Shatin to Central Link

Verification & Assurance of As-constructed Conditions

and Quality of Workmanship

of the Hung Hom Station Extension

(East West Line Platform Slab,

North South Line Platform Slab

and the Connecting Diaphragm Walls)

Engineering Expert Report

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Introduction

1. The Shatin to Central Link ("**SCL**"), with a total length of seventeen (17) km, consists of two sections: (1) Tai Wai to Hung Hom Section: an extension of the Ma On Shan Line from Tai Wai via Southeast Kowloon to Hung Hom where it will join the West Rail Line; and (2) Hung Hom to Admiralty Section: an extension of the East Rail Line from Hung Hom across the Victoria Harbour to Wan Chai North and Admiralty.

2. The Hung Hom Station ("**HUH**") Extension is an underground station being constructed under Contract Number 1112 - Hung Hom Station and Stabling Sidings ("**the Contract**") of the MTR Corporation Limited ("**MTRCL**"). Works under the Contract mainly comprise construction of platforms and diaphragm walls ("**D-Walls**") for the Tai Wai to Hung Hom Section and Hung Hom to Admiralty Section, as well as stabling sidings.

3. The designer of the works of the Contract is Atkins China Limited ("**Atkins**"). The main contractor is Leighton Contractors (Asia) Limited ("**Leighton**"), and construction of the works was commenced in March 2013.

4. Within the HUH Extension, there are two D-Walls running in the north-south direction, i.e. Diaphragm Wall (East) ["**D-Wall (East)**"] and Diaphragm Wall (West) ["**D-Wall (West)**"]. Two platform slabs, i.e. the East West Line ("**EWL**") Platform Slab ("**EWL Slab**") and the North South Line ("**NSL**") Platform Slab ("**NSL Slab**"), are supported by the D-Walls in the underground, with the EWL Slab atop the NSL Slab. Underneath the NSL Slab are *in-situ* geologic materials.

5. The as-constructed conditions and quality of workmanship regarding the construction of the platform slabs of the HUH Extension of the SCL were revealed by the media, indicating that: (1) some of the threaded section of reinforcement steel bars connecting the D-Walls and the platform slabs might have been cut short; (2) some of the reinforcement steel bars connecting the platform slabs to the D-Walls might not have been fully engaged with the couplers being used to connect them; (3) the reinforcement connection details between the EWL Slab and D-Wall (East) might

have been modified without prior approval; and (4) the as-constructed connection details between the EWL Slab and D-Wall (East) might not follow the modified details proposed by Leighton.

6. A Commission of Inquiry ("**the Commission**") was appointed by the Chief Executive in Council on 10th July 2018. Mr. Michael J. Hartmann, former Non-Permanent Judge of the Court of Final Appeal, has been appointed as Chairman and Commissioner of the Commission, and Professor Peter G. Hansford, Professor of Construction and Infrastructure Policy at University College London, has been appointed as Commissioner. The Commission is looking into the facts and circumstances surrounding the steel reinforcement fixing works in respect of the construction works of the D-Walls and platform slabs at the HUH Extension of the SCL.

7. I was nominated by China Technology Corporation Limited ("**China Technology**") to serve on an expert panel ("**Expert Panel**") to provide technical support to the Commission. Other members of the Expert Panel include Professor Don McQuillan of RPS and Royal Academy of Engineering Visiting Professor of Engineering Design at Queens' University Belfast as the Chair (nominated by the Commission), Dr. Mike Glover of Ove Arup & Partners (nominated by the MTRCL), Mr. Colin Wade of Ove Arup & Partners (nominated by the MTRCL), Mr. Nick Southward of Tony Gee and Partners (Asia) Limited (nominated by Leighton), and Ir Professor Francis T.K. Au of The University of Hong Kong (nominated by the HKSARG).

8. My opinions on the as-constructed conditions and quality of workmanship regarding the construction of the platform slabs of the HUH Extension of the SCL are presented in this Engineering Expert Report.

CONTENTS OF MY ENGINEERING EXPERT REPORT

9. The contents of my Engineering Expert Report include:

(a) My professional qualifications and experience that qualify me as the engineering expert for the investigation;

(b) The information that I used to form my expert opinions;

(c) My expert opinions on relevant engineering issues; and

(d) My declaration of duty to the Commission.

MY PROFESSIONAL QUALIFICATIONS AND EXPERIENCE

10. I am an Associate Professor of the Department of Civil Engineering of The University of Hong Kong. Moreover, I am also an Adjunct Professor of the College of Mining Engineering, Taiyuan University of Technology (太原理工大學) under the 100 International Talents Scheme of Shanxi Province, Peoples' Republic of China.

11. I received my BSc(Eng)(Hon) degree in civil engineering with First Class Honors from The University of Hong Kong in 1982. In 1984, I pursued my postgraduate study on a Rotary Foundation International Graduate Scholarship at the University of California, Berkeley, U.S.A. where I received my MS and PhD degrees in geotechnical engineering in 1985 and 1990, respectively.

12. I am a Registered Professional Engineer in Civil, Environmental and Geotechnical disciplines, i.e. RPE (Civil, Environmental, Geotechnical), in Hong Kong by virtue of the Engineers Registration Ordinance (Chapter 409) of the Hong Kong Special Administrative Region ("**HKSAR**"), a Chartered Engineer (CEng) of the United Kingdom, and a Registered Professional Engineer (PE) of Texas, U.S.A.

13. I am a Fellow of the Hong Kong Institution of Engineers (FHKIE), a Fellow of the Institution of Civil Engineers (FICE) of the United Kingdom, a Fellow of the American Society of Civil Engineers (FASCE), and a member of the Chinese Institute of Civil and Hydraulic Engineering of Taiwan.

14. I was an Assistant Professor of Civil Engineering at Northeastern University, Boston, Massachusetts, U.S.A., and Texas A&M University, College Station, Texas, U.S.A. for a total of seven and a half years. Moreover, I was an Assistant Research Engineer of Texas Transportation Institute of College Station, Texas, U.S.A. for seven and a half years.

15. After my return to Hong Kong in 1998, I was Chief Engineer of Binnie Black & Veatch Hong Kong Limited (now Black & Veatch Hong Kong Limited) and Assistant Secretary for Financial Services and the Treasury of the Hong Kong Special Administrative Region Government ("**HKSARG**") before I joined the Department of Civil Engineering of The University of Hong Kong as an Associate Professor in 2003. Mr. Frederick S.H. Ma, Chairman of the MTRCL, was the Secretary for Financial Services and the Treasury during my tenure with the HKSARG.

16. I am also a Senior Research Fellow (Geotechnical/Pavement Materials) of the Hong Kong Road Research Laboratory.

17. I am the Immediate Past Chair of the Asian Civil Engineering Coordinating Council composed of thirteen (13) society members, namely the American Society of Civil Engineers ("**ASCE**"); the Chinese Institute of Civil and Hydraulic Engineering (CICHE); Engineers Australia (EA); the Indonesian Society of Civil and Structural Engineers (HAKI); the Institution of Civil Engineers, India (ICE India); the Institution of Engineers, Bangladesh (IEB); the Institution of Engineers, Pakistan (IEP); the Japan Society of Civil Engineers (JSCE); the Korean Society of Civil Engineers (KSCE); the Mongolian Association of Civil Engineers (MACE); the Philippine Institute of Civil Engineers (PICE); the Vietnam Federation of Civil Engineering Associations (VFCEA), and Nepal Engineers' Association (NEA).

18. I am a Past Director and a Past Chair of the Region 10 (International Region) Board of Governors and a Past Chair of the Region 10 Assembly, all of the ASCE, among many other professional services.

19. I was elected to the Election Committee (Engineering Subsector) of the HKSAR 2017-2022. I am a member of the Electoral College of the HKSAR for the 13th

National People's Congress of the People's Republic of China (香港特別行政區第十三屆全國人民代表大會選舉會議成員). I am also serving on the Appeal Tribunal Panel Section 45 of the Building Ordinance (Cap 123) of the Development Bureau of the HKSARG 2015-2021, among many other community services.

20. I served on the Geotechnical Engineers Registration Committee 2010-2013 and the Technical Committee for *Code of Practice for Site Supervision* 2012-2017 of the Buildings Department ("**BD**") of the HKSARG. I also served on the Review Panel under Land (Miscellaneous Provisions) Ordinance of the Highways Department ("**HyD**") of the HKSARG 2010-2016.

21. I have never been convicted of any criminal offence with the exception of traffic violations; nor have I been the subject of an adverse finding by any statutory, disciplinary or professional organization or tribunal, nor, as far as I am aware, am I the subject of any investigation by any statutory, disciplinary or professional organization or tribunal.

22. I have no interest in China Technology on which behalf I have been instructed by Lim & Lok, and I have not provided any advice or assistance to China Technology in respect of this project.

23. I have no interest in the MTRCL and I have not provided any advice or assistance to the MTRCL in respect of this project.

24. I have no interest in Atkins and I have not provided any advice or assistance to Atkins in respect of this project.

25. I have no interest in Ove Arup and Partners and I have not provided any advice or assistance to Ove Arup and Partners in respect of this project.

26. I have no interest in Tony Gee and Partners (Asia) Limited and I have not provided any advice or assistance to Tony Gee and Partners (Asia) Limited in respect of this project.

27. I served the HKSARG as Assistant Secretary for Financial Services and the Treasury during 2002-2003. However, I do not have any current interest in any departments of the HKSARG except for my appointment as an expert witness for the Environmental Protection Department in a court proceeding regarding an alleged violation of the Noise Control Ordinance (Cap. 400).

28. Details of my professional experience and qualifications are given in Appendix I.

AVAILABLE INFORMATION

29. My opinions presented in this Engineering Expert Report are based on my knowledge of engineering practice in Hong Kong, my on-site observations during the two (2) site visits on 17th and 19th December 2018, and my review of various documents available in the public domain.

MY TECHNICAL ANALYSIS

The Expert Panel

30. I was accepted by the Commission to serve on the Expert Panel on 14th December 2018 (Friday), three (3) days before the 1st site visit scheduled on 17th December 2018 (Monday), and was informed that there would be a meeting of the Expert Panel on 18th December 2018 (Tuesday) and a possible 2nd site visit on 19th December 2018 (Wednesday). Therefore, the time available for me to prepare for the site visits and the meeting of the Expert Panel was extremely limited.

31. Membership of the Expert Panel was unknown to me until the 1st site visit in the morning of 17th December 2018. Moreover, there was no formal agenda for the experts' meeting on 18th December 2018. Even the topics to be discussed at the meeting were not available before the meeting.

32. During the site visits, no member of the Expert Panel was allowed to take photographs except the chair of the Expert Panel, i.e. Professor Don McQuillan. We were told that the photographs taken during the two visits would be distributed to members of the Expert Panel after clearance with the Commission. On 2nd January 2019, I received some of the photographs taken on 17th December 2018 for preparation of this Engineering Expert Report. However, no picture taken on 19th December 2018 was available for my use.

