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Consulting Engineers

MTRCL Shatin to Central Link Contract 1112

Hung Hom Station & Stabling Sidings

Commission of Inquiry

Original Hearing

Structural Engineering Expert Report

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1. Introduction

Leighton Contactors (Asia) Ltd ("LCAL") is constructing Contract SCL 1112, Hung Hom Station and Stabling Sidings, which forms part of the new Shatin to Central Link ("SCL") railway being constructed for the Mass Transit Railway Corporation Ltd ("MTRCL").

In respect of the diaphragm walls and platform slabs at the Hung Hom Station, a Commission of Inquiry ("COI") was established to inquire into the facts and circumstances surrounding the steel reinforcement fixing works and any other works which raise concerns about public safety and to ascertain whether the works were executed in accordance with the Contract.

In January of 2019, the remit of the COI was further extended by Government to consider other areas of the Project, namely the North Approach Tunnels, the South Approach Tunnels and the Hung Hom Stabling Sidings and other process related elements of the construction work carried out by LCAL. Hearings for the extended scope of the COI commenced in June 2019.

In the interim, an investigation process has been undertaken from December 2018 to April of 2019 to open up certain parts of the platform slabs which were the subject of the original hearings. The purpose was to: (i) verify the as constructed condition of certain areas of the slabs; and (ii) randomly test whether the steel reinforcement bars had been properly connected to the couplers embedded in the diaphragm wall.

In July 2019, the MTRCL issued the Final Report on Holistic Assessment Strategy for the Hung Hom Station Extension ("Holistic Report"),[1] which proposes "suitable measures" to be carried out to the Works.

2. Instructions

I have been retained by O'Melveny and Myers (Counsel for LCAL) to provide my expert opinion on the following three principle items and the list of pertinent issues associated with each (as listed below). I have set out below the relevant sections of my report that address these issues.

2.1. Coupler connections / coupler engagement

(a) For structural safety purposes, what is the required minimum engagement length of threaded rebar into couplers? In particular, should the minimum engagement length (taking into account the allowable tolerance measurement of 3mm) be at least 37mm as set out in Section 3.3.13 of the Holistic Report?

Please refer to sections 6.1-6.10 below

(b) Based on the required minimum engagement length, what conclusions can be made about the number of defective coupler connections arising from the "Results of PAUT/Direct Measurement" in Appendix B3 of the Holistic Report?

Please refer to sections 6.10-6.11 below

[1] [OU5/3229ff]

(c) Does the number of defective coupler connections have any impact on the structural integrity of the as-built works?

Please refer to Sections 6.6-6.7 below

(d) Are the proposed suitable measures in Appendix C5 of the Holistic Report necessary to ensure that the as-built works are structurally safe? In answering this question, please consider and comment (as necessary) on the Updated Design referred to in Section 4.3 of the Holistic Report.

Please refer to Sections 6.10-6.12 below

2.2. Shear link reinforcement and partial utilisation of shear

(a) What are your comments on the shear link investigation referred to in Sections 3.5.27 to 3.5.28 and Appendix B8 of the Holistic Report?

Please refer to Sections 7.1-7.2 below

(b) What conclusions (if any) can be drawn based on this shear link investigation?

Please refer to Sections 7.3-7.8 below

(c) Are the proposed suitable measures in Appendix C6 of the Holistic Report necessary to ensure that the as-built works are structurally safe? In answering this question, please consider and comment (as necessary) on the Updated Design referred to in Section 4.3 of the Holistic Report.

Please refer to Section 7.9 below

(d) What works (if any) should be performed in order to address any structural or other concerns arising from the investigation of the shear links?

Please refer to Section 7.9 below

2.3. Construction joint

(a) What are your comments on the results of the inspection of the horizontal construction joint at D-Wall panels EH69 and EM94 in Areas C1 and C2 as referred to in Section 3.5.34 of the Holistic Report?

Please refer to Section 8.2 below

(b) What conclusions (if any) can be drawn from these results?

Please refer to Sections 8.3-8.4 below

(c) Are the proposed suitable measures in Appendix C7 of the Holistic Report necessary to ensure that the as-built works are structurally safe? In answering this question, please consider and comment (as necessary) on the Updated Design referred to in Section 4.3 of the Holistic Report.

Please refer to Sections 8.5-8.7 below

(d) What works (if any) should be performed in order to address any structural or other concerns arising from the construction joint?

Please refer to Section 8.7 below

3. Expert's declaration

I understand that my primary duty in preparing this report and giving evidence is to the COI, rather than to the party who engaged me and I have complied with that duty.

I have endeavoured in this report and in my opinions to be accurate and to have covered all relevant issues concerning the matters stated which I have been asked to address.

I have endeavoured to include in my report those matters, which I have knowledge of or which I have been made aware, that might adversely affect the validity of my opinion.

I have indicated the sources of all information that I have used.

I have not, without forming an independent view, included or excluded anything which has been suggested to me by others (in particular my instructing solicitors).

I understand that:

- My report, subject to any corrections before swearing as to its correctness, will form the evidence to be given under oath or affirmation.
- I may be cross examined on my report by a cross examiner assisted by an expert.
- I am likely to be the subject of public adverse criticism if the COI concludes that I have not taken reasonable care in trying to meet the standards set out above.

I believe the facts I have stated in this report are true and that the opinions I have expressed are correct.

4. My First Report

In January 2019, I gave evidence to the COI, that can be summarised as follows:

- That after the construction of the diaphragm walls ("D-Wall"), LCAL constructed the connection between the platform slab and the eastern D-Wall using an improved detail that provided superior strength and robustness.
- The change of detail was compliant with all the relevant design codes used for the design of the station structure, and the resulting structure is now stronger and more robust than the original accepted detail.
- That the preliminary testing of the bar coupler assemblies carried out at the CASTCO testing centre on 21 November 2018 demonstrated that a bar coupler which had 26.4mm embeddment of the connecting bars provided the coupler with sufficient strength to resist the ultimate design loading in the bars used in the design of the as constructed works ("Works").
- That regardless of what has occurred during construction, the redundancy in the structural design of the works meant that the Works continue to be a safe design, suitable for its designed use.

At the time I gave this evidence in January 2019 the opening up investigation had just started and therefore its full results could not have been considered by the COI.

