : GL7 – GL15
- Area B: GL15 – GL22
- Area C: GL22 –GL49.5 which is sub-divided further into
  - Area C1: GL22 – GL31
  - Area C2: GL31 – GL40
  - Area C3: GL 40 – GL49.5

[1] Also courtesy of SNC-Lavalin Atkins.
[2] MTRCL PowerPoint presentation for the Commission on 21 September 2018 **[A1/22].**

5. The following summary chronology produced by MTRCL[3] **[A1/20]** is noted:

- The contract commenced on 15 March 2013
- The D-wall construction was implemented between July 2013 and July 2015.
- The EWL slab was constructed between May 2015 and August 2016.
- The NSL was constructed between December 2015 and May 2016.

6. At my meeting with the MTRCL **[Appendix VII-1]**, I was informed that the project was estimated to be more than 95% complete with the EWL line due to open in mid-2019. I was also informed that train testing commenced in April 2018 and was ongoing.

7. In January 2017 allegations were made by a sub-contractor about the efficacy of the connection between the new platform slabs and the diaphragm wall, specifically that threaded steel starter bars for the platform slabs had been cut and/or improperly connected so as to affect the integrity of the mechanically coupled steel rebar joints. Appropriate remedial action was deemed to have been implemented[4] **[C12/7922-7927]**. Towards the end of May 2018, reports appeared in the local media which suggested that steel fixing works in both the diaphragm walls and platform slabs at the Hung Hom Station Extension might be defective which generated significant public safety concerns **[G3/1760]**[5].

8. The Highways Department ("**HyD**") instructed MTRCL to produce a report to allay concerns[6] **[G3/1777-1778**]. The report was issued on 15 June 2018 ("**MTRCL Report**") **[B1/1-46]** but within a short space of time was found to be inaccurate **[G3/1763-1764]**[7].

9. On 10 July 2018 the Chief Executive in Council set up a Commission of Inquiry[8] **[A1/1]**.

10. According to the MTRCL Report, there were some incidents of non-compliant coupler connections which were discovered and remediated in the period between

[3] MTRCL PowerPoint presentation for the Commission on 21 September 2018 **[A1/20]**
[4] Email dated 6 January 2017 from Chinat to Mr Anthony Zervaas of Leighton Contractors (Asia) Limited ("**LCAL**") **[C12/7923-7924]**
[5] § 21 of WS of Chan Fan **[G3/1760].**
[6] Letter dated 31 May 2018 from HyD to MTRCL **[G3/1777-1778]**
[7] §31 of WS of Chan Fan **[G3/1763-1764]**
[8] Gazette No. 5166 **[A1/1]**

August 2015 and December 2015[9] **[B1/33-35]**. A subsequent review also uncovered the fact that the detail originally approved for connecting the top of the EWL platform slab to the east diaphragm wall had been altered allegedly without the knowledge of the Buildings Department ("**BD**") [10] **[G3/1763-1764]**. In addition it was found that sign-off records were either retrospectively prepared, deficient and/or missing[11] **[H20/40054-40059]** and no reliable "as-constructed" drawings were available[12] [**H7/2206**].

11. In addition to the Commission of Inquiry the Government also established an Expert Adviser Team ("**EAT**") to review the construction and long-term performance of the works on a holistic basis [13]**[G3/1766] [G3/1850-1851]**. A report produced by MTRCL **[B20/26099–26136]** approved by the EAT was recently published on 4 December 2018 with recommendations for a three-staged approach to investigate the as-built details and to ensure the long-term integrity and safety of the structures are not detrimentally affected[14]**[B20/26099 – B26136]**.

<u>Parties involved</u>

12. The main parties involved in the design and primary construction were:

- MTR Corporation Limited (**MTRC**L)

  MTRCL was appointed by the Government under an Entrustment Agreement dated 29 May 2012 to procure and project manage the design, construction, commissioning and delivery of the SCL project[15] **[B1/2]**.

- Atkins China Ltd (**Atkin "A"**)

  Atkins "A" was appointed by MTRCL as project engineer for the structural, civil and geotechnical design of the works[16] **[J1/58][B10/7817-7822][C27/20802]**.

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[9] §6.2, MTRCL Report **[B1/33-35]**
[10] §31, WS of Chan Fan **[G3/1763-1764]**
[11] §§2-12, WS2 of Ho Hon Kit **[H20/40054-40059]**
[12] §68(3)- §68(4), WS of Lok Pui Fai **[H7/2206]**
[13] §§37-38, WS of Chan Fan **[G3/1766].** Terms of Reference of EAT **[G3/1850-1851]**
[14] Holistic Proposal for Verification & Assurance of As-constructed Conditions and Workmanship Quality of Hung Hom Station Extension, Rev B **[B20/26099 – B26136]**
[15] §1, MTRCL Report **[B1/2]**
[16] §11, WS of John Blackwood **[J1/58]**; Section 1.12.1, Consultancy Agreement C1106 **[B10/7817-7822]**; §12, WS of Brett Buckland **[C27/20802]**

- Transport and Housing Bureau ("**THB**"), Highways Department ("**HyD**") and the Railways Development Office ("**RDO**")

  THB is responsible for the general oversight on the planning and implementation of all cross boundary and domestic railway projects including the SCL project, HyD operates under THB and is involved in planning, monitoring, and coordinating various activities associated with the implementation of the SCL Project. It also chaired the Project Steering Committee monthly meetings and through the RDO monitored on-site construction and formed part of the MTRCL's Project Control Group[17] **[G3/1754-1755][G3/2063-2069]**.

- Development Bureau ("**DEVB**") and the Buildings Department ("**BD**")

  DEVB oversees the policy on urban renewal, private building control and land registration. The Works Branch thereof is responsible for, amongst other duties, the management of the approved lists of public works contactors who are eligible to tender for public works contracts **[H6/1124][H6/1129]**[18]**.** BD is the government department responsible for enforcing the Buildings Ordinance, which covers the planning, design and construction of buildings and associated works. The building works of the SCL Project are regulated by the BD via the Instrument of Exemption regime[19] **[H7/2108-2114]**.

- PYPUN-KD & Associates Limited ("**PYPUN**")

  PYPUN was appointed by the Government as the Monitoring and Verification (M&V) Consultant to assist HyD with monitoring and verification of certain aspects of he works of the SCL Project[20] **[K1/12]**.

- Leighton Contractors (Asia) Ltd ("**LCAL**")

  LCAL was the registered Main Contractor appointed to construct the works[21]**[C11/7595]**.

- Atkins China Ltd (**Atkin "B"**).

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[17] §§14-36, WS of Chung Kum Wah [G3/2063-2069]; §7, WS of Chan Fan [G3/1754-1755]
[18] §1, WS of Maurice Loo [H6/1124]; §6, WS of Chau Siu Hei [H6/1129]
[19] §§4-18, WS of Cheung Tin Cheung [H7/2108-2114]
[20] §8, WS of Mak Yu Man [K1/12]
[21] §9, WS of Karl Speed [C11/7595]

Atkins "B" was appointed by LCAL as its temporary works design engineer and design consultant [22] **[J1/58][J1/46-48][C27/20804]**.

- Intrafor Hong Kong Limited **("Intrafor")** was appointed by LCAL as the registered sub-contractor for the specialist reinforced concrete diaphragm wall (**D-wall**) construction[23] **[F1/51]**.

- **Hung Choi** was appointed by Intrafor as the sub-contractor for the steel fixing of the D-walls[24] **[I1/21]**.

- **Fang Sheung** was appointed by LCAL as the sub-contractor for the steel fixing of the reinforced concrete platform slabs[25] **[E1/29.1]**.

- China Technology Corporation Limited ("**Chinat**").

  Chinat was appointed by LCAL as the sub-contractor for the formwork and concreting of the platform slabs[26] **[D1/16]**.

- **BOSA** was appointed by LCAL as the supplier of the mechanical couplers and threaded steel reinforcement starter bars [27] **[C6/4842+]**.

THE STRUCTURES

Provisional "as-constructed" drawings

13. Based on photographic evidence taken during construction, and other records, MTRCL and LCAL recently compiled "as-constructed" drawings at the intersection of the EWL slab, eastern diaphragm wall and OTE slab (at Areas B and C)[28] **[B19/25480-25689][C26494-C26495]**. These, however, are currently regarded as "provisional" because of some uncertainties which require verification by opening-up

[22] §12, WS of John Blackwood **[J1/58]**; Schedule 2, Consultancy Agreement **[J1/46-48]**; §19, WS of Brett Buckland **[C27/20804]**
[23] §98, WS of Jean-Christophe, Jacques-Oliver Gillard **[F1/51]**
[24] §3, WS of Chui Tim Choi **[I1/21]**
[25] §1, WS of Pun Wai Shan **[E1/29.1]**
[26] §19.6, WS of Jason Poon **[D1/16]**
[27] Subcontract SC011 with BOSA **[C6/4842+]**
[28] Joint statement made by LCAL and MTRCL **[B19/25480-25689]**

parts of the completed structure. This process of invasive investigation has recently commenced but is expected to extend through to at least the end of February 2019[29].

Description of structure

14. The extension comprises an underground reinforced concrete box tunnel with the top of the EWL slab located at approximately existing ground level and the top of the NSL slab located some 10,620mm below existing ground level as illustrated by the schematic diagram at **Appendix II-1**.