Observations made during the 1st Site Visit on 17th December 2018

33. I was informed to meet at the elevator outside Mannings Health & Beauty Store in the concourse of Hung Hom Station at 9 a.m. on 17th December 2018 for the 1st site visit. However, all the other members of the Expert Panel, with the exception of Professor Francis T.K. Au, were gathered elsewhere without my knowledge. I only met them at the HUH Extension Site Office.

34. The Expert Panel was first briefed by Mr. Neil NG, Project Manager – SCL Civil – NSL of the MTRCL in the morning of 17th December 2018 at the HUH Extension Site Office.

35. We were informed that the opening-up of the structures constructed would serve two purposes: (i) verifying the as-constructed conditions due to gaps in the objective records of concerns on the reliability of the records; and (ii) assessing the workmanship in the coupler connections and steel bar fixing in light of allegations raised, and workmanship in other known/suspected irregularities such as honeycombs in the concrete at the soffit of the EWL Slab as revealed to the HKSARG on 28th August 2018 and non-compliant installation of shear links. However, it is stipulated in clause 6.4.2 of the MTRCL proposal of 4th December 2018 that "*it is noted that purposes (i) and (ii) are not mutually exclusive*."

36. We were informed that the EWL Slab would be opened up at twenty-four (24) selected locations to unveil the connection details of reinforcement steel bars between

the EWL Slab and D-Wall (East). Moreover, another twenty-eight (28) randomly selected locations on the EWL Slab would be opened up to evaluate the quality of workmanship of the connections between reinforcement steel bars and couplers. Four (4) of these locations are located at the top surface of the EWL Slab and twenty-four (24) are at the bottom surface. Similarly, twenty-eight (28) randomly selected locations on the NSL Slab would be opened up to evaluate the quality of workmanship of the connections between reinforcement steel bars and couplers. However, all these twenty-eight (28) locations are at the top surface of the NSL Slab, as it is very difficult to access the bottom surface of the NSL Slab. Couplers on different rows of reinforcement steel bars at each selected locations may be sampled. Three (3) couplers would be exposed at each of the fifty-six (56) random locations, resulting in the testing of one hundred sixty-eight (168) of randomly selected coupler samples. If the selected couplers are on an inner row of reinforcement steel bars, all the couplers on the outer row(s) of reinforcement steel bars will also be examined and included as extra samples. These random locations of couplers were selected by Professor Eddy K.F. Lam of the Department of Statistics and Actuarial Science of The University of Hong Kong on the basis of statistical significance.

37. I understand that the two platform slabs, i.e. the EWL Slab and the NSL Slab, would be analyzed separately. Statistical inference of the test results of the randomly selected samples given by the MTRCL are tabulated as follows:

Total sample number = 84	
Total no. of failures in the samples	**Max. failure rate in the population**
0	3.5%
1	5.5%
2	7.3%
3	9.0%
4	10.6%
5	12.1%

The maximum failure rate so inferred is dependent on the sample size. Details of the sampling methodology or statistical inference have not been revealed by the MTRCL. Moreover, failure is not defined in the MTRCL Proposal of 4th December 2018.

38. We managed to inspect two opened-up locations at the soffit of the EWL Slab within an air duct during the 1st site visit on 17th December 2018.

39. Three (3) couplers on the bottom layer of reinforcement steel bars were exposed at location E46 of the EWL Slab as shown in Figure 1(a). However, we were informed that these three couplers were not part of the eighty-four (84) randomly selected samples. The selected samples are on the 3rd layer of reinforcement steel bars from the bottom. Therefore, these three (3) couplers would serve as additional samples.

40. Two (2) couplers on the bottom layer of reinforcement steel bars were exposed at location E112 of the EWL Slab as shown in Figure 1(b). Further opening up was still required to expose the third coupler.

(a) Location E46

(b) Location E112

Figure 1. Exposed couplers at the soffit of the EWL Slab inspected on 17th December 2018

41. The progress of opening-up of locations on the top surface of the EWL Slab was briefly inspected. As the EWL Slab was covered by mass concrete for the platform and the track bed, it would take some time to get to the EWL Slab and to expose the couplers. At the time of inspection, no coupler has been exposed.

42. No defects of concrete were inspected by the Expert Panel during the 1st site visit on 17th December 2018.

Meeting of the Expert Panel on 18th December 2018

43. The Expert Panel met at the Tsuen Wan Law Courts Building in the morning 18th December 2018 at 11:00 a.m.

44. No agenda of the meeting was available to the participants prior to the meeting. In fact, there was no discussion among the participants on the items to be discussed at the meeting beforehand. Therefore, I could not do any preparatory works specific for the meeting. I consider many problems might have arisen from this lack of specific preparation by members of the Expert Panel.

45. The 1st bullet point of Item 1 "General Code requirements" of the Joint Expert Memorandum signed by all the experts provides that "*All agreed there was no requirement for ductility couplers*." The validity of the agreement warrants more discussion.

46. As these important documents on the use of couplers:

    - *Quality Supervision Plan on Enhanced Site Supervision & Independent Audit Checking by MTRC & RC for Installation of Couplers (Type II – SEISPLICE Standard Ductility Coupler)* ("**QSP**") prepared by Leighton and approved by the Building Authority ("**BA**") of the HKSARG;
    - *AC133 "Acceptance Criteria for Mechanical Splice Systems for Steel Reinforcing Bars"* published by the International Code Council;
    - *Code of Practice for Structural Use of Concrete 2004* ("**CoP 2004**") published by the BD of the HKSARG (BD 2004); and
    - *Code of Practice for Structural Use of Concrete 2013* ("**CoP 2013**") published by the BD of the HKSARG (BD 2013);

    were not available at the meeting for reference and I have not reviewed them in detail before the meeting, I relied on the opinions of the other members of the Expert Panel to make the agreement. I also understand that there is no requirement for seismic resistant design for reinforced concrete structures in Hong Kong for the time being. Therefore, I consider it is possible that ductility couplers may not be required by CoP

2004 or CoP 2013. Unfortunately, my agreement to the 1st bullet point of Item 1 "General Code requirements" of the Joint Expert Memorandum was an incorrect decision as elaborated below.

47. Upon detailed review of CoP 2013 (BD 2013), I concluded that the statement "*there was no requirement for ductility couplers*" is incorrect. In fact, the requirement for the bottom reinforcement steel to be at least 50% of the top tensile steel as stated in the 2nd bullet point is for ductility as stipulated in clause 9.9.1.2(a) of CoP 2013. There is at least inconsistency between the 1st bullet point and the 2nd bullet point of Item 1 "General Code requirements" of the Joint Expert Memorandum. These important code requirements for ductility will be further elaborated in this Engineering Expert Report.

48. More importantly, the use of ductility couplers was specified in the QSP prepared by Leighton and submitted by the MTRCL to the BA of the HKSARG on 12th August 2013. Therefore, in contrary to the 1st bullet point of Item 1 "General Code requirements" of the Joint Expert Memorandum, ductility couplers are certainly required for connecting the reinforcement steel bars of the platform slabs to those of D-Walls of the HUH Station Extension.

49. Item 6 of the Joint Expert Memorandum requires further clarification. As the amount of bottom reinforcement steel in the EWL Slab is similar to the amount of top reinforcement steel, I consider at least some of the bottom reinforcement steel is redundant, resulting in the redundancy of connection couplers. On the basis of the redundancy of couplers in the bottom of the EWL Slab, I agreed that further opening-up at the soffit of the EWL Slab might be unnecessary from a technical viewpoint. However, it should not be interpreted that "any further opening-up was unnecessary". From the technical viewpoint, more opening-up effort should be spent on components of the EWL Slab that are structurally more critical.

50. However, opening-up at the randomly selected locations may still be required to serve the original purposes of the MTRCL, as these purposes are not necessarily technical.

51. The soffit of the EWL Slab can be assessed through the air ducts underneath. I was informed by the Safety Manager of the MTRCL that these air ducts are not considered

as confined space with respect to safety hazards. The working environment within these air ducts cannot be considered comfortable as the headroom is only approximately 1.2 m. However, opening up the soffit of the EWL Slab is certainly workable provided normal safety precautions are taken by the construction workers.

52. In fact, the original investigation program may not be adequate to unveil all the as-constructed details of the reinforcement connection details between the EWL Slab and D-Wall (East) to evaluate whether they have followed the modified designs. Improvement of the current investigation program and proposed additional open-up locations as depicted in Figure 6 will be elaborated later in this Engineering Expert Report.

Observations made during the 2nd Site Visit on 19th December 2018

53. It should be recorded that Professor Francis T.K. Au of The University of Hong Kong did not attend the 2nd site visit due to a prior academic commitment.

54. All the remaining members of the Expert Panel inspected an opened-up location at location E44 at the top surface of the EWL Slab where two couplers were exposed. Eight (8) to nine (9) threads were exposed at one coupler and one (1) to two (2) were exposed at the other.

55. At this juncture, Professor Don McQuillan gave two instructions on site: (1) Measure the embedment lengths of the reinforcement steel bars in these two couplers on the same day. If it was not possible to make both measurements on the same day, at least measure the embedment length of the reinforcement steel bar in the coupler with eight (8) to nine (9) threads exposed; and (2) confirm whether there were 40-mm diameter Type B reinforcement steel bars manufactured on site. It was understood that 50-mm diameter Type B reinforcement steel bars were manufactured on site for the vertical reinforcement steel of the D-Walls.

56. I understand that the 2nd instruction was to avoid false alarm when too many threads were observed to be exposed.

57. I inspected the exposed threads of the reinforcement steel bar embedded in the coupler and found that some of the exposed threads were flattened slightly. It might thus be difficult for the reinforcement steel bar to penetrate the coupler due to possible damage of some of the threads.

58. From a construction and economic standpoint, it does not make any practical sense to connect a Type B reinforcement steel bar to a reinforcement steel bar already cast in D-Wall (East).

59. Let me first explain how a Type B reinforcement steel bar function. The number of threads on a Type B reinforcement steel bar is twice that of a Type A reinforcement steel bar. As a result, it is more expensive as more labor is required to prepare a Type B reinforcement steel bar. A coupler is typically installed on the Type B reinforcement steel bar so that the end of the coupler flushes with the end of the reinforcement steel bar after manufacturing to protect the threads from being damaged. In a typical connection, the reinforcement steel bars to be connected are brought into end-to-end contact. The coupler is then unscrewed from the Type B reinforcement steel bar to engage on the reinforcement steel bar to be connected, typically a Type A reinforcement steel bar, so that it is not necessary to rotate the Type A reinforcement steel bar to be connected. As a result, the extra threads of the Type B reinforcement steel bar will be exposed after connection. This type of connection is typically used for the vertical reinforcement steel bars in D-Walls or piles, as the vertical reinforcement steel bars being connected are very long and very difficult to maneuver on site.