5. Structural assessment of Works

With the intense scrutiny of this Project, it is not surprising that a considerable number of consulting engineering companies ("Consultants") have been commissioned to carry out structural engineering checks on the design of the station to verify the original structural design.

5.1. Consultants involved

The Consultants involved are as follows:

- Atkins China Limited ("Atkins"), the original designer of the structure, commisioned by MTRCL to carry out the original design of the structure and the subsequent assessment reports of the as constructed condition of the station structure.
- Ove Arup and Partners (Hong Kong) Ltd ("Arup"), commissioned by MTRCL in Autumn 2018 to carry out a 3 stage holistic verification study of the as-constructed condition of the station structure.
- Aecom, Hong Kong office ("Aecom"), commissioned by MTRCL in January 2019 to carry out an independent design review and structural assessment of the station structure.
- Cowi, United Kingdom office ("Cowi"), commissioned by LCAL to carry out an independent design review and structural assessment of parts of the EWL slab of the structure.
- CEEK Limited ("CEEK"), a Hong Kong based consultant commissioned by LCAL to independently analyse Area A of the EWL NSL and Mezzanine Slab.
- EIC Activities Pty Ltd ("EIC"), an Australian based consultant commissioned by LCAL to review and assess the findings of the Holistic and Verification Reports.[2]

I have reviewed the reports of these consultants[3] for the purposes of giving my expert opinion on the relevant issues. However, except as mentioned below, I have not carried out the same calculations myself and cannot confirm the accuracy of those calculations.

5.2. The benefit of independent structural checking

Industry norm is that for complicated structures a consulting engineering company would perform the structural design and then an independent consultant would carry out an independent structural design check. The check by the independent consultant is deemed to provide sufficient comfort that any errors in the structural design would be caught and that the design would perform satisfactorily when constructed. The fact that this industry process

[2] I understand that EIC is an engineering consulting company that is owned by the same ultimate corporate group as LCAL.

[3] The Stage 3 Assessment Reports prepared by Atkins [OU6/3942ff], Ove Arup [OU6/8579ff] and AECOM [OU6/9680-9681]; EIC Report dated 23 August 2019 [OU7/9743ff] with attached CEEK Report in Appendix A; EIC Report dated 29 August 2019 [OU7/9829ff]; EIC Report dated 30 August 2019 [OU7/9838ff]; EIC Report dated 23 September 2019 [OU7/10020ff]]; CEEK Reports attached to letter from LCAL to MTRCL dated 27 September 2019; CEEK report submitted by LCAL to MTRCL on 18 October 2018.

works is evidenced by the fact that structures when constructed do not fail, collapse or are required to be demolished and rebuilt, except in very rare cases.

In my experience, engineering consultants, when acting as independent checking engineers, can be very diligent and thorough in their checking of designs. I have noted that independent checking engineers tend to deliberately look for the worst in other people's engineering; an unfortunate character trait in many other social interactions, but one that is reassuring in present context, as this ensures that any errors or design misconceptions are very likely to be identified.

5.3. Findings of the Consultants

Five separate and independent companies have carried out structural analysis and checking of the station structures, and all typically reach the same conclusions, that the design is safe and is over-provided by a considerable margin. That is, they conclude that there is a substantial amount of spare structural capacity in the Works.

5.4. Concrete strength of slabs

The design of the station was carried out on the basis of grade 45D/20 concrete, i.e. the 28 day characteristic strength of the concrete would be 45 MPa. The characteristic strength is defined as that strength below which not more than 5% of test results of the concrete strength are expected to fall.

The strength of concrete used on construction sites is usually more than the design strength for the following reasons:

- So that the concrete suppler can be sure that the concrete supplied will meet this criteria, i.e. when the concrete samples are tested on site at 28 days old strength tests will pass.
- The pressure of construction programmes often means that a high early age strength is needed so that the formwork can be stripped and the concrete becomes self-supporting as soon as possible after its pouring. If a concrete achieves a high early age strength, it normally means that the concrete will achieve a greater strength at 28 days than its required 28 day characteristic strength.

EIC Activities have carried out an assessment of the as constructed strength of the concrete used in the construction of the station structure and this is presented in EIC's Shear Report attached to LCAL's letter to the MTRCL of 30 August 2019.[4]

The actual 28 day concrete strength (after consideration of the standard deviation) is a minimum of 48.8 MPa in the NSL Area A slab. In other areas it is typically 60 MPa, but is 70MPa in the EWL Area A slab. Refer to Appendix B for the summary of the concrete strengths extracted from EIC Shear Report.

[4] [OU7/9837ff]

5.5. Bending strength of slabs

All of the Consultants' reports show that the bending strengths of the platform slabs are significantly in excess of the design requirements.

Of particular interest however is the EWL slab in Area A. As a result of the opening up investigations and the statistical analysis carried out by MTRCL, a large reduction factor of 65% has been applied to the strength of the concrete section at the location of coupled connections in the slabs in this area.

EIC have reviewed the calculations of Atkins, Arup, Aecom and Ceek in this area. The results of this analysis have been extracted in Appendix C.[5] These show a large variance in the percentage utilisations of the concrete section in similar areas, but of note they all demonstrate that the design of the platform slabs are safe and code compliant.

Refer to section 6.10 below for my comments on the couplers in this area and the use of the reduction factor.

5.6. Shear strength of slabs

5.6.1. Arching action

Arup has presented a calculation in their Appendix C of Volume 7 of their Stage 3 Assessment Report which demonstrates the arching action of the platform slabs. Arching is a real effect whereby load that is applied to the slab is transferred to its ends (i.e. the D-Walls) by the establishment of a line of thrust within the depth of the slab – shown below in Figure 1 the black line superimposed on an extract of the Arup sketch in their calculation section C2.4.