15. The slabs span between D-walls which are spaced at approximately 20,220mm (internal dimension). The slabs are rigidly connected to the D-walls with full fixity joints. In addition there is a shear key at each interface. The method originally specified and approved for connecting the slabs to the D-walls was to use mechanical rebar couplers cast into the D-walls for both the top and bottom steel reinforcement.

16. The D-walls are 1,200mm thick[30] **[J4/3329]** and are formed of intermittent "hit" and "miss" panels. The "hit" panels are founded on competent bedrock and, in addition to the obvious function, during both the temporary and permanent phases of construction, of retaining the external soils and preventing water penetration, carry the vertical loads to bedrock. They are identified as e.g. "EH" or "WH" (where "H" means "hit"). The "miss" panels are infills between the "hit" panels. These are taken to a shallower depth and, once constructed; the ground around the bottom of the panels is pressure-grouted using cast-in tubes, to form a water seal. They can be identified as e.g. "EM" or "WM" (where "M" means "miss"). Each D-wall "hit" panel has a pre-formed stop end which incorporates a water bar to seal the interface. The varying depth of each D-wall panel results in different vertical and lateral bending stiffnesses. The "miss" panels are not designed to take the vertical load.

17. The schematic diagram in **Appendix II-2** illustrates the general construction sequence of a D-wall. As a panel trench is being excavated, bentonite "mud" is inserted to prevent the sides collapsing. The steel reinforcement cage (rebar) is then inserted complete with cast-in components e.g. grout and coring tubes, stop ends and mechanical couplers etc. Then concrete is pumped into the trench panel

[29] Appendix C (Stage 2a) to Holistic Proposal Rev B **[B20/26136]**
[30] Atkins' presentation **[J4/3329]**

via cast-in "tremie" tubes; the concrete being denser than the bentonite displaces the bentonite and fills the trench bottom-up. The tops of the D-walls are always constructed higher than required, for reasons which will be explained later, and are then cut-down to the structural formation level.

18. The upper 3,000mm deep reinforced concrete EWL slab is the first element to be constructed, following the D-wall installation, using the excavated ground surface, once prepared, as the soffit formwork. Then "top-down" construction techniques, in conjunction with de-watering, are used to excavate below the EWL slab and to construct the lower 2,000mm thick NSL slab, again casting it on the excavated and prepared ground surface. The combined mass of the two slabs is designed, in the permanent condition, to resist buoyancy (i.e. the tendency of the tunnel to float upwards).

19. In order to prevent potential failure of the D-walls in the temporary excavation stages and to limit their deflections, temporary raking propping, referenced as an "Excavation Lateral Support" ("**ELS**") system, was installed to support the D-walls off the soffit of the thick EWL slab in order to allow excavation to proceed to the lower level required for the NSL slab.

20. Construction joints ("**CJs**") are provided longitudinally in both the EWL and NSL slabs so as to allow for shrinkage and other movement, and to allow construction to proceed in manageable lengths of between 10m and 20m. Alternate panels are constructed and then the gap panels are in-filled, as will be discussed later. Steel rebar continuity across the CJs for both the top and bottom mats also relies on mechanical couplers.

21. External to the east and west D-walls, there is a cantilevered slab and wall structure to support the extract ventilation equipment above the trains; known as the Over Track Exhaust ("**OTE**").

<u>Staged-construction sequence</u>

22. It is important to understand the staged-construction sequence as a prelude to understanding how the structure behaves in both its temporary and permanent states. Reference should be made to the series of nine illustrative diagrams in **Appendix II – 3**.

23. Hung Hom Station has a high natural ground water level which fluctuates with tidal levels. The level of the ground water within the D-walls has to be lowered by pumping during the progressive excavation and construction stages. On completion, the pumping is deactivated and the ground water is allowed to regain equilibrium; a process known as "recharging".

24. The head of water i.e. the difference in level between the water externally to the level below the NSL slab creates a very significant uplift pressure not only on the NSL slab but also on the entire tunnel structure. The overall mass of the completed tunnel structure must be greater than the uplift pressure to prevent flotation of the tunnel (i.e. buoyancy).

25. To obtain an approximate simplistic understanding of the deflection characteristics of the D-walls at these various construction stages, a geotechnical engineer in RPS carried out a simplistic computer analysis under my direction **[**see **Appendix VIII]**. This analysis relied on the soil parameters and ground water data which were used by Atkins in, for example, their Design Report TWD-004B3[31] **[B12/9014–9021]**. The results indicate simplistically that a maximum D-wall final deflection of approximately 82mm, at the NSL slab, is anticipated. This gives a good indication of how the wall deflects during the key construction stages but is very conservative, however, because this simple model does not take into account factors such as the interaction of the D-walls with the very stiff slabs which, if implemented, would significantly reduce the magnitude of the deflections. In addition, the soil parameters used in this type of design are usually conservative because of ground uncertainties resulting, therefore, in higher predicted deflections than one might experience in practice.

<u>Structural behaviour</u>

26. I am concerned only with Areas B and C and C in particular, which is where malpractice was alleged[32] and where unauthorized construction changes have been

[31] Atkins' Design Report TWD-004B3 **[B12/9014 – B9021]**

[32] §§30 and 33, WS of Jason Poon **[D1/19-20]**, §§5, 9 and 10, PS of Jason Poon dated 10 July 2018 **[D1/765.2-765.4]**; §6, PS of Jason Poon dated 31 July 2018 **[D1/831.2-831.3]** ; See also Table of photographs from Chinat **[A1/415-421]**

made[33]. Also, in order to simplify the structure I am deliberately, at this stage, ignoring the following facts:

- The lower NSL slab is also supported by barrettes (individual load-bearing D-wall panels).
- The upper EWL slab in its permanent state receives support via columns and walls built-off the lower NSL slab.
- Escalator, stair and lift openings etc. have been formed in the upper EWL slab.
- Air ducts punctuate the D-wall panels.
- Columns supporting the existing Hung Hom station podium and roof had their loads transferred to the new EWL slab, using underpinning techniques, thus allowing the supporting piling to be disconnected and removed.
- Existing columns external to the new tunnel were also underpinned.

27. In its completed state [**Appendix II–4]** the D-walls of the box structure are trying to move inwards because of the external soil and water pressures but are prevented from so doing by the upper EWL and lower NSL slabs acting as struts in compression.

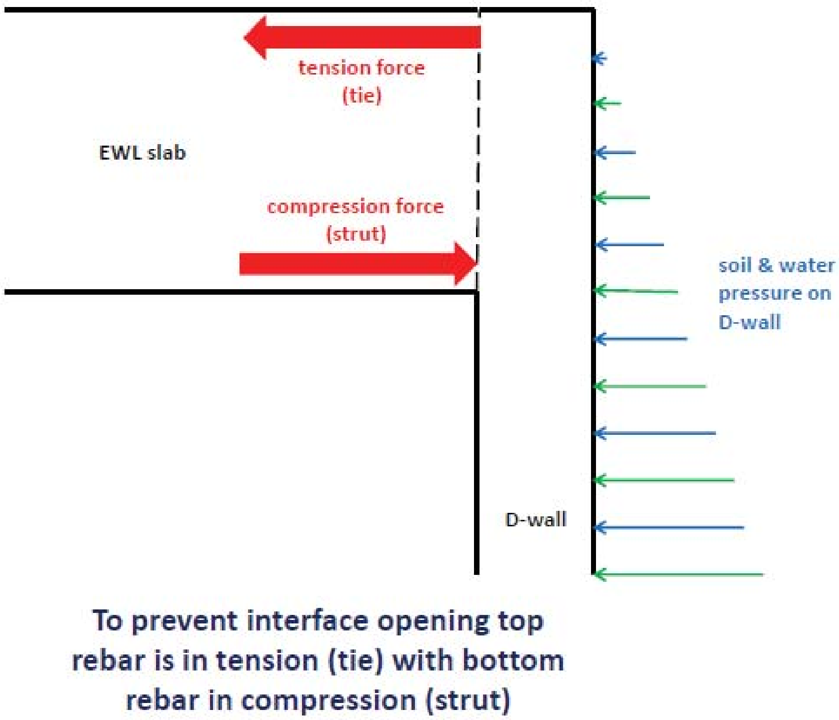
28. In so doing the top of the D-wall at the interface with the upper EWL slab **[Appendix II–5]**, if unrestrained, is always pushing inwards against the bottom of the D-wall and, at the same time, is trying to pull out from the top of the EWL slab because of the inward curvature of the D-wall between the EWL and NSL slabs. To prevent that happening, it is necessary to provide a tie-in at the top.

29. Locking-up the connection between the EWL slab and the top of the D-wall in this way produces a "bending moment" (i.e. a force multiplied by a distance which causes bending to occur). An example of a bending moment, by way of explanation, is shown in **Appendix II–6**. The magnitude of the subject bending moment is influenced by the relative stiffness of the rigid EWL slab and the more flexible D-wall. This locked-up joint then significantly reduces the deflection of the D-walls

[33] §68, WS of Lok Pui Fai **[H7/2205-2206]**

below the EWL slab, thus minimizing ground movements which have the potential to damage adjacent buildings and structures.

30. The diagram illustrating this concept in **Appendix II–5** is copied below to highlight the very important point that, at the interface between the EWL slab and the D-wall, the bottom of the slab is always in compression with the joint trying to close, whereas the top is always in tension trying to open.