60. The coupler that we observed on site has already been cast in D-Wall (East) so that it could not be rotated any more. It does not make any practical sense to bring in a Type B reinforcement steel bar, unscrew the coupler, and screw the Type B reinforcement steel bar into the coupler already cast in D-Wall (East). It is much more economical and efficient to simply screw a Type A reinforcement steel bar into the coupler.

61. Moreover, I consider that it may still a workmanship problem even if it were a Type B reinforcement steel bar. A Type B reinforcement steel bar should have twenty-two (22) to twenty-three (23) threads taking the tolerance into account. If only eight (8) to nine (9) threads are exposed after installation, at least thirteen (13) to fourteen (14) threads were engaged in the coupler. As there were only twenty-two (22) threads in the coupler, at most eight (8) to nine (9) threads were engaged in the coupler by the other bar under the condition of zero tolerance or two (2) full threads were exposed. Even if the exposure of two (2) full treads was acceptable, the exposed two (2) threads were on the wrong side of the coupler in accordance with the manufacturer's installation instruction, as they were on the reinforcement steel bar to be connected and not on the connecting reinforcement steel bar.

62. The opened-up locations at locations E60 and E72 of the EWL Slab were inspected only by me and Mr. Colin Wade while we were accompanied by staff of the MTRCL. Both locations are within an air duct of headroom of approximately 1.1 m. However, Professor Don McQuillan instructed Mr. Colin Wade to take photographs for him. As of now, I do not have opportunity to review these photographs.

63. Two (2) reinforcement steel bars were exposed at each of the two locations. One of the two (2) bars at location E60 was connected by a coupler and the other bar appeared to be a through bar. However, the coupler was not within D-Wall (East). Both reinforcement steel bars exposed at location E72 appeared to be through bars. However, only the connection details at the eastern edge of D-Wall (East) were inspected and the connection details at the western edge of D-Wall (East) were not inspected at all. Therefore, there is still doubt whether these exposed reinforcement steel bars are through bars into the EWL Slab. Further opening up is required to confirm the exposed steel bars are really through bars. Nonetheless, the observations revealed that the original connection details have been modified.

Basis of Design

64. The *Code of Practice for Structural Use of Concrete* ("**CoP**") is published by the BD of the HKSARG from time to time for the design, construction and quality of control of reinforced and prestressed concrete buildings and structures where the concrete is made with normal weight aggregates. Although the CoP is not a statutory document, the compliance with the requirements of the CoP is deemed to satisfy the relevant provisions of the Buildings Ordinance and related regulations.

65. It should be noted that the design and construction of the HUH Extension is under the jurisdiction of the Buildings Ordinance. Therefore, the requirements of the CoP are of direct relevancy to the design and construction of the HUH Extension.

66. Clause 2.1.2 "Design method" of CoP 2013 provides that "*The design method outlined in this code of practice is the limit state design method. In addition, consideration should be given to the requirement for durability,* ***<u>ductility</u>*** *and fire resistance. Equally important are the consideration of suitable materials, workmanship and quality control*." (emphasis bolded and underlined)

67. Clause 2.1.5 "Ductility" of CoP 2013 provides that "*The structure should be designed and constructed so that it has a certain degree of ability to deform beyond elastic limit without excessive strength or stiffness degradation*."

68. It is evident from CoP 2013 that the ductility should definitely be a design consideration for the structures of the HUH Extension. Therefore, ductility couplers are required for the reinforcement steel bars connecting the D-Walls and the platform slabs.

69. The requirements for detailing for ductility are stipulated in Clause 9.9 "*DETAILING FOR DUCTILITY*" of CoP 2013.

70. Clause 9.9.1.2(d) "Laps and type 1 mechanical couplers" of CoP 2013 provides that

"*For laps and type 1 mechanical couplers, no portion of the splice shall be located within one effective depth from the column/wall face.*"

71. Clause 9.9.1.2(e) "Type 2 mechanical couplers" of CoP 2013 provides that "*Type 2 mechanical couplers complying with the requirements given in clause 3.2.8.4 may be used in any location.*" Therefore, Type 2 mechanical couplers are ductility couplers.

72. It is evident from clauses 9.9.1.2(d) and 9.9.1.2(e) of CoP 2013 that Type 2 mechanical couplers have to be used for the connection of reinforcement steel bars between the platform slabs and the D-Walls.

73. More importantly, the MTRCL submitted the QSP to the BA of the HKSARG on 12th August 2013 (Ref.: 1112-COR-DM(SCL)-STO-000060) with a duly signed "Certification of preparation of plans and documents" (Certificate Ref.: 1112-IoE-PM(SCLCS)-STO-000025) on the use of ductility couplers for D-Wall reinforcement cage and slab construction at the HUH Station Extension. Therefore, the use of ductility couplers for the connection of reinforcement steel bars between the platform slabs and the D-Walls becomes a construction requirement.

74. In contrary to the 1st bullet point of Item 1 of the Joint Expert Memorandum, ductility couplers are required by the QSP and CoP 2013. Details of the Type 2 mechanical coupler, i.e. ductility couplers, will be elaborated later in this Engineering Expert Report.

Strength of the Reinforcement Steel Bar

75. At the onset, I need to clarify a few definitions used in engineering nomenclature as shown in Figure 2 to facilitate further discussion in this Engineering Expert Report. When stress is exerted onto a reinforcement steel bar, the percentage elongation is increased proportionally, i.e. the deformation under stress is elastic, until the applied stress reaches the upper yield field. The upper yield strength is the maximum value of stress exerted onto the steel bar prior to the first decrease in stress. The term yield

strength used in the discussion refers to the upper yield strength shown in Figure 2. The tensile strength of the reinforcement steel bar is the maximum stress that can be exerted onto the reinforcement steel bar. The reader is advised to understand the difference between the yield strength and the tensile strength of reinforcement steel bar. These two definitions are often used in CoP 2004 and CoP 2013, and the Construction Standard CS2 "Steel Reinforcing Bars for the Reinforcement of Concrete" published by the Development Bureau (2012) of the HKSARG ("**CS2:2012**") and CS2:1995 published by the Development Bureau (1995).

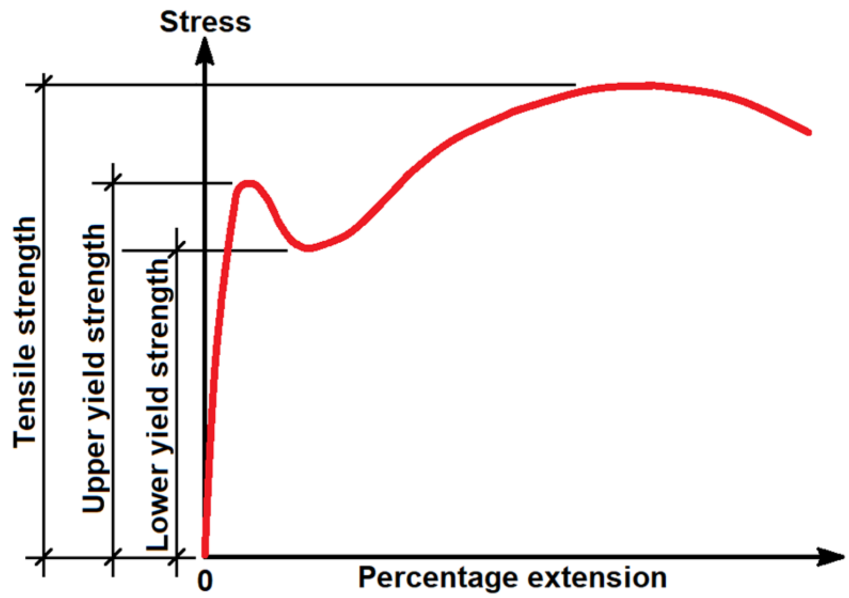


Figure 2 Definition of yield strength and tensile strength

76. Although Grade 460 reinforcement steel might be adopted in the design of the HUH Station Extension prior to 2013, all the reinforcement steel bars available in the Hong Kong market by 2013 is Grade 500B or Grade 500C. Therefore, recommendations of CoP 2013 and CS2:2012 on reinforcement steel bars should be followed.

77. In fact, BOSA Technology (Hong Kong) Limited ("**BOSA**"), the supplier of Type II mechanical couplers to Leighton, issued a clarification statement (澄清聲明) on 13th August 2018 (http://bosa-tech.com/images/pressrelease-201808.jpeg as of 7th January 2019) in response to a media report stating that the tensile strength of the coupler assembly manufactured by BOSA was not less than 625 MPa in compliance with the requirements of the BD of the HKSARG. The tensile strength requirement of the

coupler assembly of 625 MPa indicates that the characteristic strength of the reinforcement steel bar is 625/1.25 = 500 MPa. The requirement will be elaborated later in this Engineering Expert Report.

78. Clause 3.2.2 "Characteristic strength" of CoP 2013 provides that "*The characteristic strength of reinforcement, unless stated otherwise, means the proof or yield strength below which 5% of all possible test results would be expected to fall.*"

79. Clause 3.2.3 "Strength classes" of CoP 2013 provides that "

Grade	*Specified characteristics strength, f_y (N/mm^2)*
250	*250*
500B *500C*	*500*

Table 3.3 – Strength of reinforcement"

80. Clause 1.6.2 "Tensile properties" of CS2:2012 provides that

"*The tensile properties of the steel reinforcing bars as determined in accordance with Cl. 6.1 and 6.4 shall comply with the specified characteristic values for the tensile properties as given in Table 5.*

Table 5 – Characteristic tensile properties

Grade	***Yield strength, R_e*** *(MPa)*	***Tensile/yield strength ratio, R_m/R_e***	***Total elongation at maximum force, A_{gt}*** *(%)*
250	*250*	*1.15*	*5.0*
500B	*500*	*1.08*	*5.0*
500C	*500*	*≥1.15 and <1.35*	*7.5*

NOTES:

1. *Values of R_e specified are characteristic with $p = 0.95$.*
2. *Values of R_m/R_e specified are characteristic with $p = 0.90$. For grade 500C steel reinforcing bar, the upper limit of R_m/R_e is 1.35.*
3. *Values of A_{gt} specified are characteristic with $p = 0.90$.*
4. *Values of R_m and R_e are calculated using the nominal cross-sectional area.*

The absolute maximum permissible value of yield strength of grade 500 steel reinforcing bar is 650 MPa.

For yield strength (R_e) of grade 500 steel reinforcing bar, the upper yield strength (R_{eH}), which is the maximum value of stress prior to the first decrease in force, shall apply. Re shall be determined from the 0.2% proof strength ($R_{p0.2}$) in accordance with Appendix A if a yield phenomenon is not present. For Grade 250 steel reinforcing bars, R_e shall be determined from $R_{p0.2}$."