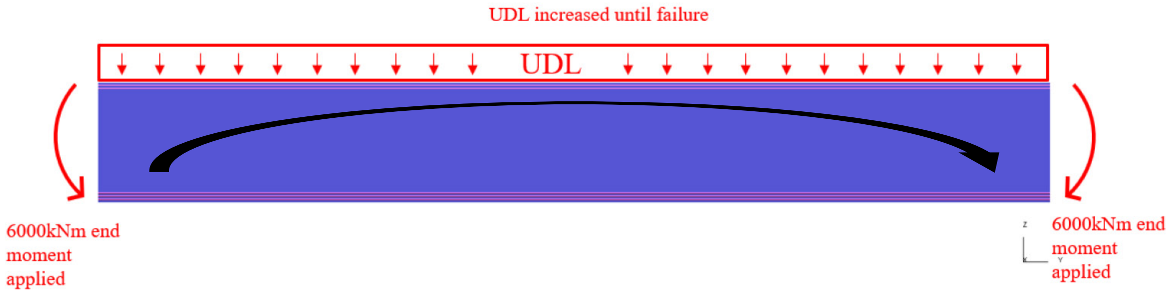


Figure 1 – Development of Arching action in concrete slab

Arup's non-linear computer analysis modelled the effect of the establishment of the arch by applying load to the concrete slab until such time that the model showed the slab had failed. The results showed that the slab is able to withstand two to three times the amount of load that the slab had been designed for using the methods in the HK Code of Practice for the Structural Use of Concrete 2004 version ("HKCOP"). In other words, the actual shear capacity of the slab proves to be at least twice the shear capacity of the slab when calculated using the shear capacity calculation methods specified in the HKCOP.

[5] I understand that LCAL will submit this analysis by EIC to MTRCL shortly and it will be disclosed to the COI in due course.

It is important to note that this analysis did not include the beneficial effect of the shear links in the slab – the only reinforcement considered was top and bottom longitudinal steel as per Figure 2 below.

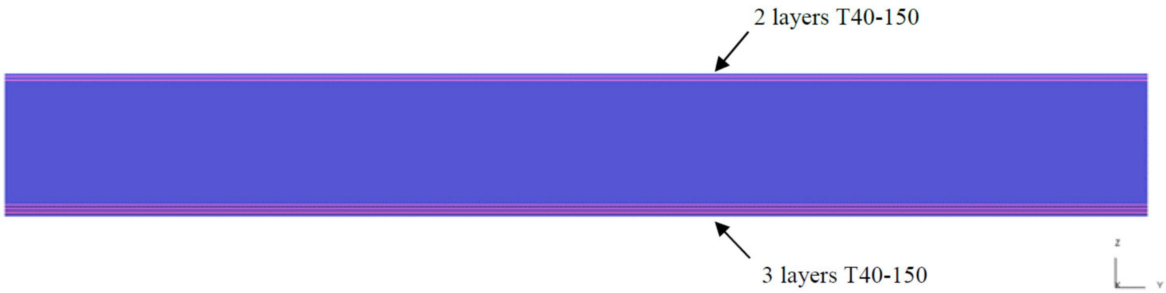


Figure 2 – Reinforcement in Concrete Model

Therefore, the analysis shows that there is two to three times the reserve of shear strength in the slab when compared to what is achieved when a design is computed using the codified methods in the HKCOP.

This inherent reserve of strength provides an additional safety factor, providing further re-assurance that the structure is safe for use.

5.6.2. Atkins shear calculations

The Holistic Report appears to determine the extent of the "suitable measures" to be implemented solely upon the Stage 2 calculation findings of Atkins. I have reviewed their shear calculations contained in the Atkins calculation reports and find that they are conservative for the following reasons:

- In their determination of v_c, the concrete shear stress capacity, they have not used the correct tensile steel area;
- The beneficial contribution of the axial load compressing the slabs has not been considered; and
- The actual concrete strength of the as constructed slab has not been considered. The slabs were designed for Grade 40 concrete, but LCAL have supplied a concrete that has a statistical characteristic strength that is significantly higher (i.e. typically 60MPa with a low of 48.8 MPa in the NSL Area A slab and a peak of 70MPa in the EWL Area A slab)..

I do not therefore consider the Atkins conclusions to be realistic and representative of the structure that has been constructed [redacted]

5.6.3. Revised calculations

Appendix D of my report shows the plans of the areas of the platform slabs where it is my understanding that "suitable measures" are required to account for the perceived concern over the as constructed shear links. The plans show the areas of the slabs where Atkins' calculations assess that minimum shear links are required.

EIC have reviewed these calculations and corrected them to include all three items above in sequential order.[6] The base data remains unchanged and the only part of the Atkins calculation that was revised was the calculation for v_c, the concrete shear capacity, based on the corrected input.

The results of EIC's work are given in Appendix D. A summary of the relevant calculations is presented below. [7]

Location Referenced to the Areas Design Strip			Utilisation Ratios			
Strip	X-Cor	Y-Cor	v*/vc - Atkins	v*/vc - With Actual Steel	v*/vc - With Axial Compression	v*/vc - With actual Strength
Y0	1.295	21.62	1.017	0.736	0.640	0.531
Y0	1.295	21.77	1.109	0.806	0.696	0.578

The utilisation ratios of the above areas have been re-calculated progressively considering the above key missing factors. Red colour shows the result of a calculation which fails (i.e. where the applied shear stress from loading is greater than the concrete shear capacity and thus minimum shear links are required), green colour shows the result of a calculation which passes when the above factors are included (i.e. where the applied shear stress from loading is less than the concrete shear capacity and thus no minimum shear links are required).

The EIC calculations prove that only 2.5m² of platform slab require minimum shear links, out of a total of 23,647m² of slab. This is 0.01% of the total.

However, in my opinion, these areas have already been constructed with a satisfactory amount of minimum shear links which are code compliant. Please refer to section 7 below.

5.7. Strength of D-Walls

All of the Consultants have agreed that there is no concern in relation to the safety, code compliance or as-constructed condition of the D-Walls.

5.8. Strength of D-Wall / EWL connection

The East D-Wall / EWL slab construction joint has been analysed and structurally checked by Atkins, Arup and Aecom. For details of this joint, please refer to section 7.4.5 of my previous report of January 2019.

Atkins, Arup and Aecom have used a variety of methods, all of which confirm that the design of the as constructed joint will perform as intended and there is no concern in relation to structural safety or code compliance.

[6] [OU7/9838ff].
[7] I understand that LCAL will submit this analysis by EIC to MTRCL shortly and it will be disclosed to the COI in due course.

These three Consultants all demonstrate that the reservations expressed by Professor Au in January 2019 were unfounded.

It is my understanding that the design of this joint has now been accepted by Government, and is no longer considered to be of concern.