31. The resulting simplistic deflection and bending moment visualization diagrams for the completed permanent state (which do not include the influence of the lower NSL slab) are shown in **Appendix II–7**.

32. The top tension steel rebar in the EWL slab, in conjunction with the non-reinforced concrete section across the shear key, also transfers vertical load from the EWL slab into the "H" panels of the D-walls **[Appendix II–8]** and hence into the bedrock[34] **[J6/4065-4495][J6/4497-4498]**.

[34] Atkins' letter dated 26 November 2018 (with Appendices A-D) and letter dated 30 November 2018 **[J6/4065-4495][J6/4497-4498]**

Structural reserve capacity

33. As previously stated, both the EWL slab and NSL slab have been sized not only for structural performance but to provide sufficient mass to counteract buoyancy due to the significant ground water pressure which tries to cause the box tunnel to float upwards. Generally when a section is deepened to provide more mass the steel rebar is less highly stressed. That intent is confirmed in the second paragraph of the Atkins' presentation to the Buildings Department[35] **[J4/3330]** where it is stated that the EWL slab was designed as a 3m deep slab and "*not a thinner slab with extra concrete*". It should also be highlighted that the dead load accounts for some 90% of the total load and live load in service for only 10%[36]**[J4/3330]**.

34. It is also to be noted that when the original coupler connection detail between the top of the EWL slab at the east D-wall was first changed from the approved detail (to be explained later), additional mid-span rebar was added to the EWL slab to compensate for any notional loss of fixity[37] **[B12/8999]**.

35. It is important, in case the structural behaviour would be in any way compromised by proved malpractice or inferior workmanship and/or materials, to estimate the reserve capacity of the structural elements. Elsewhere reserve capacity is referred to as "utilization" and is expressed as a percentage where "utilization" is the ratio obtained by dividing the actual maximum force in a D-wall panel by the capacity of the panel. This is based on ultimate load and strength analysis.

36. Atkins carried out an initial utilization analysis on the EWL slab to east D-wall connection at each wall panel location based on the modified through bar detail[38] **[B17/24479-24503]**. The results are summarized by Ove Arup & Partners Hong Kong Ltd ("**Arup**") engaged by MTRCL as peaking at approximately 70% with an average value of approximately 60%[39] **[G13/10718]**.

37. Subsequent to their Stage 1 Report, Arup then carried out a holistic study to assess the as-built structures and reviewed the utilizations of all the main

[35] Atkins' presentation **[J4/3323+,** see in particular **J4/3330]**
[36] Atkins' presentation **[J4/3330]**
[37] Section 1.3.5, Atkins' Design Report TWD-004B3 **[B12/8999]**
[38] Drawings showing level of strength utilisation **[B17/24479-24503]**
[39] Paragraph 5, Holistic Study to verify as constructed condition (Stage 1: EWL slab/diaphragm wall connection), Rev A (13 October 2018) **[G13/10718]**

elements[40]**[B19/25114-25446]**. In summary they concluded the following utilization values:

- EWL slab at east D-wall connection: generally below 50% and peaking at 60%. **[B19/25128]**
- EWL slab mid-span: generally 60% - 70% and peaking at above 80%.**[B19/25128]**
- D-walls: generally peaking at around 90% at the junction with the EWL slab and decreasing considerably with depth but reaching a secondary peak at the lower NSL slab junction. **[B19/25129]**

38. Arup concluded that the EWL slab has significant reserve capacity, which is partly due to the fact that in its permanent state it relies on intermediate support whereas in the temporary condition it was full-spanning between the D-walls. By inference, however, Arup highlighted that the critical elements in terms of structural capacity are the D-walls particularly at the connection with the EWL slab.

39. Arup therefore carried out their own spot-check review of the D-walls in particular[41] [42] **[B19/25447-25476][B20/26004-26048]**. They initially reviewed the inclinometer readings taken during the construction which yielded results much lower than those predicted in the Atkins analysis. They then carried out a modified Plaxis (i.e. a soil-structure interaction software package) retro-analysis using more realistic soil properties and loading conditions which resulted in lower D-wall utilization values and hence even higher reserve capacity throughout.

40. Table 8 in their report (Rev B, 27 November 2018) summarizes the D-wall utilization values[43] **[B20/26012]**. Based on Serviceability Limit State ("SLS"), which tends to govern heavily reinforced concrete sections, Arup computed values of 61% and 76% at the EWL and NSL slab connections respectively.

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[40] Arup's assessment report on Holistic Study to verify as constructed condition-REP/0002, Rev A (9 November 2018) [**B19/25114-25446]**

[41] Arup's design spot checks for diaphragm walls - Plaxis analysis-REP/003, Rev A (14 November 2018) [**B19/25447-25476]**, Rev B (27 November 2018)**[B20/26004-26048]**

[42] Arup's design spot checks for diaphragm walls - Plaxis analysis-REP/003, Rev A (14 November 2018)[**B19/25447-25476]**, Rev B (27 November 2018)**[B20/26004-26048]**

[43] Table 8. Arup's design spot checks for diaphragm walls - Plaxis analysis-REP/003, Rev B (27 November 2018)**[B20/26012]**

41. In respect of the slabs, Table 6 shows that the EWL slab at the D-wall connection has a maximum bending moment which is lower by approximately 34% than the Atkins value which enhances its utilisation further[44] **[B20/26011]**. By comparison, the maximum bending moment in the lower NSL slab at the D-wall junction increases by approximately 11% thus slightly reducing its reserve capacity.

Code requirements

42. The relevant Code of Practice[45] **[H8/2818-3015]** to which the project was designed contains two basic requirements which I understand are intended to cater for the potential for seismic activity. These are a) ductility and b) the continuity of the slab tensile reinforcement across the D-wall end supports. The ductility requirements as set out in Section 9.9 relate to reinforced concrete frames comprising beams and columns but not specifically to slabs and walls **[H8/2969-2972]**. In terms of rebar continuity across end supports, it is stated at §9.3.1.3 that "…*half the calculated span reinforcement should be anchored into the support*" **[H20/2964]**.

43. An Information Note issued by the Hong Kong government in October 2015 states that "*Based on seismic hazard studies, the seismicity of Hong Kong is classified as 'low to moderate'. The seismic hazard in Hong Kong is much lower than places like Japan, Taiwan and the western USA which lie close to the earth's more seismically active zones along crustal plate boundaries. However, the seismic risk in Hong Kong cannot be regarded as negligible.*"[46] **[A1/695+]**.

44. Accordingly the BD currently does not have any specific design and construction requirements in respect of seismicity[47] [**H20/40457]** but requires compliance with the ductility requirements of the Code, including couplers. In other words, code-compliance is deemed to provide some inherent structural resilience against seismic events. I understand that Hong Kong has been thinking for a long time about including seismic design as part of structural design but that the Code, which was supposed to supplement the intended seismic code, had actually usurped the seismic code. I note, however, that MTRCL do include specific seismic design requirements

[44] Table 6. Arup's design spot checks for diaphragm walls - Plaxis analysis-REP/003, Rev B (27 November 2018)**[B20/26011]**
[45] Code of Practice for Structural Use of Concrete 2004 (Second Edition)**[H8/2818-3015]**
[46] Information Note 08/15 (October 2015)**[A1/695]**
[47] Letter dated 8 November 2018 from DoJ to LL **[H20/40457]**

and loading in their performance specifications[48]**[B20/26392]** as contained in Section 4 (Civil Engineering) of their New Works Design Standard Manual (*see*. Section 4.4.13 on Earthquake Loads)[49].

SLAB TO D-WALL CONNECTIONS

45. Major concerns have arisen over the structural integrity and safety of the structures because of changes which were made to the slab-to-D-wall joints during construction.

As-originally designed and approved connections

46. Different slab to D-wall connections were originally proposed and approved by the Buildings Department (**BD**) and summarized as follows:

- EWL slab at west D-wall

  This was essentially a longitudinal capping beam, 2,000mm deep, constructed integrally with the top of the west D-wall as illustrated by the diagram on the left of the page[50] **[C1/34]**. The soffit of the EWL slab is 1,000mm below the underside of the capping beam.

- EWL slab at east D-wall

  It was generally intended to connect steel starter rebars, for both the top and bottom steel of the EWL slab, to couplers which were cast into the D-walls and subsequently exposed, as illustrated by Figure 5[51] **[G13/10728]**.

- NSL slab at both west and east D-walls

  The detail is similar, for the 2,000mm thick lower slab, as for the EWL slab to east D-wall detail, relying on couplers at both the top and bottom of the NSL slab as illustrated by the diagram on the right of the page (which applies to both the EWL and NSL slabs)[52] **[C1/34]**.