81. Therefore, the characteristic strength of reinforcement steel bar, i.e. 500 MPa, should not be confused with the tensile strength of the reinforcement steel bar. Moreover, the characteristic strength of the reinforcement steel bar should not be used to determine the minimum engagement length of the reinforcement steel bar with the ductility coupler, which may be governed by the tensile strength of the reinforcement steel bar. The required tensile strength of the coupler assembly will be elaborated later in this Engineering Expert Report.

82. Clause 3.1.3.3 "Tensile properties" of CS2:2012 provides that

"*3.1.3.3.1 Where the characteristic value C_v is specified as a lower limit as given in Table 5, the results shall be deemed to comply with this Standard if either:*

(a) all individual values of the test results are greater than or equal to the specified characteristic value C_v; or

(b) $\bar{x} \geq C_v + a_1$ and all individual values of the test results are greater than or equal to the minimum values given in Table 8

where

$\bar{x}$ is the average value of the test results; and

a_1 is the increment for calculation of batch release criteria.

(a_1 is 10 MPa for R_e, zero for R_m/R_e and 0% for A_{gt})

Table 8 – Absolute minimum and maximum values of tensile properties

Performance characteristic	***Minimum value***			***Maximum value***		
	250	***500B***	***500C***	***250***	***500B***	***500C***
R_e, *MPa*	*243*	*485*	*485*	*N/A*	*650*	*650*
R_m/R_e	*1.13*	*1.06*	*1.13*	*N/A*	*N/A*	*1.38*
A_{gt}, *%*	*4.0*	*4.0*	*6.0*	*N/A*	*N/A*	*N/A*

3.1.3.3.2 Where the characteristic value C_v is specified as an upper limit as given in Table 5 (i.e. for R_m/R_e of grade 500C), the results shall be deemed to comply with this Standard if either:

(a) all individual values of R_m/R_e are equal to or lower than the specified upper value of characteristic value of 1.35; or

(b) $\bar{x} \leq 1.35$ for R_m/R_e and all individual values for R_m/R_e are equal to or lower than the maximum values given in Table 8.

3.1.3.3.3 All individual values of R_e for grade 500 steel reinforcing bar shall be equal to or lower than the maximum value of 650 MPa as given in Table 8."

83. In summary, the yield strength of Grade 500B and 500C reinforcement steel bars have to be between 485 MPa and 650 MPa with a mean yield strength greater than 510 MPa. Therefore, the mean tensile strength of reinforcement steel bars should be greater than 1.15×510 = 586.5 MPa. For Grade 500C reinforcement steel bars, the mean tensile strength should be less than 1.35×500 = 675 MPa with a maximum not greater than 1.38×500 = 690 MPa.

Strength of the Coupler Assembly

84. Appendix A of the QSP provides that

"*The application is permitted for inter-storey columns provided that the following performance criteria are met:*

1. *Permanent elongation of the splicing assemblies after loading to 0.6F_y should not exceed 0.1mm in accordance to Clause 3.2.8.2 of CoP for Structural Use of Concrete 2004.*
2. *Pass all AC133 cyclic and static (compression and tension) test.*
3. *Tensile strength of the splicing assemblies shall be at least 529 MPa on CS2 Grade 460 rebar.*
4. *Tensile stress of the splice to be at least 95% of the actual strength of the connected rebars.*
5. *The splicing assemblies shall fail in bar-break mode (i.e. failure occurs in the reinforcing bar).*

Use of Type II coupler in any location of the structure is allowed in ACI 318. With the enhanced acceptance criteria of having "bar-break" (failure occurs in the reinforcing bar) as required mode of failure, the mechanical re bar coupler should be allowed to be used in any location."

85. Appendix A of the QSP made reference to CoP 2004 and CS2:1995 as it referred to Grade 460 reinforcement steel bars. However, it also made reference to AC133 "Acceptance Criteria for Mechanical Splice Systems for Steel Reinforcing Bars", published by the International Code Council to supplement the deficiencies of CoP 2004 and CS2:1995.

86. AC133 "Acceptance Criteria for Mechanical Splice Systems for Steel Reinforcing Bars", published by the International Code Council, defines the process to uniformly evaluate mechanical connector systems of steel reinforcing bars used in building and construction. A copy of AC133 is enclosed in Appendix II for easy reference.

87. It should be noted that the requirements for the cyclic and static tension and compression tests for Type II mechanical couplers stipulated in Table 1 of AC133 are identical to those stipulated in clause 3.2.8.4 of CoP 2013.

88. Clause 3.2.8 "Mechanical couplers" of CoP 2013 provides that

"*3.2.8.1 General*

Mechanical couplers are classified as

(a) *Type 1 mechanical couplers that conform to clause 3.2.8.3.*
(b) *Type 2 mechanical couplers that conform to clause 3.2.8.4.*

3.2.8.2 Butt jointed bars in compression only

The load may be transferred between butt jointed bars by means of end bearing where sawn square cut ends are held in contact by means of a suitable sleeve or other coupler.

3.2.8.3 Performance of type 1 mechanical couplers

Type 1 mechanical coupler satisfying the following criteria may be used as an alternative to tension or compression laps:

(a) *when a representative gauge length assembly comprising reinforcement of the diameter, grade and profile to be used, and a coupler of the precise type to be used, is tested in tension the permanent elongation after loading to 0.6fy should not exceed 0.1 mm; and*
(b) *the coupled bar assembly tensile strength should exceed 287.5 N/mm^2 for grade 250, 540 N/mm^2 for grade 500B and 575 N/mm^2 for grade 500C.*

3.2.8.4 Performance of type 2 mechanical couplers

Type 2 mechanical coupler should satisfy the following criteria:

(a) *The splicing assemblies shall be tested to establish that they comply with the requirements given in clause 3.2.8.3.*
(b) *Static tension test: The splicing assemblies must develop in tension the greater of 100 percent of the specified tensile strength, R_m, of the bar, and 125 percent of the specified yield strength, fy, of the bar.*
(c) *Static compression test: The splicing assemblies must develop in compression 125 percent of the specified yield strength, f_y, of the bar.*
(d) *Cyclic tension-and-compression test: The splicing assemblies shall be tested in four stages as given in Table 3.4, and must sustain Stages 1 through 3 without failure. If the conditions of acceptance for the static tension test are complied with in Stage 4, the static tension test may be omitted.*

The use of type 2 mechanical coupler should comply with the requirements given in clause 9.9.

Stage	*Tension*	*Compression*	*Cycles*
1	*$0.95f_y$*	*$0.5f_y$*	*20*
2	*$2\varepsilon_y$*	*$0.5f_y$*	*4*
3	*$5\varepsilon_y$*	*$0.5f_y$*	*4*
4	*Load in tension to failure*		
Notes: *1. ε_y is the strain of reinforcing bar at actual yield stress.* *2. The actual ultimate tensile strength of the bar is obtained by testing samples from a referenced reinforcing bar. The test samples are obtained from the same referenced reinforcing bar.*			

Table 3.4 – Cyclic tension-and-compression test"

89. Therefore, the coupler assemblies must develop in mean tension the greater of 586.5 MPa or 1.25×510 = 637.5 MPa, i.e. 637.5 MPa. Moreover, the coupler assemblies must develop in mean compression of 1.25×510 = 637.5 MPa. In addition, the results of tests on all test units of continuous production must satisfy the long-term quality requirements stipulated in clause 3.2.2 of CS2:2012 (Development Bureau 2012).

90. The tensile strength of 529 MPa required for the coupler assembly stipulated by Ir K.P. Yim in the press conference of the MTRCL on 24th December 2018 was probably taken Appendix A of the QSP for Grade 460 reinforcement steel bars.

91. It is obvious that tensile strength of 529 MPa is not acceptable for the coupler assembly in accordance with requirements stipulated in Appendix A of the QSP. Even if the characteristic strength of the reinforcement steel bar is taken to be 460 MPa, the minimum tensile strength of the coupler assembly to pass the static tension requirement of AC133 would be 1.25×460 = 575 MPa. Moreover, it is required that the coupler assembly has to fail in the bar-break mode, i.e. the coupler assembly must be stronger than the reinforcement steel bars that it connects. As elaborated earlier, it is very likely that Grade 500 reinforcement steel bars are being used for the construction of the HUH Station Extension, resulting in the requirement of a

minimum tensile strength of the coupler assembly of 625 MPa, as stated in the clarification statement of BOSA of 13th August 2018.

92. The tensile strength of the coupler assembly with connecting reinforcement steel bars fully engaged of 1,003 MPa as released by the MTRCL in the press conference on 24th December 2018 is questionable. I cannot comprehend how the actual tensile strength of the coupler assembly can be measured after the connecting reinforcement steel bars fail. In other words, how can a Grade 500B or Grade 500C reinforcement steel bar resist an applied tensile stress of 1,003 MPa without breaking so as to evaluate the tensile strength of the engagement between the reinforcement steel bar and the coupler?

93. Ir K.P. Yim postulated openly in the MTRCL press conference of 24th December 2018 that it was structurally adequate for the reinforcement steel bar to engage only six (6) full threads of the coupler, i.e. 60% of all the number of threads recommended by the manufacturer. His postulate fails in two aspects: (1) the tensile strength of the coupler assembly of 1,003 MPa has not been proven experimentally; and (2) even if (1) can be proven experimentally and the tensile strength of the coupler assembly is proportional to the extent of engagement, the engagement of six threads is still inadequate to provide a tensile strength of 625 MPa for Grade 500B or Grade 500C reinforcement steel bars.