The Holistic Report does however express reservation about the as constructed condition of this joint, and on the basis of these reservations, is recommending “suitable measures” be taken in order to strengthenf the connection. I have addressed this in section 8 below.

6. Coupler connections and engagement

6.1. MTRCL testing of coupler assemblies

The MTRCL carried out two sets of comprehensive tests on partially engaged coupler assemblies. The first in February 2019, which after comments from Government were repeated in April 2019. [8]

These tests considered the partial engagement of threads on one side of the coupler of the following arrangements:

- 6 threads.
- 7 threads.
- 8 threads.

Static tension tests and cyclic tension tests were carried out as well as measurement of the permanent deformation of the coupler at the completion of the static tension test.

6.2. At the time of construction, what did the construction team understand was the requirement of embedment length from BOSA

Much has been heard during the COI of BOSA's requirements for coupler installation. It was suggested that a "butt to butt" connection of the embedded bars inside the coupler was needed in order for the coupler to perform satisfactorily.

However, there is no evidence that LCAL was aware or should have been aware of a "butt to butt" requirement during the actual construction period.

6.2.1. BOSA's material submission

The factual evidence given to the COI is that, based on contemporaneous documents provided by BOSA,[9] the typical threaded end of the rebar for the Project was 44mm with 11 threads (each thread of 4mm).

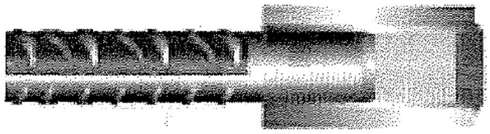
I note that other structural engineering experts in their oral testimony given to the COI, and and the MTRCL in their recent reports on the testing of the couplers, have indicated that a typical rebar for the Project has 10 threads on the assumption that the 44mm threaded ends should be discounted by half a full thread (i.e. 2mm) and by 2mm for a chamfer (i.e. 4mm should be discounted). This is inconsistent with the factual evidence before the COI. I have therefore disregarded such opinion evidence.

I have re-reviewed the original materials related submission form prepared by BOSA and submitted by LCAL to MTRCL on the 28th June 2013, document 1112-MSF-LCA-CS-000005. This document was prepared by BOSA for LCAL to submit to MTRCL and provides the necessary material submission information for MTRCL to be able to give approval for the use of the

[8] For the February 2019 tests see [OU2/907.29-907.65] and April 2019 tests see [OW1/230 – 290].
[9] See [H25/44527.1] BOSA Seisplice System Thread Strength Calculation and [A1/A575] BOSA Technical and Quality Assurance Manual Type A.

couplers in the Works. It also provides a method statement for the use of the couplers and their installation.

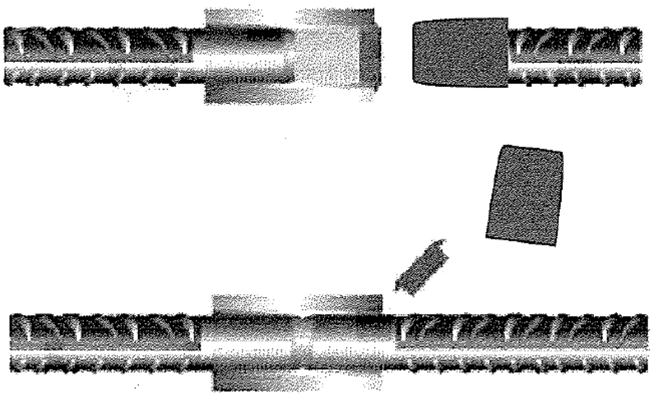
Although this evidence has already been presented, I remind the COI of the installation instructions as follows[10]:



STEP 1 : Position the 1st stage bar

Ensure the Coupler is fully screwed into the bar prior to being cast in concrete.
Protective cap should be fitted on Coupler end to prevent ingress of foreign material.

BOSA instruct that the coupler is to be fully threaded onto the parent bar – the parent bar being the bar that has a threaded end located at a construction joint in any particular concrete pour.



STEP 2 : Connect the continuation bar

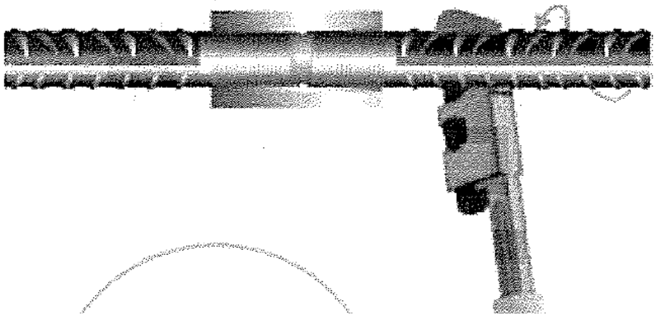
Position the continuation bar.

Remove both protective cap on the Rebar and the coupler

Fully engage the thread using hand to the coupler. This should develop full tensile strength of the splice once fully engaged

BOSA then instruct that the continuation bar is to be installed by "fully engage the thread using hand to the coupler". This English used for this instruction could be better, but my interpretion is that bar should be screwed into the coupler by hand, and in doing so, the by hand operation will fully engage the thread.

[10] These installation illustrations are taken from the BOSA Method Statement of 28th June 2013, document 1112-MSF-LCA-CS-000005 pdf page 116 for the Type II ductility coupler.



STEP 3 : Lock the splice

Use a typical pipe wrench to tighten the splice.

No special torque amount is required.

BOSA finally instruct that the splice should be tightened and do not require any special torque to do so.

6.2.2. No butt to butt requirement

There is no explicit instruction to screw the continuation bar into the coupler so that its end is touching the parent bar, i.e. to make the two bars "butt to butt".

Neither is there any wording that could possibly be contrived to be such an instruction.

It is possible that someone might interpret the words "fully engage the thread" to mean that all of the threads of the continuation bar should be engaged with the internal threads of the coupler, but actually this could not have occurred, as explained below.

My own interpretation of the meaning of the words is that the continuation bar should be screwed in as far as it can be made to go. This is consistent with the instruction to "fully engage the thread using hand" and to then "use a typical pipe wrench to tighten the splice". Notably, it states that "no special torque amount is required". If these instructions were followed faithfully, it would be sufficient if the bars was screwed in as far as possible by hand and tightened with a typical wrench. There is no instruction that all threads need to be screwed in or a "butt to butt" connection be achieved.