[48] Email dated 16 November 2018 from MTRCL to LL **[B20/26392]**
[49] Section 4 (Civil Engineering) of MTRCL's New Works Design Standard Manual https://kupdf.net/download/nwdsm-section-04-civil-engineering-a5-18-apr-2013-pdf_59f6c6d2e2b6f51c75d9517f_pdf
[50] Letter dated 26 June 2018 from DEVB to LCAL **[C1/34]**
[51] Figure 5, Arup Holistic Study to verify as-constructed condition (Stage 1: EWL Slab/Diaphragm wall connection) REP/0001, Rev A (13 October 2018) **[G13/10738]**
[52] Letter dated 26 June 2018 from DEVB to LCAL **[C1/34]**

As-constructed connections

47. The NSL slab at both sides and the EWL slab at the west D-wall connection were constructed as intended. The EWL slab at the east D-wall connection for various reasons, however, was only adhered to for a small number of panels which, based on the provisional "as-constructed" drawings[53] **[B19/25480-25689][C34/26494-26495]**, are located mainly in Area B between GL14 and GL22. Part of Area C between GL29 and GL 30 is also shown to be constructed as per the original detail.

48. Early on it was found necessary to re-organize the rebar arrangement at the top of the east D-wall connection to facilitate the installation of tremie pipes and other cast-in ducts and to improve buildability[54] **[F35/24262-24265**]. This resulted in the decision to omit the inverted "U" bars in the top of the east D-wall [55][56]**[C27/20806][H11/5542-5543]** (referred to elsewhere as the "first change").

49. As a result of the missing top-of-wall "U" bars, as well as the other difficulties encountered such as the lack of alignment of the embedded D-wall couplers in other areas of the east D-wall, the modified detail further evolved into a generic type of connection (referred to elsewhere as the "second change"), which it is alleged was not formally approved, where:

- Some 450mm to 500mm of the top of the D-wall was cut-down.
- All the layers of short top rebar with their couplers were removed.
- Equal layers of through bars were provided[57] **[C27/20836-20837][C27/20856-20857]**.

50. This is categorized as the "Type 1" connection[58] **[C34/26494-C26495]**. Based on the provisional "as-constructed" drawings, the majority of the Area C slab EWL slab to D-wall connection was constructed to this detail. It is to be noted that in certain D-

[53] Joint statement made by LCAL and MTRCL **[B19/25480-25689] ;** Leighton design proposal drawings showing the as built details (27 August 2018) **[C34/26494-26495]**
[54] §§13-34, WS3 of Jean-Christophe, Jaques-Olivier Gilliard **[F35/24262-24265]**
[55] §§23-24, WS1 of Brett Buckland **[C27/20806]**
[56] §§2.1-2.3, MTRCL's Incident Report on Diaphragm Wall Reinforcement Details of HUH Station dated 27 July 2015 submitted to the BD
[57] §§19-20, WS of Justin Taylor **[C27/20836-20837]**; Diagrams 3 and 4 **[C27/20856-20857]**
[58] Leighton's design proposal drawings providing as built details (27 August 2018) **[C34/26494-C26495]**

wall panels, "U" bars were, for whatever reason, subsequently instructed to be reintroduced[59] **[F35/24265]**.

51. Several minor variations of this generic detail evolved further to address specific locational requirements. For example, over a small length of east D-wall between GL34 and GL37, a detail similar to the west D-wall connection, categorized as "Type 3", is shown to have been constructed. The D-wall has been cut-down to the underside of the EWL slab. The EWL slab and the OTE structure have then been integrally cast over the D-wall and tied into it. Where there are no air ducts punctuating the east D-wall, the "Type 4" connection applies[60] **[C34/26494-C26495]**.

52. Helpful simplified coloured diagrams illustrating the changes from original to as-built can be found in the report prepared by Tony Gee and Partners (Asia) Limited ("**Tony Gee**") (16 September 2018) engaged by LCAL[61] **[H14/35284–H35295]** and more succinctly in Exhibit "JT-3" to the Witness Statement of Justin Taylor[62] **[C27/20855–C20857]**.

53. The Code of Practice at §8.7.1 permits mechanical couplers, or welding, to be used in lieu of lapped rebar[63] **[H8/2946]**. There is no evidence in either of the generic changes, or variations thereof, that there has been any reduction in the area of steel rebar provided and therefore no reduction in the tensile and/or shear capacity of the EWL top-of-slab to D-wall connection. If Tony Gee's evidence is accepted, the steel rebar provision has been enhanced and there is more load capacity in the connection than originally designed for[64] **[H14/35293-34295]**.

54. It should also be highlighted, as previously mentioned, that the cutting down of D-walls is a normal construction event. The top sacrificial part of any D-wall, which is unreinforced and which contains soil contamination and hence weaker concrete, is normally removed by rock-breaking equipment. Any further trimming down of the wall should be done by hand to avoid the risk of cracking the wall. The

[59] §32, WS3 of Jean-Christophe, Jaques-Olivier Gilliard **[F35/24265]**
[60] Leighton's design proposal drawings providing as built details (27 August 2018) **[C34/26494-C26495]**
[61] Section 7, Report by Nick Southward of Tony Gee on Change of Details at Eastern Diaphragm Walls and Slabs (16 September 2018) Rev.0 **[H14/35284–35295]**
[62] Diagrams in Exhibit "JT-3" to WS of Justin Taylor **[C27/20855-20857]**
[63] § 8.7.1, Code of Practice for Structural Use of Concrete 2004 (Second Edition)**[H8/2946]**
[64] Section 9, Report by Tony Gee on Change of Details at Eastern Diaphragm Walls and Slabs (16 September 2018) Rev.0 **[H14/35293–35295]**

photographic records taken during construction confirm this methodology was adopted[65] **[B19/25568+]**.

55. The formation of a "construction joint" at the top of the cut-down D-wall is analogous to forming a normal shear key in a wall or slab (e.g. the vertical shear key specified for the NSL slab and the EWL slab at the east side). Provided the contact interface is roughened and aggregate is exposed there will be no discontinuity across the joint.

BOSA MECHANICAL COUPLERS

56. The couplers, which are an important component of the slab to D-wall connections in respect of their quality and installation, lie at the heart of the allegations of malpractice but one has to ask "What are the advantages, if any, in cutting a threaded bar because, intuitively, it will take time to do it?" I therefore requested that some timed experiments be conducted. One such experiment was carried out at BOSA's factory on Tuesday 6 November 2018, the outputs of which are recorded in **Appendix IV**. It involved screwing a 4 m long Type 2(A) T40 threaded rebar into a BOSA Seisplice Type 2 ductility coupler using non-steel fixers. (Note – a "T"40 can also be referred to as a "Y"40 bar or an "H"40 bar as is the most recent convention).

57. The mechanical couplers specified for the project were "BOSA Seisplice Type 2 ductility couplers" **[H9/4262+]**. The word "ductility" infers the couplers have been designed for cyclical loading as would arise from a seismic event in contrast with the normal non-ductility BOSA Servisplice Type 1 couplers. Unlike non-ductile couplers which can only be used in certain structural zones, the ductility couplers can be used anywhere **[H9/4271]**.

58. The terminology however is misleading because it is not the coupler that exhibits ductility during cyclical seismic loading; rather it is the Type 2 steel rebar, which yields under cyclical stressing, and which gives the "system" its ductility. The Seisplice coupler itself and the threaded end of the rebar is always stronger by design than the rest of the rebar; therefore the rebar will always yield and/or fail

[65] Annex F (site photographs), Joint Statement of LCAL and MTRCL dated 16 November 2018 **[B19/25568+]**

first[66] **[H9/4180+][H10/4790+]**. This is not true of the Servisplice coupler, which has a thinner wall, with its Type 1 threaded rebar.

59. The two types of coupler have different visual appearances in that the Seisplice coupler has a shiny ring at each end whereas the Servispice coupler has a uniform dark appearance **[Appendix IV–3]**. Moreover the Seisplice couplers are supplied with a red protective plastic cap **[Appendix IV–4]** whereas the Servisplice couplers have a blue cap[67].

60. The Type 1 non-ductile bar has its threaded end formed by **[H9/4064]**:

- Machining off the upstand ribs on the steel rebar.
- Rolling the threads (i.e. not cutting threads) so that the diameter of the bar is not reduced.

61. In contrast, the Type 2 rebar undergoes a different process **[H9/4147, 4156-4157]** in that:

- The end of the bar is inserted into a press which squeezes the upstand ribs almost, but not quite, flat. This process is called "crimping" and compresses the bar end thus locking-in beneficial stress; a process otherwise known as "strain hardening" or "work hardening".
- The threads are also rolled so that the diameter of the bar is not reduced.

62. A Type 2 bar is easily identified visually even when screwed fully into a coupler because of the 5mm length of crimped ribs. It was confirmed by BOSA that each ductility coupler can accept either a Type 1 or a Type 2 threaded bar but one should follow the company's requirements under its quality assurance manual and not mix system components[68].

63. The experiment at BOSA's factory recorded a time of some 47 seconds for two non-steel fixers to fully screw a 4m long T40 Type 2 bar into a Seisplice coupler **[Appendix IV–5 to 9]**. One would expect experienced steel fixers on site to take a

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[66] See AC133 Test Report **[H9/4180+]**; Site inspection/audit witness record and inspection report **[H10/4790+]**
[67] Paulino Lim of BOSA **[Day 36/p.89:1-23]**
[68] Paulino Lim **[Day 36/pp.89:24-93:21; 105:5-107:19]**

shorter time in ideal conditions albeit there are factors which can slow down the procedure e.g. imperfect alignment of the couplers and threaded starter bars, rebar congestion and worker fatigue etc.