Acceptance Criterion of Embedment Length in Coupler

94. The dimensions of Seisplice Type A couplers made by BOSA are tabulated as follows with definitions of terminologies shown in Figure 3:

Bar size (mm)	Coupler		Tolerance TOL (mm)	Metric thread × pitch
	D (mm)	L (mm)		
12	19	29	2.0	M13 × 2.0
16	25	38	2.0	M17 × 2.0
20	30	47	2.5	M21 × 2.5

Bar size (mm)	Coupler		Tolerance TOL (mm)	Metric thread × pitch
	D (mm)	L (mm)		
25	38	58	2.5	M26 × 2.5
32	48	73	3.5	M33 × 3.5
40	60	88	4.0	M40.5 × 4.0
50	75	110	4.0	M50 × 4.0

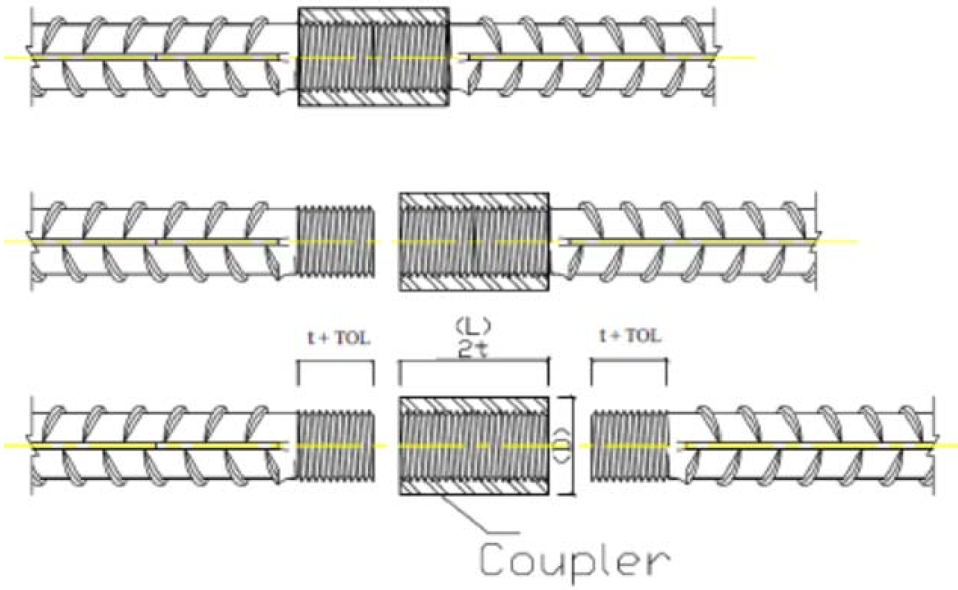


Figure 3. Dimensions of Seisplice Type A couplers

95. The threaded section of a 40-mm diameter Type A reinforcement steel bar is 44 mm. The number of full threads is between ten (10) and eleven (11). With a tolerance of 4 mm, the total number of full threads is between eleven (11) and twelve (12).

96. The acceptance criterion recommended by the manufacturer is shown in Figure 4. It can be deduced from Figure 4 that the minimum embedment length is 40 mm and the minimum number of full threads engaged should be ten (10).

Summary:

1. After connection has been fully tightened, one should see a maximum of TWO FULL THREADS to ensure a proper installation.
2. Please refer to our SEISPLICE Technical and Quality Assurance Manual Dimensions table for our thread length tolerance. Appendix 4 of our Coupler Specifications.
3. Under normal circumstances, we provide a positive tolerance of half a thread.
4. As illustrated in the above scenario, the exposed thread, if any, always occurs at the top of the continuation bar.

Figure 4. Acceptable thread tolerance

97. The technique of Phased Array Ultrasonic Testing is used to measure the embedment length of the reinforcement steel bar into the coupler non-destructively. The accuracy of measurement of the technique is ±3 mm. It should be noted that the technique is measuring the embedment length and not the engagement length between the reinforcement steel bar and the coupler or the actual number of threads engaged. However, the magnitude of the force transmitted through the coupler depends on the

engagement length and the number of threads engaged, and not on the embedment length. Therefore, tolerance between the embedment length and engagement length should be allowed.

98. When the embedment length measured is 40 mm, the actual embedment length is between 37 mm and 43 mm. The probability of the actual embedment length exceeding 40 mm is practically 50% while that less than 40 mm is also 50%. Therefore, if the embedment length measured is 40 mm, there is practically a 50% probability that the actual embedment length is less than 40 mm and does not meet the installation requirement of the manufacturer.

99. If the measured embedment length of 37 mm is adopted to be the acceptance criterion, the probability that the actual embedment length exceeding 40 mm is practically zero. In other words, it is almost certain that the actual embedment length is less than 40 mm, the embedment length recommended by the manufacturer.

100. Mr. Frank Chan, Secretary for Transport and Housing, emphasized in his response to the oral questions raised by the Honorable YIU, Si-wing (姚思榮議員) during the Special Meeting of the Panel on Transport ("**the Panel**") of the Legislative Council of 31st August 2018 that safety of absolute certainty was required for the operation of the railway. His exact wording was "我哋要確保鐵路運作係百份百安全". In his opening remarks at the meeting of the Subcommittee on Matters Relating to Railways ("**the Subcommittee**") of the Panel on 7th December 2018, Mr. Frank Chan stated again repeatedly and clearly that safety is the top priority of the administration. In accordance with his statements, the actual embedment length should not be less than 40 mm. As a result, the measured embedment length should not be less than 43 mm to achieve the target – safety of absolute certainty.

101. Adopting an acceptance criterion for the measured embedment length to be between 40 mm to 43 mm would result in a practical probability of failure of 50% to 0%. However, taking 43 mm may be too stringent. Therefore, the acceptable measured embedment length is recommended to be 40 mm, in consistent with the manufacturer's recommendation.

Field Data on Measurements of Embedment Lengths Available to Date

102. The field data on measurements of embedment lengths as released by the HyD of the HKSARG are tabulated as follows:

Test Results of Couplers Exposed for the 1st Purpose (As at 4th January 2019)

Coupler No.	Location of Coupler Tested	No. of Exposed Thread	Embedment Length - Preliminary Results (mm)
EWL-E44-TT-T1-01-C1	At the top of Area B of the EWL Slab near D-Wall (East)	1-2	31.61
EWL-E44-TT-T1-02-C1	At the top of Area B of the EWL Slab near D-Wall (East)	8-9	6.22
EWL-E32-TT-T1-01-C1	At the top of Area Hong Kong Coliseum (HKC) of the EWL Slab near D-Wall (East)	0-1	36.65
EWL-E32-TT-T1-02-C1	At the top of Area HKC of the EWL Slab near D-Wall (East)	0-1	42.02
EWL-E32-TT-T1-03-C1	At the top of Area HKC of the EWL Slab near D-Wall (East)	0-1	41.51
EWL-E35-TT-T1-01-C1	At the top of Area HKC of the EWL Slab near D-Wall (East)	0	47.01
EWL-E35-TT-T1-02-C1	At the top of Area HKC of the EWL Slab near D-Wall (East)	0-1	45.70
EWL-E35-TT-T1-03-C1	At the top of Area HKC of the EWL Slab near D-Wall (East)	0-1	43.45
EWL-E37-TT-T1-01-C1	At the top of Area HKC of the EWL Slab near D-Wall (East)	1-2	39.84
EWL-E37-TT-T1-02-C1	At the top of Area HKC of the EWL Slab near D-Wall (East)	1-2	42.51
EWL-E46-TT-T1-01-C1	At the top of Area B of the EWL Slab near D-Wall (East)	0-1	44.04
EWL-E46-TT-T1-02-C1	At the top of Area B of the EWL Slab near D-Wall (East)	2-3	33.00

Test Results of Couplers Exposed for the 2nd Purpose (As at 4th January 2019)

Coupler No.	Location of Coupler Tested	No. of Exposed Thread	Embedment Length - Preliminary Results (mm)
EWL-E46-BB-B1-01-C1	At the bottom of Area B of the EWL Slab near D-Wall (East)	2-3	34.91
EWL-E46-BB-B1-02-C1	At the bottom of Area B of the EWL Slab near D-Wall (East)	3-4	29.65
EWL-E46-BB-B1-03-C1	At the bottom of Area B of the EWL Slab near D-Wall (East)	2-3	34.32
EWL-E70-BB-B1-01-C1	At the bottom of Area C1 of the EWL Slab near D-Wall (East)	2-3	40.51
EWL-E70-BB-B1-02-C1	At the bottom of Area C1 of the EWL Slab near D-Wall (East)	1-2	36.78
EWL-E40-TT-T1-01-C1[1]	At the top of Area B of the EWL Slab near D-Wall (East)	9-10	39.21
EWL-E40-TT-T1-02-C1[1]	At the top of Area B of the EWL Slab near D-Wall (East)	10-11	40.81
EWL-E40-TT-T1-03-C1[1]	At the top of Area B of the EWL Slab near D-Wall (East)	11-12	38.57
EWL-E65-BB-B1-01-C1	At the bottom of Area C1 of the EWL Slab near D-Wall (East)	0	42.43
EWL-E107-BB-B1-01-C1	At the bottom of Area C3 of the EWL Slab near D-Wall (East)	0	35.34
EWL-E107-BB-B1-02-C1	At the bottom of Area C3 of the EWL Slab near D-Wall (East)	6-7	9.40
EWL-E107-BB-B1-03-C1	At the bottom of Area C3 of the EWL Slab near D-Wall (East)	0	40.91
EWL-E90-BB-B1-01-C1	At the bottom of Area C2 of the EWL Slab near D-Wall (East)	0	41.43
EWL-E90-BB-B1-02-C1	At the bottom of Area C2 of the EWL Slab near D-Wall (East)	0	43.82
EWL-E90-BB-B1-03-C1	At the bottom of Area C2 of the EWL Slab near D-Wall (East)	0	43.85

Test Results of Couplers Exposed for the 2nd Purpose
(As at 4th January 2019)

Coupler No.	Location of Coupler Tested	No. of Exposed Thread	Embedment Length - Preliminary Results (mm)
EWL-E50-BB-B1-01-C1	At the bottom of Area B of the EWL Slab near D-Wall (East)	0	34.80
EWL-E77-BB-B1-01-C1	At the bottom of Area C2 of the EWL Slab near D-Wall (East)	0	45.22
EWL-E96-BB-B1-01-C1	At the bottom of Area C3 of the EWL Slab near D-Wall (East)	0	42.02
EWL-E112-BB-B1-02-C1	At the bottom of Area C3 of the EWL Slab near D-Wall (East)	0	48.72
EWL-W58-BB-B1-01-C1	At the bottom of Area C1 of the EWL Slab near D-Wall (West)	0-1	40.04
EWL-W58-BB-B1-02-C1	At the bottom of Area C1 of the EWL Slab near D-Wall (West)	0	45.85
EWL-W58-BB-B1-03-C1	At the bottom of Area C1 of the EWL Slab near D-Wall (West)	0-1	39.22
NSL-E68-TT-T1-01-C1	At the top of Area C1 of the NSL Slab near D-Wall (East)	0	39.38
NSL-E68-TT-T1-02-C1	At the top of Area C1 of the NSL Slab near D-Wall (East)	1-2	40.07
NSL-E80-TT-T1-01-C1	At the top of Area C2 of the NSL Slab near D-Wall (East)	0	38.01
NSL-E80-TT-T1-02-C1	At the top of Area C2 of the NSL Slab near D-Wall (East)	0-1	38.33
NSL-E80-TT-T1-03-C1	At the top of Area C2 of the NSL Slab near D-Wall (East)	0	36.36

Note: [1]The steel bar may be Type B reinforcement bar.

103. It should be noted that the data are preliminary as measured on site. Final test reports are to be provided by the MTRCL in due course.

104. The length of the threaded section of the reinforcement steel bar is theoretically the sum of the embedment length of the reinforcement steel bar in the coupler and the length of exposed threads. The reinforcement steel bar may have been cut if the length of the threaded section of a Type A reinforcement steel bar is less than 44 mm while that of a Type B reinforcement steel bar is less than 88 mm, without taking the possible tolerance of 4 mm into account. The actual lengths of the threaded sections should be longer than these values due to the inclusion of tolerances in the manufacturing process.