It is now clear from the evidence that the threaded length of the bars varied. [11] That is, BOSA did not produce threaded ends of exactly 44mm. That is, the threaded ends of bars may have been more or less than 44mm or 11 threads. The interior of the couplers provided 88mm of space for the bars to be screwed in from either end. [12] On that basis, and as a matter of simple arithmetic, it is clear that many bars would not have been able to achieve a "butt to butt" connection. For example, if two bars of less than 44mm were screwed in from opposite ends of the same coupler, it would be impossible to achieve a "butt to butt" connection. The same applies if one bar has a threaded end of 44mm and the other has a threaded end of less than 44mm. In this context, it is entirely reasonable to interpret the instructions in BOSA's material instruction to mean that the bar should be screwed in as far as possible by hand and tightened by a wrench. It would make no sense if BOSA's instructions are interpreted to impose a

[11] The PAUT tests and direct measurements of rebar confirm that many bars were not 44mm. It is clear from the results in Appendix B3 of the Holistic Report [OU5/3309ff] that the threaded ends of the bars tested as part of the opening up exercise were of varying lengths. Some of the tested bars were less than 40mm. Very few of them were 48mm or more. No threaded ends were longer 52mm or more.

[12] See section 15.4 of my First Report of January 2019.

requirement that may not have even been possible to achieve for many of the bars that BOSA threaded on site.

Evidence has already been presented of the difficulty of manually lifting, aligning and rotating heavy 40mm diameter bars so that the end can be screwed into a coupler. This practical reality lends itself to interpreting BOSA's instructions to mean (as I have suggested above) that the bars are screwed in as far as psosible by hand and then tightened with a wrench (even if this does not achieve "butt to butt" connection). These practicalities would have been known to BOSA when they set out the instructions. If BOSA actually intended a "butt to butt" connection, they should certainly have made this clear in their instructions in the material submission.

6.2.3. BOSA "how to measure thread length document"

As previously presented in evidence, BOSA produced another instruction manual,[13] which LCAL relied upon for their installation works, that specifies BOSA's method to visually inspect that the couplers have been installed correctly. This states that a tolerance of zero to a maximum of two threads should be visible on the side of the continuation bar. As each thread is typically 4mm in length, this allows for around 8mm of thread for each bar to be exposed out of the coupler.

This document makes no mention of a "butt to butt" requirement. As explained above, a threshold of two exposed threads (i.e. around 8mm) would mean that a "butt to butt" connection would rarely be achieved using the bars threaded by BOSA on the Project. For example, if two bars with threaded ends of 44mm were screwed in and two threads were exposed for each, there would be a gap between the bars of 8mm. It follows that both bars would need to have threaded ends of at least 52mm in order to achieve a "butt to butt" connection with two threads exposed for each bar. Notably, the results of the testing of the coupler connections (as set out in Appendix B3 of the Holistic Report) indicate that none of the bars tested had threaded ends with a length of at least 52mm.[14] On that basis, it is clear that BOSA's specified threshold of two exposed threads is not consistent with, and cannot support, a "butt to butt" requirement.

6.2.4. What did BOSA actually intend?

With the benefit of hindsight and BOSA's revelation to the COI that the connections should be "butt to butt", I can understand what perhaps BOSA was intending to achieve with their threshold of "max 2 visible threads" (as set out in the manual provided to LCAL). BOSA has stated that the fabrication tolerance on the threaded ends of bars can be up to 1 extra thread, i.e. 12 threads or a threaded end of 48mm in length). If both the parent bar and continuation bar have 12 threads, and the coupler is fully screwed on to the parent bar as far as it can go, then it necessarily stands that two threads will be visible outside the coupler on the continuation bar. In this instance, you have a coupler assembly that is "butt to butt" with 2 visible threads showing on the bars. This is consistent with the illustrations in BOSA's material submission (as set out above).

[13] [C10/7011-7016]
[15] [OU5/3309ff]

In fact, the evidence before the COI indicates that the bars threaded by BOSA on the Project rarely had a threaded end of 48mm.[15] As noted, the bars were intended to have thread ends of around 44mm in length but their actual length varied. Indeed, some were less than 44mm. As it was pointed out in the recent COI hearings, it is not plausible that the threaded ends of less than 44mm were shaved or cut by a few millimetres.[16] It follows that BOSA's position that a "butt to butt" connection was not always possible using the bars that BOSA threaded on site.

It therefore makes sense that there is no explicit instruction that the bars should be "butt to butt" inside the coupler. This would only have been possible in limited cases and was not consistent with BOSA's stated threshold of "max 2 visible threads".

6.3. What length of embedment could the construction team have reasonably been expected to be working to in the course of its construction work?

A team of experienced professionals constructed the station. This included the steel fixing labourers, their own management supervision, the supervision of engineering and management staff from LCAL, MTRCL and Py-Pun. It is accepted that this team were working to achieve a tolerance of two visible threads in the coupler connections.

In my opinion, it is inconceivable that anyone in this team would have known of the alleged "butt to butt" requirement. This alleged requirement is not reflected in the instructions given by BOSA in its material submission and is not consistent with the threshold of two visible threads specified in BOSA's manual.

6.4. What length of embedment is specified by design codes?

In short, none. It is not specified. The HKCOP[17] makes no mention of embedment lengths, tolerances on embedment lengths or anything that could be such interpreted.

The HKCOP merely specifies the performance characteristics required for the coupler.

LCAL would not have been able to check their correct use of the coupler by reference to this or any other design code.

6.5. What is a design code compliant coupler assembly?

6.5.1. The HKCOP requirements

A coupler assembly can be said to be compliant with the HKCOP if it meets the requirements specified in clause 3.2.8 of the HKCOP (i.e. the 2004 version, which is the design code that was used for the Project), as well as the requirements for the couplers that were specified by BD when consultation vetting / approval was given by BD for the design of the station structure.

The HKCOP requires that couplers in tension are to meet the following:

- Have a tensile strength exceeding 483 MPa.

[15] [OU5/3309ff]
[16] See cross-examination of Dr Wells on Day 4/32:4-16.
[17] [H8/2818ff].

- Exhibit a permanent deformation of not more than 0.1mm when loaded to 0.6 f_y.