64. The starter rebars are also supplied as two different types in terms of the threaded length **[Appendix IV–1 and 2]**:

- Type A is referred to as a rebar with "standard" splicing and has a threaded end length of approximately 50% of the coupler length. So when it is fully engaged it occupies one half of the coupler. The threaded end of the rebar it connects with fills the remaining half of the coupler[69] **[B1/426][H9/4088]**.

- Type B is referred to as a rebar with "positional" splicing and is used in situations where the individual steel bars of pre-fabricated rebar cages (which cannot be rotated) are to be joined. The threaded length of a Type B rebar is twice the length of a Type A rebar and therefore is approximately the same length as the coupler. The coupler is screwed onto the full length of the Type B threads; then the two bars are brought into contact; the coupler is counter-rotated so that one half engages the other bar end, leaving the other bar with a half coupler length of thread visible[70] **[B1/427][H9/4089]**.

65. In all cases, however, the threaded rebars do not require any torqueing[71] **[H9/4276]**; i.e. they only need to be screwed hand tight and then given a final part-turn by a pipe wrench to avoid any tendency to work loose **[Appendix IV–10]**. Once connected to a coupler, however, the subsequent lay-up of rebars in a mat precludes any possibility of them working loose.

66. BOSA on request also produced a Thread Strength Calculation Table for a T40 rebar with one to ten threads incrementally engaged with a Seisplice Type 2 ductility coupler[72]**[Appendix V–1]**. The objective was to determine the minimum number of connected threads which would theoretically produce a connection at least as strong as the rebar. The calculated tensile strength of the T40 rebar is

[69] §28.1, WS of Kobe Wong **[B1/426]**; BOSA Technical and Quality Assurance Manual Servisplice Standard Normal Coupler **[H9/4088]**

[70] §28.2, WS of Kobe Wong **[B1/427]**; BOSA Technical and Quality Assurance Manual Servisplice Standard Normal Coupler **[H9/4089]**

[71] BOSA Technical and Quality Assurance Manual Servisplice Standard Normal Coupler **[H9/4276]**; Paulino Lim **[Day 36/pp.86:15-87:10; 104:10-105:3]**

[72] BOSA Thread Calculation Table **[H25/44527.1+]**

defined as its specified tensile stress (i.e. 529MPa) multiplied by its cross sectional area (i.e. 1256.6mm^2) which gives 664.74kN. This is the ultimate load (force) required to break a T40 bar. Before it breaks it will permanently stretch; a process known as "yielding" or becoming "plastic". An online definition states "*Ultimate strength is the stress at which a material fails or breaks. The main difference between yield strength and tensile strength is that yield strength is the minimum stress under which a material deforms permanently, whereas tensile strength describes the maximum stress that a material can handle before breaking.*" BOSA calculated that a minimum of six engaged threads would achieve a tensile strength of 755.87kN which is stronger than a T40 bar by a factor of safety of 1.14. By interpolation an engagement length of 5.5 threads is also stronger than a T40 bar with a factor of safety of 1.05.

67. BOSA in conjunction with a Site Monitoring Team from BD carried out a recent set of tensile tests on 21 November 2018 to validate the calculated thread strengths. They used CASTCO Testing Centre Limited, an independent HOKLAS (Hong Kong Laboratory Accreditation Scheme) accredited laboratory[73] **[H25/44485-44526]**. The BD summary table[74] **[B25/44520]** shows that for:

- 30% engagement the threads failed at 419MPa under an applied load of 526.11kN (compare with 529MPa and 664.74kN).

- 50% engagement the coupler failed (the reason is not given and is not obvious to me but it suggests a deficient coupler) at 630MPa under an applied load of 791.54kN.

- 60%, 70% and 100% engagement the bar broke at an average 687MPa under an average applied load of 863.55kN (compare with 529MPa and 664.74kN).

Failure of the coupler, induced by using larger diameter non-ribbed bars, occurred at 788MPa under an applied load of 990.41kN.

68. BOSA does not have any test results for partially engaged connections which are subjected to lateral shear loading.

[73] BD test documents dated 21 November 2018 **[H25/44485-44526]**
[74] BD test summary table **[H25/44520]**

CIC BAR CUTTING EXPERIMENTS

69. On Thursday 8 November 2018, CIC on request kindly performed experiments in which bars were cut using different equipment to compare the times taken. The outputs are recorded in **Appendix VI**. To summarise:

- A portable shear cutter could only cut a T16 bar as maximum. The resulting bar ends were severely distorted **[Appendix VI– 2 to 4]**.
- A portable electric band hacksaw took on average 47 seconds to cut a T40 rebar. The threads were physically undamaged and could be screwed into a coupler **[Appendix VI–7 to 14]**.
- A hydraulic bending and shearing machine took only some 6 seconds to cut a T40 rebar. The bar end was distorted and the threads were severely damaged, precluding any attempt to insert the bar into a coupler **[Appendix VI–15 to 18]**.
- A portable electric disc grinder took some 32 seconds to cut a T40 rebar. The threads were physically undamaged and could be screwed into a coupler. There was evidence of heat scorching on the bar end **[Appendix VI–19 to 24]**.

INCIDENTS OF DEFECTIVE COUPLERS

70. The incidents of defective bar coupling, including the one which generated the Non Conformance Report NCR 157 where the threaded end of a rebar was cut and photographed[75] **[B6/4121+][C12/8148+]**, are well-documented. Other incidents are, unfortunately, not so well-documented or recorded.

71. Importantly, however, all the evidence that is available (including NCR 157) appears to demonstrate that this type of malpractice was confined to the bottom rebar mat of the EWL slab. Reference in particular should be made to the Witness Statements and corresponding hearing transcripts of Jason Poon and Kobe Wong respectively[76] **[D1/227-8][B1/437–442]**.

72. It has not been confirmed if these bottom mat coupler incidents relate to the D-wall interface or the CJ interface or both (as appears to have been the position with NCR

[75] NCR 157 **[B6/4121+][C12/8148+]**

[76] "PCHJ-5", WS of Jason Poon **[D1/227-8]**; §§66-88, WS of Kobe Wong **[B1/437–442]**; Jason Poon **[Day 7/pp.120-122]** and Kobe Wong **[Day 29/p.154:2 -10]**

157). However the inference generally is that the D-wall connections are in question.

PHOTOGRAPHIC EVIDENCE OF BAR CUTTING BY CHINAT

73. Chinat has produced, as part of its evidence, a photograph **[D1/228]** taken by Mr Jason Poon purporting to show the threads of a what is reasonably assumed to be a T40 rebar being cut by an electric band hacksaw. The location of the incident has been identified and is likely to be Area C1-4[77]. The photograph shows that the lower layers of rebar in the bottom mat have been completed and work is progressing on the top layer.

74. **Appendix IX** contains an enlarged copy of the Chinat photograph. This shows the T40 bar as having 13 threads. The top of the band hacksaw[78] is seen to be below the level of the rebar.

OPENING-UP OF THE WORKS

75. As part of the implementation of the MTRCL Holistic Proposal (Rev B)[79]**[B20/26099–26136]** opening-up of the works commenced on 10 December 2018 and continues on a weekly rolling programme basis with an anticipated completion date of early to mid-March 2019. There are two basic purposes:

    1. Validation of the as-constructed drawings at the EWL slab to top of east D-wall connection.

    2. Investigation of the workmanship and efficacy of the rebar connections formed by mechanical couplers.

76. I witnessed the opening up on two occasions (Monday, 17 December 2018 and Wednesday, 19 December 2018). Some of the photographic records I took during those two visits are contained in **Appendix X**[80].

77. The two EWL soffit openings witnessed and recorded during my first visit on 17 December 2018 revealed a total of five bottom layer T40 rebars and couplers. Two coupled connections at the E46 opening, i.e. the one in the center and the one at the

[77] Jason Poon **[Day 7/pp.117:6-117:11; 120:16-123:15]**; Gabriel So **[Day 19/pp.2:15-5:7]**
[78] The band hacksaw used is similar to the one shown in **Appendix VI-5**.
[79] Revised Holistic Proposal (Rev B) **[B20/26099–26136]**
[80] See also **[A1/Items 50-51]**

right hand side **[Appendix X-3 and 5]**, had been ultrasonically tested for thread engagement. The third coupled connection was subsequently similarly tested. The three results recorded were 34.91mm, 29.65mm and 34.32mm **[OU/48]**.

78. During my second visit on 19 December 2018, I witnessed and recorded three further openings. One was a EWL top-of-slab opening at E44 **[Appendix X–11-14]**. The left hand coupled connection showed 9–10 threads exposed on the left hand side rebar. Both connections were ultrasonically tested with thread engagement being 31.61mm and 6.22mm **[OU/48]**. The 6.22mm engagement relates to the rebar with 9–10 threads exposed.

79. Inside the air duct at E72, two openings were implemented, one on each side of the underlying east D-wall **[Appendix X–15-17]**. One of the OTE side couplers were used as shown and was set outside the line of the D-wall face to facilitate a construction joint. On the other side of the D-wall, through bars were used.

80. Opening-up records are being made available to me **[Bundle OU]** and are being tracked and reviewed on a daily basis while I am in the UK.

<u>MISCELLANEOUS DEFECTS</u>

81. In addition to the two major issues of alleged coupler malpractice and the amended top-of-wall construction detail, other non-structural secondary defects have been identified or alleged during the course of inspection and investigation.