105. The measurement accuracy of the embedment length of reinforcement steel bar in the coupler is ±3 mm and the number of exposed threads is not a definite number due to the configuration of the threads. Therefore, the maximum possible length of the threaded section equals the measured embedment length + 3 mm + maximum number of threads exposed × 4 mm and the minimum possible length of the threaded section equals the measured embedment length - 3 mm + minimum number of threads exposed × 4 mm.

106. The results of the calculations on the maximum possible lengths, the minimum possible lengths and the average lengths of the threaded sections of the reinforcement steel bars for couplers exposed for the 1st purpose are tabulated in Table 1.

Table 1. Analysis of test results of couplers exposed for the 1st purpose as of 4th January 2018

Coupler No. (1)	**Min. no. of exposed threads** (2)	**Max. no. of exposed threads** (3)	**Embed-ment length measured (mm)** (4)	**Max. length of threaded section (mm)[1]** (5)	**Min. length of threaded section (mm)[2]** (6)	**Average length of threaded section (mm)** (7)
EWL-E44-TT-T1-01-C1	1	2	**31.61**	**42.61**	**32.61**	**37.61**
EWL-E44-TT-T1-02-C1	8	9	**6.22**	45.22	**35.22**	**40.22**
EWL-E32-TT-T1-01-C1	0	1	**36.65**	**43.65**	**33.65**	**38.65**
EWL-E32-TT-T1-02-C1	0	1	42.02	49.02	**39.02**	44.02
EWL-E32-TT-T1-03-C1	0	1	41.51	48.51	**38.51**	**43.51**
EWL-E35-TT-T1-01-C1	0	0	47.01	50.01	44.01	47.01
EWL-E35-TT-T1-02-C1	0	1	45.70	52.70	**42.70**	47.70

EWL-E35-TT-T1-03-C1	0	1	43.45	50.45	**40.45**	45.45
EWL-E37-TT-T1-01-C1	1	2	**39.84**	50.84	**40.84**	45.84
EWL-E37-TT-T1-02-C1	1	2	42.51	53.51	**43.51**	48.51
EWL-E46-TT-T1-01-C1	0	1	44.04	51.04	**41.04**	46.04
EWL-E46-TT-T1-02-C1	2	3	**33.00**	48.00	**38.00**	**43.00**

Notes:
[1]Column (5) = Column (4) + 3 mm + Column (3) × 4 mm
[2]Column (6) = Column (4) - 3 mm + Column (2) × 4 mm

107. It can be observed in column (4) of Table 1 that five (5) out of the twelve (12) samples do not meet the manufacturer's requirement on minimum embedment length of 40 mm, i.e. 5/12 = 41.7%. Even if the acceptable embedment length measured is reduced to 37 mm, which I do not agree, four (4) out of twelve (12) samples still cannot meet the requirement, i.e. 4/12 = 33.3%. Nonetheless, it is an evident indication of poor workmanship on the connections of reinforcement steel bars between the platform slabs and the D-Walls.

108. Moreover, it can be observed in column (5) of Table 1 that the threaded sections of two (2) of the twelve (12) reinforcement steel bars are definitely shorter than 44 mm. There is a probability that the threaded sections of eleven (11) out of twelve (12) reinforcement steel bars are shorter than 44 mm as tabulated in column (6) of Table 1, i.e. the minimum possible lengths of the threaded sections of reinforcement steel bars are shorter than 44 mm. The most probable number of reinforcement steel bars with threaded sections shorter than 44 mm is five (5) out of twelve (12) as tabulated in column (7) of Table 1, i.e. the average lengths of the threaded sections of reinforcement steel bars are shorter than 44 mm.

109. The results of a similar analysis on the set of data of EWL Slab obtained for the 2nd purpose are tabulated in Table 2.

Table 2. Analysis of test results of couplers exposed at EWL Slab for the 2nd purpose as of 4th January 2018

Coupler No. (1)	Min. no. of exposed threads (2)	Max. no. of exposed threads (3)	Embed-ment length measured (mm) (4)	Max. length of threaded section (mm)[1] (5)	Min. length of threaded section (mm)[2] (6)	Average length of threaded section (mm) (7)
EWL-E46-BB-B1-01-C1	2	3	**34.91**	49.91	**39.91**	44.91
EWL-E46-BB-B1-02-C1	3	4	**29.65**	48.65	**38.65**	**43.65**
EWL-E46-BB-B1-03-C1	2	3	**34.32**	49.32	**39.32**	44.32
EWL-E70-BB-B1-01-C1	2	3	40.51	55.51	45.51	50.51
EWL-E70-BB-B1-02-C1	1	2	**36.78**	47.78	**37.78**	**42.78**
EWL-E40-TT-T1-01-C1[3]	9	10	**39.21**	**82.21**	**72.21**	**77.21**
EWL-E40-TT-T1-02-C1[3]	10	11	40.81	**87.81**	**77.81**	**82.81**
EWL-E40-TT-T1-03-C1[3]	11	12	**38.57**	89.57	**79.57**	**84.57**
EWL-E65-BB-B1-01-C1	0	0	42.43	45.43	**39.43**	**42.43**
EWL-E107-BB-B1-01-C1	0	0	**35.34**	**38.34**	**32.34**	**35.34**
EWL-E107-BB-B1-02-C1	6	7	**9.40**	**40.40**	**30.40**	**35.4**
EWL-E107-BB-B1-03-C1	0	0	40.91	**43.91**	**37.91**	**40.91**
EWL-E90-BB-B1-01-C1	0	0	41.43	44.43	**38.43**	**41.43**
EWL-E90-BB-B1-02-C1	0	0	43.82	46.82	**40.82**	**43.82**
EWL-E90-BB-B1-03-C1	0	0	43.85	46.85	**40.85**	**43.85**
EWL-E50-BB-B1-01-C1	0	0	**34.80**	**37.80**	**31.80**	**34.80**
EWL-E77-BB-B1-01-C1	0	0	45.22	48.22	**42.22**	45.22
EWL-E96-BB-B1-01-C1	0	0	42.02	45.02	**39.02**	**42.02**
EWL-E112-BB-B1-02-C1	0	0	48.72	51.72	45.72	48.72
EWL-W58-BB-B1-01-C1	0	1	40.04	47.04	**37.04**	**42.04**
EWL-W58-BB-B1-02-C1	0	0	45.85	48.85	**42.85**	45.85
EWL-W58-BB-B1-03-C1	0	1	**39.22**	46.22	**36.22**	**41.22**

Notes:
[1]Column (5) = Column (4) + 3 mm + Column (3) × 4 mm
[2]Column (6) = Column (4) - 3 mm + Column (2) × 4 mm
[3]These may be Type B reinforcement steel bars with standard length of 88 mm.

110. It can be observed in column (4) of Table 2 that ten (10) out of the twenty-two (22) samples of EWL Slab do not meet the manufacturer's requirement on minimum embedment length of 40 mm, i.e. 10/22 = 45.5%. Even if the acceptable embedment

length measured is reduced to be 37 mm, which I do not agree, seven (7) out of twenty-two (22) samples still cannot meet the requirement, i.e. 7/22 = 31.8%.

111. If the sampling and testing process is stopped now, the maximum failure rate in the population would be 45.5% or 31.8%, depending on the acceptance criterion, plus the margin of error of sampling.

112. Even if the results of all the remaining tests would satisfy the embedment length requirement, the maximum failure rate in the population of the EWL Slab would exceed 12.1% in accordance with the statistical criteria provided by the HyD of the HKSARG. The actual maximum failure rate in the population is unknown as the HyD of the HKSARG did not provide the maximum rate in the population when the number of failures in the samples exceeds five (5). Nonetheless, it is an evident indication of poor workmanship in the connections of reinforcement steel bars between the EWL Slab and the D-Walls.

113. Moreover, it can be observed in column (5) of Table 2 that the threaded sections of four (4) of the nineteen (19) Type A reinforcement steel bars in EWL Slab are shorter than 44 mm with certainty. Moreover, two (2) of the three (3) Type B reinforcement steel bars are shorter than 88 mm with certainty. Therefore, six (6) out of twenty-two (22) reinforcement steel bars have the lengths of threaded sections shorter than the respective design values with certainty. There is a probability that the threaded sections of twenty (20) out of twenty-two (22) reinforcement steel bars in the EWL Slab are shorter than the design values as tabulated in column (6) of Table 2. The most probable number of reinforcement steel bars with threaded sections shorter than the design values is sixteen (16) out of twenty-two (22) of the EWL Slab as tabulated in column (7) of Table 2.

114. The results of a similar analysis on the set of data of the NSL Slab for couplers exposed for the 2nd purpose are tabulated in Table 3.

Table 3. Analysis of test results of couplers exposed at the NSL Slab for the 2nd purpose as of 4th January 2018

Coupler No. (1)	**Min. no. of exposed threads** (2)	**Max. no. of exposed threads** (3)	**Embed-ment length measured (mm)** (4)	**Max. length of threaded section (mm)[1]** (5)	**Min. length of threaded section (mm)[2]** (6)	**Average length of threaded section (mm)** (7)
NSL-E68-TT-T1-01-C1	0	0	**39.38**	**42.38**	**36.38**	**39.38**
NSL-E68-TT-T1-02-C1	1	2	40.07	51.07	**41.07**	46.07
NSL-E80-TT-T1-01-C1	0	0	**38.01**	**41.01**	**35.01**	**38.01**
NSL-E80-TT-T1-02-C1	0	1	**38.33**	45.33	**35.33**	**40.33**
NSL-E80-TT-T1-03-C1	0	0	**36.36**	**39.36**	**33.36**	**36.36**

Notes:
[1]Column (5) = Column (4) + 3 mm + Column (3) × 4 mm
[2]Column (6) = Column (4) - 3 mm + Column (2) × 4 mm

115. It can be observed in column (4) of Table 3 that four (4) out of the five (5) samples of the NSL Slab do not meet the manufacturer's requirement on minimum embedment length of 40 mm, i.e. 4/5 = 80%. Even if the acceptable embedment length measured is reduced to 37 mm, which I do not agree, one (1) out of five (5) samples still cannot meet the requirement, i.e. 1/5 = 20%.

116. If the sampling and testing process is stopped now, the maximum failure rate in the population would be 80% or 20%, depending on the acceptance criterion, plus the margin of error of sampling.

117. Even if the results of all the remaining tests would satisfy the embedment length requirement, the maximum failure rate in the population of EWL Slab would be 5.5% or 10.6%, depending on the failure criterion adopted, in accordance with the statistical criteria provided by the HyD of the HKSARG. It is an evident indication of poor workmanship in the connections of reinforcement steel bars between the EWL Slab and the D-Walls.