The HKCOP specifies no tests for couplers used in compression. On the basis of the HKCOP alone, it can be seen that the deformation test is not applicable for couplers that are to be used in compression.

6.5.2. BD additional requirements

The BD consultation approval specifies QA procedures for the supply and installation of the couplers, including the following testing requirements:

For couplers without a ductility requirement: [18]

- Have a tensile strength exceeding 529 MPa.
- Exhibit a permanent deformation of not more than 0.1mm when loaded to 0.6 f_y.

For couplers subject to a ductility requirement:[19]

- Have a tensile strength exceeding 529 MPa.
- Exhibit a permanent deformation of not more than 0.1mm when loaded to 0.6 f_y.
- A cyclic tension and compression test.
- The failure of the test sample shall be in "bar break" mode, i.e. failure occurs in the bar away from the coupler.

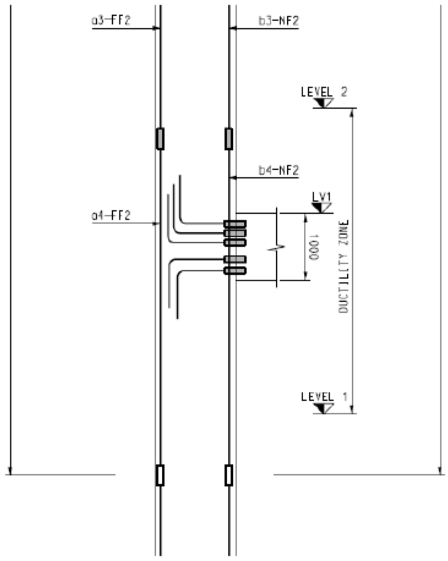
The important distinction is that it is only couplers with a ductility requirement that need to fulfil the cyclic tension and compression tests and be required to fail in bar break mode.

6.5.3. Where do couplers with a ductility requirement need to be used in the Project?

The Atkins design for the station required that some couplers in the D-Walls were subject to a ductility requirement. These couplers were located in marked "ductility zones" (as shown on the drawings prepared by Atkins for construction) that were above and below the the intersection of the D-Walls with the EWL and NSL slabs. The only exception is in Area A of the NSL where a "ductility zone" is shown in drawings to be across the intersection of the D-Wall and slab.[20] There were no ductility zones shown in the drawings for the couplers used within the slabs. As such, none of the couplers used in the slabs were subject to a ductility requirement.

See extract of drawing 1112/W/HUH/ATK/C12/721 Rev A. Panels WH8 & WH9 in Figure 3 below showing the location at the intersection of the slab with the D-Wall where couplers were subject to a ductility requirement (i.e. "ductility zones"):

[18] [C13/8229ff at C8262-8264, C8280- C8282 and C8307-C8309]
[19] [C13/8229ff at C8258-8261, C8276-8279 and C8303-8306]. The tests are ests to be carried out in accordance with AC133 (Acceptance Criteria for Mechanical Connector Systems).
[20] See paragraph 130 of LCAL's Closing Submissions for COI 1.



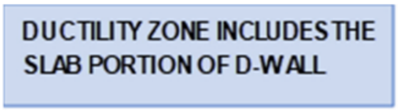


Figure 3 – Atkins Design Requirements for Ductility Zones on SCL 1112 Project.

Couplers with a ductility requirement are not specified anywhere else on the Project. In particular, they are not specified within the EWL and NSL slabs.

6.6. What do MTRCL tests results of couplers show?

6.6.1. Static tension tests

The static tension tests for all coupler assemblies passed the acceptance criteria. The partially engaged coupler assemblies can safely accommodate the ULS capacity load of the reinforcement bars, which as the calculations of all consultants have proven, is often 100% in excess of the actual ULS loading in the reinforcement bars.

6.6.2. Elongation tests

The 0.1mm permanent deformation is not met by any of partially engaged coupler assemblies.

6.6.3. Cyclic tests

The cyclic tests are satisfied where the bar being tested had 7 threads or 8 threads engaged in the coupler.

The cyclic tests failed where the bar had 6 threads engaged in the coupler because the assembly did not fail in bar break mode.

6.6.4. Importance of bar break failure mode

The bar break requirement is specified for couplers with a ductility requirement (i.e. where the design specifically requires the use of a Type II ductility coupler).

As remarked in 6.5.3 above, couplers are only required to be of the ductility type in certain zones of the D-Walls, and not within the EWL and NSL slabs.

6.6.5. Summary

The post construction testing of the coupler assemblies by the MTRCL has demonstrated that the only assembly that meets all of the tests specified in the HKCOP and the BD consultation letters is that which has ten threads engaged inside the coupler and both bars are touching (i.e. a “butt to butt” connection).

It must be re-iterated that although we know this fact now, in my opinion, there was no way that anyone on the construction team (either at MTRCL or LCAL) could have known that “butt to butt” was an integral requirement for BOSA couplers in order to meet the performance specifications of the HKCOP and those set by BD. This necessary information that has allowed me to reach these conclusions only became available after the MTRCL tests were completed and the results could be analysed (i.e. only after the COI had commenced).

6.7. What embedment length is safe to use for a coupler assembly?

In my opinion, which is based on the extensive coupler testing the MTRCL has carried out, all 40mm bar couplers with continuation bar that has 6 or more engaged threads are safe to be used in the Works for the following reasons:

- These couplers have been proved to withstand the static design loads.
- Failure of the permanent deformation test for couplers will not affect the overall safety of coupler – it can still withstand the tensile loads even though it failed the permanent deformation test.
- Failure of the cyclic tension and compression test for couplers will not affect the overall safety of coupler used in this application – a situation where the majority of the loading on the coupler is permanent and the cyclic stress range is much smaller than the test loading regime.
- In accordance with the Atkins design, all couplers used in the EWL and NSL slabs between the inner faces of the D-Walls are non-ductility couplers. Therefore, they do not need to satisfy the cyclic tension and compression tests, nor the requirement for the test failure to be in bar break mode.
- In accordance with the HKCOP, all couplers in the EWL and NSL slabs that are used in compression do not need to satisfy the permanent deformation test. The MTRCL test results have proven that the couplers can withstand the static tension test with 6 threads engaged.
- Those couplers which are in tension in the EWL and NSL slabs between the inner faces of the D-Walls have experienced the majority of their design loading as a permanent load. At Area A of the EWL slab, this is approximately 80 to 87% of the total load according to CEEK calculations[21]. At this percentage, any permanent deformation in the coupler that is in excess of what is allowable will have already occurred. There is no sign of distress or damage to the slab in this area, which is not suprising given the large percentage of excess reinforcement compared to that required by the design as

[21] I understand that LCAL will submit this analysis by EIC to MTRCL shortly and it will be disclosed to the COI in due course.

per section 5.5 above. Therefore, despite the lack of physical evidence, the fact that the statistical evidence might suggest that there might be partially engaged couplers in this area, does not change the fact that the couplers are strong enough to resist the loading without causing distress.