<u>Concrete spalling</u>

82. “Spalling” is generally classified as the delamination of a thin cement-rich layer on the surface of the slab. When discovered on the soffit of the EWL slab, the entire bottom surface was “tap-tested” and defective areas removed resulting in widespread areas which are now in the process of being remediated. **[Appendix III–2 & 3]**. It was noted that the removal of the delaminated layer has, in places, exposed the light steel mesh which is required to achieve the specified fire resistance period.

<u>Honeycombing</u>

83. Localized "honeycombing", otherwise known as "voiding", is likewise present **[Appendix III–4 to 7]** on the EWL slab soffit[81] **[**see also **H13/7499-7515]**. Where discovered, the defective concrete has been removed and a repair strategy is being formulated **[H13/7519+]**. It was noted during my inspection that the honeycombing appeared to occur only in areas where temporary openings in the EWL slab, in locations where existing underpinning and column removal has been implemented, were infilled using couplers. The limited length of the opening resulted in double the number of T40 steel starter rebars in each layer because of the rebar overlaps.

<u>Incorrectly placed shear links</u>

84. The over-dense bottom steel rebar in localized slab infill areas (to be explained later) has consequently resulted in some steel shear links in the EWL slab being misaligned and not connecting properly with the bottom steel rebar. These were discovered when the honeycomb concrete was removed **[B20/26387-26390]**. It is understood that further investigation is ongoing.

<u>Partial gap at column and wall tops</u>

85. Some of the reinforced concrete columns and walls which were constructed off the lower NSL slab have been found to have partial gaps at their interface with the EWL slab soffit **[Appendix III–3 and 4 to 7]**.

<u>D-wall water ingress</u>

86. In spite of allegations of water ingress through the D-walls, only one area of dampness was observed at a joint between two panels on the east D-wall above the lower NSL slab level **[Appendix III–11]**. It is understood that any instances of through-seepage were successfully remediated by pressure grouting in the normal manner.

<u>Alleged non-compliant concrete fill in Area A</u>

87. MTRCL when asked about this allegation at the site meeting on 5 November 2018 explained it by reference to their presentation **[A1/30].** Normal density **c**oncrete backfill was specified to a certain level for reasons of anti-flotation, thereafter lightweight concrete was specified on top of the normal concrete. LCAL used some

[81] See for example, MTRCL NCR 258-260 **[H13/7499-7515]**

recycled dense concrete at the very bottom and topped it up to the requisite level using normal dense concrete. This was an approved cost saving method[82] **[G13/10331]**[83] **[G6/5418-5419]**[84] **[G14/11332-11334]**. It did not alter the structural dead weight and therefore did not compromise the resistance to flotation.

Load test proposal

88. In response to the RDO's request of 31 May 2018 **[H8/3017]**, MTRCL commissioned CM Wong and Associates Limited ("**CM Wong**") on 6 June 2018 to develop a live load test proposal[85] **[B8/5400]**. This was issued on 22 June 2018[86] **[H8/3019] [B8/5397+]** and advocated using the lower NSL slab as the anchorage for jacking the EWL slab downwards; a concept which was fundamentally flawed in that it was highly probable that the lower, thinner NSL slab would suffer irreparable damage, if not failure. However the methodology had to be significantly revised, when the amendments to the EWL slab to east D-wall connection became known and it was realized that there was only a relatively short length of east D-wall which had been constructed as originally intended[87] **[B16/13611.1-13611.31]**. A second undated report was then issued on 30 August 2018 **[H8/3390+]** which proposed using live trains loaded with ballast as proof load **[H8/3402]**. The load test has been held in abeyance until the MTRCL holistic recommendations have been implemented[88] **[B20/26125]**.

[82] RDO-MTRCL Coordination Meeting dated 23 August 2018 paragraph 6.1 (vi)**[G13/10331]**
[83] MTRCL letter to RDO-dated 30 August 2018, paragraphs 4 and 5**[G6/5418-5419]**
[84] MTRCL letter to RDO-dated 26 October 2018 enclosing LCAL response**[G14/11332-11334]**
[85] Paragraph 1.1, CM Wong's Safety Test Outline Proposal (second issue 22 June 2018)**[B8/5400]**
[86] CM Wong's Safety Test Outline Proposal (second issue 22 June 2018) **[B8/5387+]**
[87] Communication exchanged between CM Wong and MTRCL between 25 July 2018 and 1 August 2018 **[B16/13611.1-13611.31]**
[88] Paragraph 7.1, MTRCL Holistic Proposal for verification & Assurance of As-constructed conditions and workmanship quality of the Hung Hom Station Extension, Rev B (4 December 2018) **[B20/26125]**

OPINION

EWL slab integrity if couplers are defective at east D-wall connection

89. The following summary facts inform my opinion:

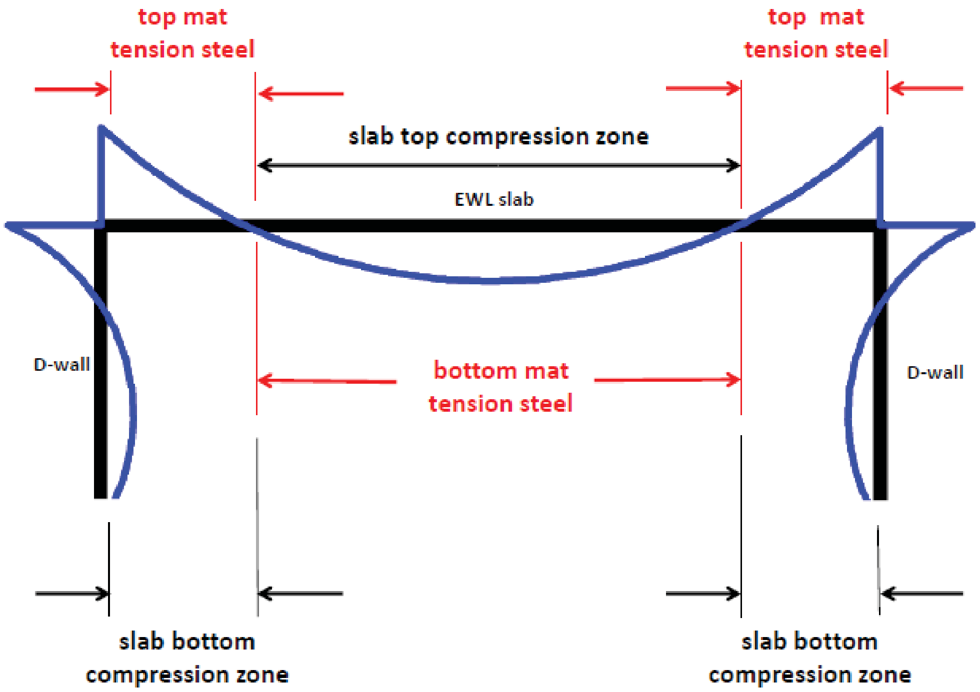
1. There is no requirement for the structures to be specifically designed for seismicity provided the design is code-compliant in respect of the ductility and bottom steel continuity clauses.

2. The geometry of the connection between the EWL slab and the east D-wall, however, precludes any ductility. The structural "plastic" deformation which might occur during seismic activity will develop lower down the D-wall. Ductile-grade couplers are not therefore required where used in the EWL slab to D-wall joint.

3. To satisfy code requirements, part of the bottom steel requires to be continued into the east D-wall. This should be equivalent to 50% of the EWL slab top tension steel (at the D-wall connection). The approved design was therefore conservative in that all four layers of bottom steel were continued through into the D-wall when 50% would have sufficed.

4. The bottom of the EWL slab at each D-wall is always in compression.

5. The flexural strength of the EWL slab is provided by the bottom mat tension steel, which is at a maximum at mid-span and which tails off towards the D-walls, and the top mat tension steel at the east and west D-wall connection. This is illustrated by the Bending Moment diagram ("blue" line) contained in **Appendix II-7** which I have annotated further and copied below for ease of reference. The top middle zone of the EWL slab is in compression, as are the bottom side zones adjacent to the east and west D-walls. Refer also again to the diagram in **Appendix II–5** which is copied at paragraph 30.



6. Shear resistance across the EWL slab to east D-wall interface is provided by the top tensile steel with a small contribution from the non-reinforced shear key.

7. Structural utilization throughout is relatively low meaning that there is adequate reserve capacity in the EWL slab and its east D-wall connections.

90. I therefore conclude that the couplers in the bottom mat of rebar at the EWL slab and D-wall interface will never be in tension (i.e. the threaded rebar will never try to pull out of the coupler). The only reason bottom mat couplers (which are always in compression) are required for the EWL slab is for code-compliance.

91. It follows therefore that for the EWL slab to function structurally and safely, no bottom couplers are required i.e. they could all be defective. It also follows that to be code-compliant, up to 50% of the coupled connections could be defective.

92. Any dowel bars inserted in lieu of defective coupled connections in the bottom mat zone of the EWL slab are likewise only ever acting in compression and are not required by calculation to take shear load.

EWL slab integrity if couplers are defective at construction joint (CJ) locations

93. The top and bottom rebar of the EWL slab in the longitudinal direction is, generally acting as distribution steel and is transferring shear across the CJ interface and locking the joint to prevent any differential vertical movement between the adjoining slab sections. In areas where there are escalator and stair openings etc. in the EWL slab, the longitudinal steel is, in addition, distributing load and providing flexural capacity.