118. Moreover, it can be observed in column (5) of Table 3 that the threaded sections of three (3) of the five (5) Type A reinforcement steel bars in the NSL Slab are shorter

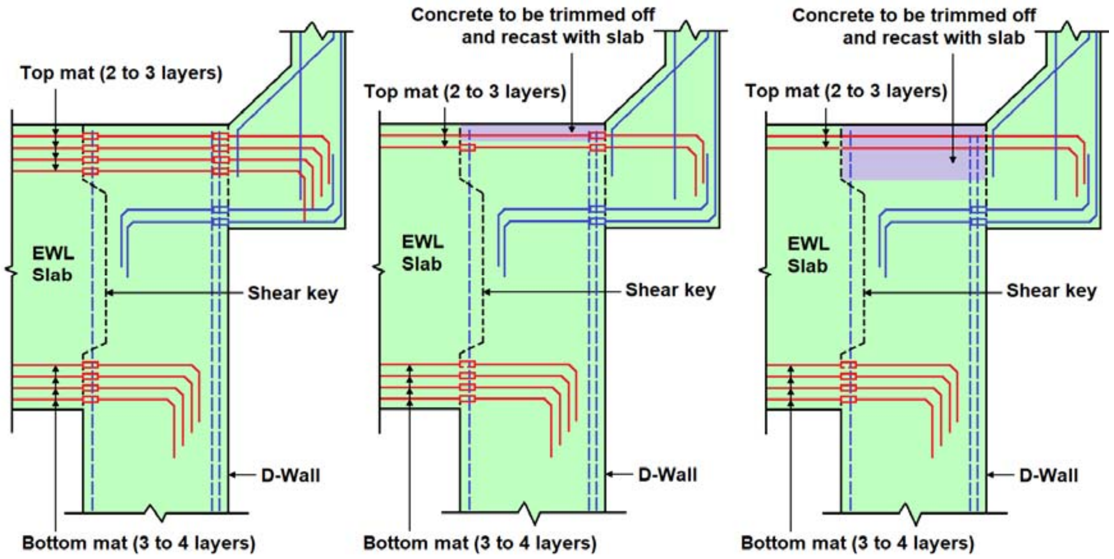
than 44 mm with certainty. There is a probability that the threaded sections of five (5) out of five (5) reinforcement steel bars in the NSL Slab are shorter than the design values as tabulated in column (6) of Table 3. The most probable number of reinforcement steel bars with threaded sections shorter than the design values is four (4) out of five (5) of the NSL Slab as tabulated in column (7) of Table 3.

119. It should be noted that the statistical criteria provided by the HyD of the HKSAR should be applied to the data tabulated in Tables 2 and 3 individually for the EWL Slab and the NSL Slab, respectively.

120. If the three sets of data are combined into a single analysis, nineteen (19) out of the thirty-nine (39) samples do not meet the manufacturer's requirement on minimum embedded length of 40 mm, i.e. 19/39 = 48.7%. Even if the acceptable embedded length measured is reduced to 37 mm, twelve (12) out of thirty-nine (39) samples still cannot meet the requirement, i.e. 12/39 = 30.8%. Even if the results of all the remaining tests would satisfy the embedment length requirement, the maximum failure rate in the population would exceed 12.1% in the EWL Slab and 5.5% or 10.6% in the NSL Slab, in accordance with the statistical criteria provided by the HyD of the HKSARG. Although the statistics may change slightly due to the change of sample size, it is an evident indication of poor workmanship in the connections of reinforcement steel bars between the platform slabs and the D-Walls.

EWL Slab and D-Wall (East) Connection

121. The three types of connections between the EWL Slab and D-Wall (East) are depicted in Appendix B of the MTRCL Proposal dated 4th December 2018. These three types of connections are reproduced in Figure 5 for easy reference.



(a) Type 1 - Couplers for all layers at top mat

(b) Type 2 - Only top layer of couplers at top mat replaced with half through steel bars

(c) Type 3 - All top layers of couplers at top mat replaced with through steel bars

Figure 5. Types of EWL Slab / D-Wall (East) connections

122. I understand that Type 1 connection was the original design by Atkins. Both Type 2 and Type 3 connections are modifications of the original design by Leighton to suit its construction procedure. However, I have no information whether the modifications have been accepted by the BA of the HKSARG.

123. As observed in the two opened-up locations, both connections may probably be Type 3. Two (2) 40-mm diameter reinforcement steel bars of the top layer in the top mat of reinforcement steel bars were observed in each of the two opened-up locations. The spacing between the two reinforcement steel bars is approximately 150 mm. A coupler located outside D-Wall (East) to the east was observed at one of the two locations connecting reinforcement steel bars. However, the 2nd and the 3rd, if any, layer of reinforcement steel bars of the top mat of bars have not been exposed for inspection. Therefore, it cannot be confirmed whether the as-constructed connections follow the modified design.

124. The coupler observed was probably installed to suit site conditions for the construction of the slab outside D-Wall (East) to the east and not part of the modified design. I have no information whether such a change in reinforcement details has been

reported to Atkins, the MTRCL or the BA of the HKSARG. Provided the requirement of embedment length of the reinforcement steel bar in the coupler satisfies the installation requirements of the manufacturer, the structural integrity of the rebar connection should not be a technical problem. However, such a change in reinforcement details from the design should be documented and reported to relevant authorities.

125. However, the two (2) opened-up locations as shown in Figure 6 are within D-Wall (East) but close to the eastern edge of D-Wall (East). The size of the opened-up is approximately 250 mm by 250 mm on plan. The depth of the opened-up could barely expose the top layer of reinforcement steel bars. Therefore, only some of the reinforcement connections on the eastern edge of D-Wall (East) could be inspected. The actual reinforcement connection details between the EWL Slab and D-Wall (East) at the western edge were practically not inspected at all, not to mention the confirmation of replacement of coupler-connected reinforcement steel bars by through steel bars into the EWL Slab. Further opening-up is thus required to confirm all top layer(s) of couplers of the top mat were replaced with through steel bars. In particular, some opened-up locations should be placed on the top surface of EWL Slab west of D-Wall (East) as shown in Figure 6 to confirm whether Type 2 or Type 3 connections were actually constructed.

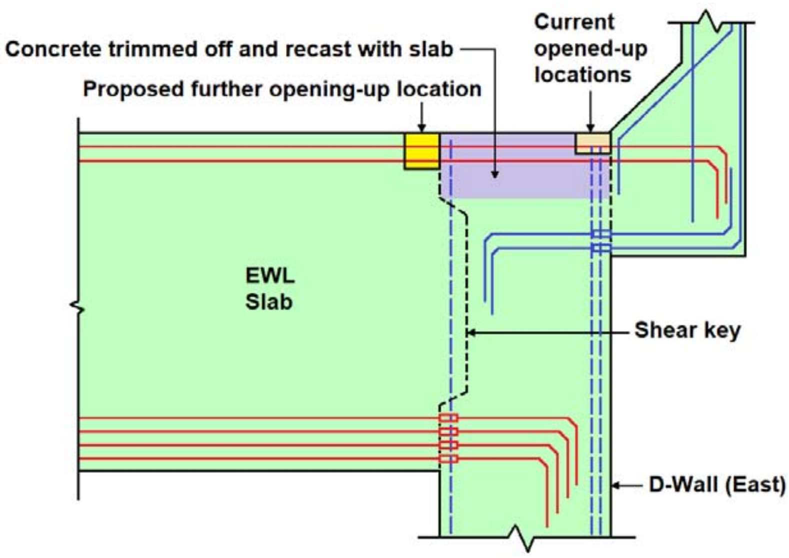


Figure 6. Proposed location for further opening-up

126. The confirmation that the top reinforcement steel bars are through bars and not laps between bars at the proposed location is of paramount importance to the structural adequacy of the EWL Slab. If there are laps between reinforcement steel bars, they probably cannot comply with the requirements of CoP 2013 on laps as elaborated below.

127. Clause 8.7.2 "Laps" of CoP 2013 provides that

"*The detailing of laps between bars shall be such that:*

(a) the transmission of the forces from one bar to the next is assured;

(b) spalling of the concrete in the neighbourhood of the joints does not occur; and

(c) large cracks which affect the performance of the structure do not occur.

Laps between bars should normally be staggered and not located in areas of high stress. Laps at any one section should normally be arranged symmetrically. At laps, the sum of the diameters of all the reinforcement bars in a particular layer should not exceed 40% of the breadth of the section at that level.

The arrangement of lapped bars should comply with figure 8.4:

(d) the clear transverse distance between two lapping bars should not be greater than 4ø or 50 mm, otherwise the lap length should be increased by a length equal to the clear space exceeding 4ø or 50mm;

(e) the longitudinal distance between two adjacent laps should not be less than 0.3 times the lap length, l_o; and

(f) in case of adjacent laps, the clear distance between adjacent bars should not be less than 2ø or 20 mm.

The permissible percentage of lapped bars in tension at any section may be 100% where the bars are all in one layer, or 50% where the bars are in several layers.

All bars in compression and secondary (distribution) reinforcement may be lapped in one section.

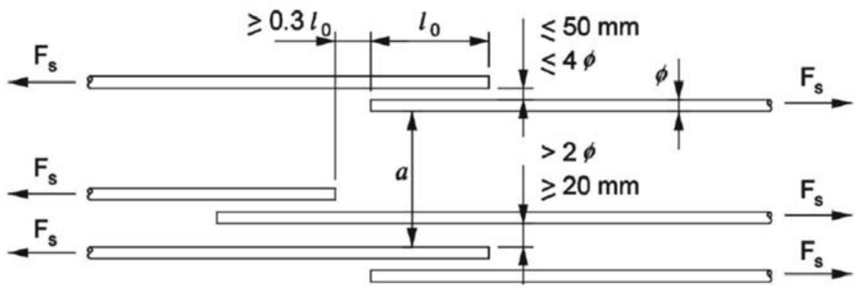


Figure 8.4 - Adjacent laps"

128. In accordance with the requirements of clause 8.7.2 of CoP 2013, laps should not be located in the areas of high stress. As the EWL Slab is connected to D-Wall (East) as a fixed joint, the top reinforcement steel bars at the proposed location of further opening-up are in high tensile stress when the EWL Slab is supporting heavy dead loads and imposed loads. Therefore, laps between reinforcement steel bars should not be placed there as in the original design. This important aspect regarding structural adequacy has to be confirmed.

129. If it is absolutely necessary to place laps between reinforcement steel bars at the suggested location of open-up, which I disagree, the laps between steel bars should be staggered and the arrangement of lapped bars should comply with figure 8.4 of CoP 2013. Therefore, the width of the open-up should at least cover two sets of reinforcement steel bars to inspect whether the laps between steel bars are staggered, if there are any lapped bars.