Safe in this context means that the use of the partially engaged coupler assemblies will not endanger the structure, or cause it to suffer distress. It means that the structure will be able to operate as intended by the designer, to withstand the design loads within the designed elastic range of the structure and will allow the structure to achieve its required design life.

6.8. Are the as constructed couplers fit for purpose?

6.8.1. Couplers with 6 engaged threads

In my opinion, all coupler connections with bars that have 6 or more threads engaged are fit for purpose and can be used in the Works.

Fit for purpose in this context means that the use of the partially engaged coupler assemblies will perform the same function as that of fully engaged coupler assemblies in the Works, in that they will provide a mechanical splice between two different reinforcement bars, that will allow the two different bars to act as one and to transfer all the loading from one bar to its connected bar.

In my opinion, in this particular application, the only relevant test to provide acceptability of the strength of the coupler assembly is that of static tensile strength. I accept that such partially engaged couplers may not be HKCOP code compliant in light of what we now know after analysing the results of the MTRCL's tests, but I would remind the reader that the HKCOP is not a statutory document. As explained in section 6.9.2 below, it is a set of guidelines only.

6.8.2. Couplers with 7 engaged threads

It <u>cannot</u> be argued by others that all 40mm bar couplers with bars that have a minimum of 7 engaged threads are not fit for purpose and cannot be used in the Works. The extensive testing of the coupler connections proved that all strength tests, both static and cylic were satisfied where 7 or more threads were engaged. In my opinion, the failure of coupler connections with 7 or more thread engaged to meet the 0.1mm permanent elongation threshold is irrelevant in this particular application of the coupler given the very low percentage of cyclic loading experienced by the couplers. We can be confident this is the case because there is no reported sign of distress or cracking of the slabs or anything else that would suggest that any permanent deformation of the as constructed couplers have caused a problem and, in any event, the requirement of the HKCOP is that this only applies to tensile zones.

6.8.3. The effect of the difference between construction sites and laboratories

It is likely that the probability of achieving the elongation requirement is increased in the practical installation of bars into the couplers, where the installation environment is not the same as laboratory conditions, i.e. in a laboratory (and for the MTRCL's tests) there was no dust, cement and rusted couplers/threads, which are unavoidable factors on a construction site. We know the elongation criteria can be met when the couplers are "butt to butt" and this is due to the ability of being able to apply torque or jamming force to the bars during installation to stop any slip in the threads. This is also likely to occur in actual installation of bars into couplers on a construction site due to the conditions that occur in the actual site environment.

6.9. Can the as-constructed couplers be considered to be code compliant?

6.9.1. The purpose of design codes

Engineering design codes are written by a group of industry experts and are intended to be an all encompassing set of structural design rules for everyday use for the design of whole structures and its components. They define loadings and how the resulting strength of structures are to be assessed and calculated and how the resulting design should be best detailed to ensure the structure acheives its design life.

The HKCOP is clearly stated as a document which only provides guidelines (refer to its foreward, page i) and recommendations (refer to its clause 1.1) for the design of concrete buildings. The foreward for the HKCOP states that it is not a statutory document.[22]

6.9.2. What is the purpose of the HKCOP?

As the COI have already heard, the HKCOP is not a statutory document, but just provides a set of requirements that if followed will ensure that the resulting design and as built structure will be "deemed to comply" with Statute.[23] As such, these "deemed to comply" requirements must be extensive and cover every possible scenario so that a "deemed to comply" approach can be guaranteed.

This does not however mean that the requirements are mandatory and must be followed. Every engineer in the course of design work must use engineering judgement to best interpret design code rules and guidelines to achieve the optimum solutions for the benefit of the projects that are worked upon.

In order to become a Chartered Civil Engineer of the Institution of Civil Engineers, an applicant needs to demonstrate a considerable set of skills, one of which is the exercise of engineering judgement.

6.9.3. What is engineering judgement?

Every graduate design engineer learns of the concept of engineering judgement early in his/her professional career. It is hard to define other than being that an ability to know the correct solution for any particular problem. The engineering professional uses his engineering judgement in design for amongst others:

- To decide the best design approach.
- To know how to solve engineering problems when they arise.
- To interpret design codes to achieve cost effective and buildable structures.

[22] [H8/2821].
[23] See Day 40/195:19-197:22; Dr. Glover's Report at paragraph 5.7, ER1.

The graduate design engineer does not have any engineering judgement at the start of his/her career. However, with a healthy dose of common sense and a willingness to listen and learn from peers, it is likely that a graduate will develop sound engineering judgement.

In the particular field of construction, an engineer uses his engineering judgement to assess if defects have occurred in the construction work and how such defects could be treated. Often, this treatment means doing nothing – the engineer uses his engineering judgement to demonstrate that the defect has no demonstrable effect and does not need any remedial works.

An un-related example of the exercise of engineering judgement is when it is discovered that a contractor has constructed a building with the dimensions of a supporting column incorrectly. Say, for example, this column was designed to have a length and a width of 1m each. But, in error, the contractor constructed this column with dimensions of 0.8m and 0.9m respectively.

The engineer will exercise engineering judgement in assessing the strength of the reduced size of column and if it can be proved by calculation that the reduced size can take the design loading, then the result of such engineering judgement will be to keep the column and not require the contractor to knock down the column and re-build to the originally intended size. This is nice example of a contractual defect being incorporated into the permanent works.

6.9.4. Code compliant couplers

If the results MTRCL's recent tests on the couplers are accepted, it appears that BOSA couplers would need to have a "butt to butt" connection in order to satisfy the strict standards set out by BD for testing of couplers (refer to 6.5.2 above). But, in every situation, it is possible to incorporate defects into the Works, provided that they have been considered and checked in the design.