94. The EWL was constructed on an alternate bay infill basis i.e. construct bay one; then bay three; then infill bay two. Bays one and three each had a stop-end vertical shutter to which were attached the couplers. When the shutters were stripped, the longitudinal threaded starter bars were then screwed into the exposed couplers. Unlike the D-walls where the couplers can become damaged or misaligned, the way in which the construction joints were formed means that there should have been minimal risk of malpractice occurring.

95. The CJs have not attracted much adverse attention or allegation of malpractice except for a reference to three non-coupled starter bars (out of five or six) in the lower rebar layer of the top mat of a CJ, in either Area C1 Bay 5 or Area C3 Bay 3, which were not rectified[89]**[B1/455-456]**. This decision was taken because a concrete pour was progressing and only the top accessible two or three uncoupled bars could be connected.

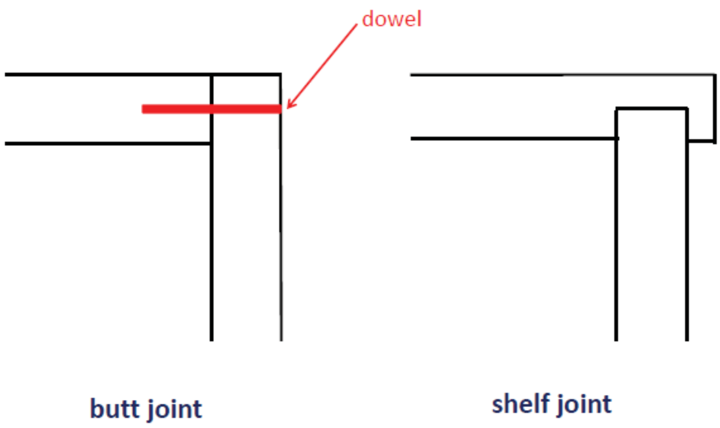
96. The incident which generated Non-Conformance Report (NCR) 157 [90]**[B6/4121+],** which was probably located in Area C3 Bay 2, occurred at the junction of the east D-wall and a CJ [91] **[B1/454][B6/4126]**. These are always areas of heavy rebar congestion because of the transverse and longitudinal layers of rebar. Three rebars were uncoupled and two were not fully connected and/or had the threaded ends cut[92]**[B1/452]**. The five defective connections were remediated, allowing the NCR to be formally closed-out[93]**[B1/453][B1/4127]**.

[89] §§34-37, WS of Andy Wong **[B1/455-456]**
[90] NCR 157 **[B6/4121+]**
[91] §29, WS of Andy Wong **[B1/454]**; see also **[B6/4126]**
[92] §18, WS of Andy Wong **[B1/452]**
[93] §23, WS of Andy Wong **[B1/453]**; NCR 157 **[B1/4127]**

97. The photographic records [e.g. **B19/25618]** generally speaking give no cause for concern but even if some coupled rebar joints were defective, as per the three recorded non-rectified rebars[94]**[B1/455],** there is so much shear over-capacity that the overall integrity of the CJ is not compromised.

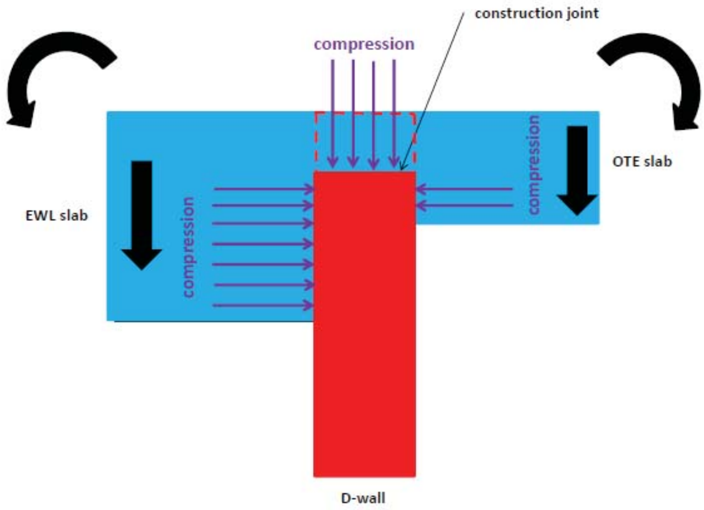
Amended EWL slab to east D-wall connection

98. In my opinion the amended detail, as represented by the "first change" and as subsequently developed to represent the "second change", is superior to the original fully coupled joint from both a structural and buildability perspective. More steel is provided across the top of the D-wall than originally intended. By way of analogy the two diagrams below show two different types of joint encountered when assembling e.g. IKEA flat-pack furniture.



99. Intuitively, and from experience, the shelf joint is superior to the butt joint in terms of its ease of construction and rigidity. The original design is analogous to the butt joint whereas the amended detail is analogous to the shelf joint. The schematic diagram below of the amended connection detail illustrates how the trimmed-down D-wall is encapsulated and "clamped" by the EWL slab bending away in one direction, the OTE bending away in the opposite direction, and the self-weight of the integral "block" of reinforced concrete (coloured in blue) which bears down on the top-of-wall construction interface. The "block" is prevented from splitting above the D-wall by the embedded tension rebar. In my opinion the "clamping" action compensates for the lack of "U" bars in the top of the D-wall.

[94] §34, WS of Andy Wong **[B1/455]**



Schematic of amended EWL slab to east D-wall to OTE connection

100. The internal stresses at the top of wall construction joint are all of a compressive nature. The diagram illustrates why no tension or shear can occur at the interface. Any tendency for a shear force to develop across the interface would be resisted by the "clamping" action of the EWL and OTE slabs which bear against the D-wall.

101. Atkins [95] **[J6/4557-4567]** subsequently produced design calculations for the first and second changes which demonstrate:

- For the first change the EWL slab-to-D-wall connection is so rigid that any rotation caused by slab deflection would create a horizontal crack in the D-wall just below the joint i.e.at the EWL slab soffit as shown in the diagram **[J6/4557]**. In this event the D-wall is more than adequately reinforced.

- The D-wall is also adequately reinforced to prevent the potential for an inclined crack to form within the D-wall, running from the internal face at the EWL slab soffit to the external face at the OTE slab soffit **[J6/4558][J6/4562]**.

[95] Atkins' calculations for internal stresses at the construction joint (cut-down wall top interface) for the first and second changes **[J6/4557-4567]**

- Notwithstanding, some 15% more rebar was inserted into the bottom mid-span of the EWL slab to re-distribute bending moment at the D-wall support to the mid-span in order to enhance the connection capacity as per the as-constructed first change detail.

- The actual shear stress at the interface (were it to occur) is 0.359 N/mm² compared with an allowable 0.65 N/mm² **[J6/4559]**.

- The integral (monolithic) nature of the amended connection means that the "*structure will act as a 'CAP' system and the horizontal interface shear force will be resisted by the bearing of the D-wall connected with the OTE structure and transfer load to the diaphragm wall.*"**[J6/4559]**. This concurs with the explanation given in paragraphs 98 to 100.

<u>**D-walls**</u>

102. The Code of Practice for Foundations 2004 **[C35/26577-26640]** defines a D-wall thus: "*A diaphragm wall may be used as a temporary lateral support wall for deep excavation or the permanent wall of a basement, or it may be designed for both temporary and permanent uses. It may also be used to support vertical loads*[96] **[C35/26623]**." As such, in my opinion, the main function of a D-wall is to resist external soil and water pressures and to prevent the ingress of water. A secondary function is that of acting as a foundation. In the subject case this is illustrated by the fact that there are "hit" and "miss" panels particularly in the east D-wall. Both fulfil the primary function but only the "hit" panels act as foundations.

103. Of note is the fact that the BD appears to make the same distinction[97] **[e.g.C1/68-147]** where different headings and required conditions are stated for a) "Foundation Works (Load Bearing Diaphragm Wall and Barrette)" **[C1/86+]**; b) "Diaphragm Wall Works "**[C1/89+]** and c) "Basement Works" **[C1/94+]**.

104. The supervision, inspection and sign-off records for the D-walls[98] appear to have been of high quality as evidenced by the generally high tolerance levels achieved with coupler placement. Not many couplers appear to have been misaligned or off-the-level at depth which demonstrates a reasonably high level of accuracy.

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[96] §6.2, Code of Practice for Foundations 2004 **[C35/26623]**
[97] Appendix G, LCAL's letter dated 26 June 2018 to Works Branch of DevB **[C1/68-147]**
[98] Panel records **[F17/11206-F23/16526]** and Coupler records **[F23/16527-F33/16733]**

105. There is no evidence of any structural or serviceability problems with the D-walls. The only instance of dampness **[Appendix III–11]** is well within the specified tolerance level.

NSL slab

106. The thinner 2,000mm deep lower level NSL slab acts like the upper thicker EWL slab but in reverse in that it tries to bend upwards. Although it was cast on the excavated, prepared ground it required to be designed as a normal suspended slab carrying its own dead load during the temporary construction stage. It is currently resisting the very significant uplift pressure caused by the high external ground water level relative to the level of the internal ground water at the underside of the NSL slab.