130. If there are laps between the 40-mm diameter reinforcement steel bars at the proposed further opening-up location, the sum of the diameters of all the reinforcement bars in a particular layer may exceed 40% of the breadth of the section at that level. The spacing between reinforcement steel bars as observed in the opened-up locations is approximately 150 mm. If the laps between reinforcement steel bars are staggered, there are three (3) 40 mm diameter bars within a breadth of 300 mm on average. The sum of all the diameters of all the reinforcement steel bars would be 3×40 = 120 mm. As 120/300 = 40%, it can barely meet the requirement of clause 8.7.2 of CoP 2013. If there are laps between reinforcement steel bars at the proposed further opening-up location and they are not staggered, there would be four (4) bars within a breadth of 300 mm on average. The sum of all the diameters of all the reinforcement steel bars would be 4×40 = 160 mm. As 160/300 = 53%, it fails to comply with the requirement of clause 8.7.2 of CoP 2013.

131. The top reinforcement steel bars of EWL Slab are in tension. As there are at least two (2) layers of reinforcement steel bars, the permissible percentage of lapped steel bars at any section should be only 50%. If all the reinforcement steel bars exposed at the

suggested open-up locations are lapped bars, they were not installed in compliance with the requirements of clause 8.7.2 of CoP 2013.

132. Both Type 2 and Type 3 modifications required the trimming off of the top of D-Wall (East) and recasting the concrete with EWL Slab, resulting in the formation of a horizontal construction joint in D-Wall (East). It should be noted that horizontal construction joint does not exist in the original D-Walls due to the construction procedure of D-Walls, as D-Wall (East) was casted under bentonite suspension and concreting had to be completed continuously. As stipulated in Item 3 of the Joint Expert Memorandum, the internal stresses at the new horizontal construction joint in D-Wall (East) in the modifications should be reviewed for structural adequacy to the satisfaction of the BA of the HKSARG.

133. I was informed that the amount of reinforcement steel actually installed into the top mat as a result of the modifications is more than that in the original design. The validity of the information has to be verified on site. The locations, concrete covers and diameters of all the top layer reinforcement steel bars in the top mat of the EWL Slab can be easily determined by a concrete cover meter and rebar detector. The actually amount of reinforcement steel, locations of reinforcement bars and concrete cover to reinforcement steel bars can be evaluated as another indicator of the quality of workmanship in the installation of reinforcement steel bars.

Defects in Concrete of the Substructure

134. It has been revealed that there are defects in the reinforced concrete substructure, such as honeycombs, exposure of reinforcement steel bars, non-compliant of shear links, spalling, voiding etc. However, the Expert Panel did not have an opportunity to inspect any of these defects during the two site visits, i.e. the site visits on 17th and 19th December 2018. Therefore, I cannot comment specifically on the conditions of these defects. In general, these defects are probably repairable as stated in the Item 4 of the Joint Expert Memorandum.

WAY FORWARD

Structural Design of the NSL Slab

135. The NSL Slab is founded on *in-situ* geologic materials. However, a gap was assumed underneath the slab in the design, i.e. the NSL Slab was not founded on *in-situ* materials. While the assumption may be conservative for some loading combinations, it may not be conservative in some other loading combinations. For example, hogging moments are induced in the NSL Slab at the connection with the D-Walls by the dead loads exerting on the slab if there is a gap underneath. Sagging moments are induced at the same location by the upward flotation forces exerted by groundwater. In the loading combination of dead loads and flotation forces, the sagging moments induced by the flotation forces are reduced by the hogging moments induced by the dead loads. The design may not be conservative as the hogging moments may not exist due to the ground support provided by the underlying *in-situ* geologic materials, resulting in the underestimation of the sagging moments in the NSL Slab. The structural adequacy of the design should be reviewed in this aspect.

Structural Integrity of the As-constructed Structures

136. It is evident that some of the connections of reinforcement steel bars using couplers cannot meet the manufacturer's requirements to be fully functional.

137. It is evident that some of the threaded sections of the Type A and Type B reinforcement steel bars are shorter than the design values of 44 mm and 88 mm, respectively.

138. The connection details between the EWL Slab and D-Wall (East) were not fully inspected to confirm the as-constructed details are the same as the modified design. Further opening-up are required, in particular at proposed locations depicted in Figure 6.

139. The connection details between the NSL Slab and D-Walls might not be adequate as elaborated previously.

140. The strength of the reinforcement steel bars may be reduced as the force transmitted through the couplers has been reduced due to inadequate embedment length. The magnitude of such reduction can be determined by laboratory testing.

141. In view of the redundancy and conservatism already incorporated in the original design, it may be possible for the MTRCL to establish the structural adequacy of the as-constructed structures using the results of the investigation, more realistic design parameters, and the reduced design strength of the reinforcement steel bars.

Sampling Strategy

142. In accordance with the MTRCL Proposal of 4th December 2018, twenty-eight (28) random locations are selected for the EWL Slab and the NSL Slab and three (3) couplers are randomly selected at each location, resulting in the selection of a total of one hundred and sixty eight (168) sampling locations for couplers.

143. However, all the couplers evaluated to date are either at the top layer or the bottom layer of reinforcement steel bars in the EWL Slab or the top layer of reinforcement steel bars in the NSL Slab.

144. In response to the questions raised by the Honorable Tien, Michael Puk-sun in the meeting of the Subcommittee of 7th December 2018, Dr. Jacob Kam of the MTRCL confirmed that selected couplers on inner layers of reinforcement steel bars would be evaluated even if the opening-up of the platform slabs would be deeper than 500 mm. The implementation of his confirmation at the Subcommittee meeting remains to be seen.

145. The sampling methodology has not been revealed to the public by the HyD of the HKSARG to ease public anxiety.

Statistical Inference

146. The methodology of statistical inference from the test results of the samples has not been revealed by the HyD of the HKSARG. However, the maximum failure rate in the population estimated on the basis of the number of failures in the samples has been published.

147. The failure rate in the population is estimated from the failure rate in the samples by statistical inference. As the number of samples is considerably smaller than the size of the population, there would be a margin of error in the estimation due to sampling. The margin of error can be estimated from the maximum failure in the population published by the HyD of the HKSARG and the probability of failure estimated by the results of sample testing. The results of analysis of the statistical inference published by the HyD of the HKSARG reveal the followings:

Total sample number = 84

Total no. of failures in the samples (1)	**Max. failure rate in the population** (2)	**Probability of failure in the sample**[1] (3)	**Margin of error**[2] (4)
0	3.5%	0.00%	3.50%
1	5.5%	1.19%	4.31%
2	7.3%	2.38%	4.92%
3	9.0%	3.57%	5.43%
4	10.6%	4.76%	5.84%
5	12.1%	5.95%	6.15%

Notes:
[1] Column (3) = Column (1) / 84 × 100%
[2] Column (4) = Column (2) – Column (3)

148. Statistical inference is not within my areas of expertise. However, as the sample size is less than 1% of the population, the margin of error appears to be very small. For example, if the rule of thumb of $1/\sqrt{n}$ is used, the margin of error is approximately $1/\sqrt{84} = 10.9\%$.

149. It may be helpful to ease the anxiety of the public if the HyD of the HKSARG can reveal the methodology of statistical inference to gain the confidence of the general public on the rigor of the statistical analysis. In fact, the Honorable Chan, Han-pan asked for the details of sampling and statistical inference of the sampling results in the meeting of the Subcommittee of 7th December 2018. Moreover, at the same meeting, the chairman of the Subcommittee, the Honorable Yick, Frankie Chi Ming, requested the administration to release such details in the next meeting of the Subcommittee in two months' time.

150. All the test data available to date are from either the top layer or bottom layer of reinforcement steel bars in EWL Slab and the top layer of reinforcement steel bars in the NSL Slab. As some of the test samples are not the part of the one hundred and sixty eight (168) samples originally selected, the sample size in the analysis may have to be adjusted to accommodate the increased sample size.

151. The statistical criteria published by the HyD of the HKSARG should be extended as the number of failures in the tested samples from the EWL Slab has already exceeded five (5).

CONCLUSIONS

152. Ductility couplers or Type 2 mechanical couplers have been adopted in accordance with the QSP of the project and CoP 2013. The strength requirements, both tension and compression, for the coupler assembly should follow the details of the QSP, and the guidelines of CoP 2013 and CS2:2012. The embedment length required should follow the manufacturer's installation instructions.

153. It is evident from the results of the investigation to date that some reinforcement steel bars connecting the platform slabs to the D-Walls do not have adequate embedment lengths in the ductility couplers in accordance with the manufacturer's installation instructions.

154. The maximum rate of failure of the couplers in the population of the EWL Slab in accordance with the statistical criteria published by the HyD of the HKSARG exceeds 12.1% in EWL Slab, as the number of failures in the tested samples exceeds five (5) regardless of the failure criterion adopted.

155. The maximum rate of failure of the couplers in the population of the NSL Slab in accordance with the statistical criteria published by the HyD of the HKSARG is 5.5% or 10.6% in the NSL Slab depending on the failure criterion adopted, as the number of failures in the tested samples is one (1) or four (4) as of to date, assuming the results of all the remaining tests will satisfy the embedment length requirement.

156. Eight (8) to nine (9) threads of reinforcement steel bar were exposed behind one of the couplers. Careful inspection of the threads reveals that the threads might be slightly damaged, rendering it very difficult to penetrate the reinforcement steel bar into the coupler to the required embedment length.

157. The lengths of the threaded sections of some Type A reinforcement steel bars are definitely shorter than 44 mm while those of some Type B reinforcement steel bars are definitely shorter than 88 mm. It is highly likely that the threaded sections of these reinforcement steel bars have been cut short although the cut threaded sections could not be found.

158. The as-constructed connection details on the top of D-Wall (East) have been modified by Leighton without prior approval. Whether the as-constructed connection details of reinforcement steel bars follow the modified designs are yet to be fully investigated. Additional opening-up locations on the top surface of the EWL Slab are proposed. The structural adequacy of the modification is subject to review and acceptance by the BA of the HKSARG,

159. The defects in the concrete of the structures have not been inspected by the Expert Panel. No specific comment on the conditions of these defects can be made.

160. The MTRCL may consider the use of more rigorous and sophisticated structural analyses for the as-constructed structures using the results of the investigation, more

realistic design parameters, and reduced design strength of reinforcement steel bars to evaluate the structural adequacy of the as-constructed structures.

161. The statistical inference for the maximum failure rate of couplers in the population should be extended as the number of failures in the tested samples has already exceeded five (5).

162. The methodologies of sampling and statistical inference adopted by the MTRCL should be released to the public to ease their anxiety on the rigor of the statistical analysis.

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7th January 2019

EXPERT WITNESS'S DECLARATION OF DUTY TO THE COMMISSION OF INQUIRY

I, YEUNG, Tak Chung Albert, declare that

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

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

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

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7th January 2019