107. In this case the top of the slab is in compression so the upper couplers at the D-walls are not required structurally although they do provide significant shear enhancement. Some rebar, however, is required to continue through into the D-walls to satisfy code requirements (similar to the bottom steel of the EWL slab). The bottom coupled connections are critical in terms of the flexure and shear capacity of the NSL slab. It should be noted that the barrettes improve the structural performance of the slab. There is no evidence of any distress in the NSL slab and no reported problems.

Couplers

108. The photographs submitted by Mr Jason Poon **[D1/227-8]** require some basic forensic analysis. Bearing-in-mind that only T40 bars were used in the EWL slab, the enlarged version **[Appendix IX]** showing 12 – 13 threads means that a T40 Type B bar has been cut through because:

- Only a T50 Type A bar has 13 threads as testified by Mr Paulino Lim[99]
- A T40 Type A bar has 11 threads
- A T40 Type B bar has 21–22 threads
- The blade of the band hacksaw is below the level of the threaded bar end

[99] Paulino Lim **[Day 36/p.95:4-19]**

The enlargement of the photograph taken by Mr Jason Poon infers that a T40 Type B was, for whatever reason, being shortened which, although contrary to BOSA's quality assurance recommendations, is not in my opinion a practice which compromises safety.

Miscellaneous defects

109. In areas where honeycombing has been removed and the bottom steel has been exposed the rebars have lost bond and therefore some dead load capacity. In my opinion the reserve capacity of the EWL slab will allow for this but it will be necessary to carry out a re-analysis on completion of the repair work.

110. The defective shear links in this area is likewise, in my opinion, not an issue. The links are active over the greater part of their length and the areas are over-reinforced because of the significant rebar lapping. As part of the repair, short shear links can, if required, be retro-installed from the slab soffit.

111. I would not classify these as inferior workmanship issues. Rather it is my opinion that the concrete mix design should have been altered by LCAL and/or their designer, Atkins, to compensate for the highly congested steel reinforcement. For example self-compacting concrete ("**SCC**") with a smaller maximum aggregate size could have been specified.

Load test proposal

112. In my opinion the proposal for a Load Test should, in light of the findings described herein, be abandoned. The east rail track on the EWL slab straddles the underlying D-wall for most, if not all, of its length and therefore the load is being transmitted directly into the foundations. No deflection of note will therefore be detected during a load test.

Performance monitoring

113. To allay public concern it is my opinion that, instead of a load test, the EWL slab should instead:

- have more sensitive instrumentation, e.g. a fibre optic system, installed
- have its structural performance monitored in terms of deflection etc.

114. The public should be told upfront that the EWL slab has been instrumented and that performance monitoring is being carried out but that no significant movement is expected.

<u>MTRCL holistic Proposal [B20/26099-26136]</u>

115. In my opinion the proposal although sensible is an "overkill" in terms of its scope. My main issue is with the nature and extent of the invasive investigation work currently being carried out. In my opinion it is unnecessary, pointless and a waste of time and resources to continue with opening up the EWL slab soffit at the east D-wall. As explained herein, the couplers are always in compression and do not contribute to the structural capacity of the slab. The focus should be switched to the top of the EWL slab to verify the as-constructed drawings. In so doing there is no need to use GPR (Ground Penetrating Radar) as a first pass and then to open up. GPR is a crude instrument in terms of its accuracy, is very slow in terms of preparation time and cannot identify a rebar directly below another one. A normal cover meter will probably be equally, if not more effective in locating the embedded rebar. I am of the opinion therefore that the GPR investigation should be dispensed with and straightforward opening up be progressed.

116. I see no reason, based on the same premise, to investigate the equivalent top interface couplers of the NSL slab (which are also in compression). The inference that the bottom NSL couplers be uncovered and inspected demonstrates a lack of sound engineering and practical knowledge and should also be abandoned. It appears to have gone unnoticed that the NSL slab is subjected to a very significant head of water. Digging down to the bottom of the slab would not only require rebars to be cut out, thus temporarily weakening the slab, but more importantly, would inevitably result in the formation of one or more geysers which would be very difficult to close down.

117. I also see no justifiable reason to invasively investigate the D-walls particularly when good records were kept and good sign-off procedures adhered to.

<u>Experts' meeting</u>

118. A Meeting of Experts was held on Tuesday, 18 December 2018. A jointly signed Memorandum was produced **[Appendix XI]**. Subject to one check by calculation of the internal stresses at the EWL slab to east D-wall construction joint at the top of

the cut-done D-wall, all the issues, on which I have expressed an opinion herein were unanimously agreed. Atkins subsequently carried out the required check and demonstrated there is no issue and that the stresses are within acceptable levels[100] **[J6/4557-4567]**.

<u>Opening-up</u>

119. As of 6 January 2019, the opening-up of the works has yielded no surprises apart from two isolated instances of defective coupled rebars; one in the top and one in the soffit of the EWL slab where it connects with the east D-wall. These were identified by PAUT (Phased Array Ultrasonic Testing) which has been calibrated and site-validated by Professor Albert Kwan of Hong Kong University, in conjunction with AES Destructive and Non-Destructive Testing Limited, and will be commented on later. The pass criterion specified by the HyD in its online results bulletin is a 37mm thread engagement length for a T40 Type A coupled assembly[101] **[G20/15039]**. This, however, is simply a nominal figure taken from BOSA's quality procedures which includes a 3mm tolerance. The BOSA tensile load test results[102] **[Appendix V–1]** give a verified pass criterion of 22mm (5.5 threads x 4mm pitch) as an absolute minimum to achieve the full rebar tension and 24mm (6 threads x 4mm pitch) to give a safety factor of 1.14. It is to be highlighted that these criteria all pertain to coupler assemblies which are in tension.

120. As at 6 January 2019 only one top coupler i.e. at E44 has been found defective based on the BOSA thread strength calculation table (i.e. with less than 22mm engagement). The rebar (reference EWL-E44-TT-T1- 02-C1) was discovered with 9-10 exposed threads and was confirmed by PAUT to have an engagement length of only 6.22mm (1.5 threads)**[OU1/48]**. The threads shown in **Appendix X-13** appear to have been damaged at the time of installation because MTRCL, when asked during the inspection of opened up locations by experts, confirmed that extreme care was taken not to damage threads during opening-up. If so, it might explain why the bar was unable to be screwed into the coupler during construction. This has to be classified as inferior workmanship. It is surprising that apparently both MTRCL

[100] Atkins' calculations for internal stresses at the construction joint (cut-down wall top interface) for the first and second changes **[J6/4557-4567]**
[101] HyD online results bulletin (2 January 2019) **[G20/15039]**(https://www.hyd.gov.hk/en/road and railway projects/scl/result/index.html)
[102] BOSA thread strength calculation table **[H25/44527.1+]**

and LCAL inspectors could miss such a defect particularly when it is in the top layer of the top rebar mat.

121. In my opinion, if this is an isolated incident and there are no adjacent rebars similarly compromised, the coupled joint can be left as is or welded. If welded, the tensile capacity will be slightly reduced. It is also worth highlighting, however, that there are only 10 or so D-wall panels where couplers were retained on the top rebar layer of the EWL slab, so the potential for finding similar defects is small.

122. Also as at 6 January 2019 only one coupler in the EWL slab soffit at the east D-wall out of 16 tested has been found with less than 22mm engagement i.e. at E107. The middle rebar of three at this location has an engaged length of only 9.4mm (2.5 threads)**[OU1/141]**. Again, this has to be classified as sub-standard workmanship. Notwithstanding, if this coupler assembly had been at the top of the EWL slab it would have been in tension and, similar to the E44 defect would need to be reviewed. The coupled rebar, however, is at the bottom of the EWL slab and as emphasised throughout this report is always in compression and the coupler and bar are redundant. The minimal engagement length is irrelevant. The other consideration of code compliance, to achieve seismicity resilience, requires only some 50% of the bottom rebar coupled assemblies to act in tension.

123. One more EWL slab soffit opening completed at E115 has exposed two couplers but these have not yet been tested **[OU1/212** and **216]**. Another proposed location at E97 is to be relocated and two other opened-up locations at E32 and E48 revealed through bars instead of couplers **[OU1/212** and **213]**. The EWL soffit coupler investigation programme is therefore almost complete. Based on the 16 tested to date it is obvious that 15 coupled joints (93.4%) can carry full tension.

124. One thing which is becoming apparent is that full 100% engagement of the threaded bars into the couplers was seldom achieved, if at all. Although this is of no structural relevance, for reasons explained herein, it suggests there were site factors, which in my opinion may not constitute poor workmanship, which prevented the rebars from being fully screwed into the couplers. Another perceived anomaly is that some results give engagements in excess of 44mm (half the coupler length). This can be explained because, if a coupler is not fully screwed onto the D-wall threaded "L" bar prior to embedment, or if a coupler has had to be replaced

post-excavation and has not been fully engaged, a greater internal gap results between the ends of the two connected rebars 44mm can therefore be exceeded.

125. It is also significant to note that, based on HyD's acceptance criterion of a 37mm thread engagement, none of the "failed" rebar threaded ends appear to have been cut. By way of explanation one has to add the exposed thread length (external to the coupler), where one thread equals 4mm, to the PAUT measured engagement length to obtain the total threaded length. As of 6 January 2019 all the "failed" rebars achieve a combined threaded length of at least 37mm.

SUMMARY OPINION ON STRUCTURAL INTEGRITY AND SAFETY

126. 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.