I believe that no reasonable engineer could be said to have exercised the engineering judgement that his/her professional qualification obligates him/her to exercise if he/she refused to accept the partially engaged couplers (i.e. with 6 or more threads engaged) in the Works for the following reasons:

- They are proven to accept the load.
- There is absolutely no sign of distress or cracking of the concrete.
- The slight rusting in the couplers that were exposed as a result of the opening up exercise have been determined to come from moisture trapped in the coupler before installation and not from water leakage due to cracks.
- The permanent deformation under the test regime loading is so small that it is insignificant when compared to the serviceability criteria and calculated crack widths.

6.10. Opening up investigation

6.10.1. Results and findings of the Holistic Report

As a result of the opening up investigation and statistical analysis carried out by MTRCL, the Holistic Report concludes that the defective rate of coupler connections is 36.6% and 33.2%

for the EWL and NSL slabs respectively, and that the respective strength reduction factors should be applied to each slab independently.[24] Further, it concludes that a higher strength reduction factor should be applied to some connections in locations where the EWL slab was connected to the D-Wall via capping beams.[25]

As a result, MTRCL proposes suitable measure work in a section of Area A of the EWL Slab (as discussed in more detail below).

6.10.2. Statistical expert evidence

The Holistic Report refers to the statistical sampling methods and statistical analysis used to assess the coupler connections at the EWL and NSL slabs.

I understand that following criteria was adopted in determining whether a coupler connection was classified as "defective" or "non-defective":

> *"For the purpose of this study, the proper installation requirement for the couplers are considered to be (i) there shall be a maximum of two full threads exposed (which is stated in the manufacturer's installation requirements); and (ii) the engagement length of the threaded steel rebar inside the coupler should be at least 40mm. As the allowable measurement tolerance of the test equipment is 3mm, equipment readings below 37mm are regarded as defective."*

I understand that Leighton's statistical expert, Dr Wells has made the following comments (among others) on the binomial approach and acceptance criteria adopted by MTRCL:

- The binomial analysis was not appropriate to the data domain.[26] It has the result that a specimen that fails the criteria by only a small amount (e.g. a few millimetres short of the engagement length criterion) is classified as "defective" and does not contribute at all to the competence of the structure.[27]
- The acceptance criteria is internally inconsistent and produces contradictory results.[28] Based on a typical rebar with a 44mm threaded end, if two threads (i.e. 8mm) are exposed, then the maximum engagement length can only be 36mm. The coupler connection would therefore fail the pass/fail criteria and be classified as "defective".
- A number of test results were discarded where a measurement could not be taken when it had already passed the visual inspection (i.e. the coupler was connected).[29] Dr Wells suggested that the correct approach would be to adopt a "Missing Values Approach" (i.e. replace the specimens with the "mean" value of the remainder of the sample rather than discarding them).[30]

[24] Section 3.3.24 and 4.2.3 of the Holistic Report [OU5/3229ff].
[25] Section 3.3.19 and 4.2.3 of the Holistic Report [OU5/3229ff].
[26] Paragraph 4.20 and 5.2 of the Expert Report of Dr Wells for the Original Inquiry.
[27] Paragraph 4.19 to 4.20 of the Expert Report of Dr Wells for the Original Inquiry
[28] Paragraph 4.34 and 4.40 of the Expert Report of Dr Wells for the Original Inquiry.
[29] Paragraph 4.11 of the Expert Report of Dr Wells for the Original Inquiry; Day 4, Page 18 (19-23).
[30] Paragraph 4.12 of the Expert Report of Dr Wells for the Original Inquiry; Day 4, Page 22 (5 25) - 23 (1-3).

Prof Yin is of the opinion that the binomial analysis is a reasonable and suitable approach for the purpose,[31] that it is appropriate because it uses the minimum number of assumptions,[32] and that it is reasonable as it involves less arbitrary decisions in the design.[33]

I understand that Dr Wells has carried out a statistical analysis of the test results in Appendix B3 of the Holistic Report by adopting an engagement length of 28mm (rather than 37mm), which significantly reduced the defective rates from those reported in the Holistic Report to 16.3% (for EWL), 6.9% (for NSL) and 10.2% (for combined EWL and NSL).[34] Dr Wells also carried out the same analysis using the engagement length of 28mm and adopting the "Missing Values Approach", which further reduced the defective rates to 14.5% (for EWL), 6.5% (for NSL) and 9.4% (for combined EWL and NSL).[35] Dr Wells is of the opinion that the correct approach is to take the combined sample for the EWL and NSL slabs.

As explained above, it is my opinion that 6 or more engaged threads can be fit for purpose and can be used in the Works. I am not a statistics expert, but in layman's terms, if 6 or more engaged threads is safe for use then in my opinion the threshold for any binomial analysis conducted should not have been set at an engagement length of 37mm (as adopted in the Holistic Report). A more appropriate threshold would be that of 28mm adopted by Dr Wells when re-analysing the test results.

I also agree with Dr Wells' suggestion that a continuous method of assessment would have been more appropriate given that the load bearing capacity of the coupler connections diminishes gradually as the embedded length of the bar decreases. It makes no sense to completely disregard the strength of partially engaged couplers when in fact the testing that MTRCL have carried out demonstrates that these partially engaged couplers can carry the full design loading.

In summary, I would proceed in making any engineering judgement on the tests results in Appendix B3 of the Holisitic Report by setting a threshold of 28mm for the embedded length and then using the figures derived from a statistical analysis of the coupler connections that satisfy this threshold. Dr Wells suggests that defective rate of coupler connections would fall down to 9.4% or 10.2% depending on whether a "Missing Values Approach" is adopted or not. It is therefore reasonable to assume when making an engineering assessment of the relevant parts of the Works in the EWL and NSL that the defect rate of couplers is no more than 10.2%.

6.11. Couplers in Area A of the EWL Slab

I understand that the only suitable measure work proposed by the MTRCL to be carried out with respect to the couplers is in a section of the Area A EWL slab.

At a location approximately 500mm away from the East D-Wall into the EWL slab, the main transverse spanning reinforcement is spliced using couplers, as per Figure 4 below.

[31] Prof Yin's Expert Reports for COI1 and COI2, paragraph 1.3.5.
[32] Prof Yin's Expert Reports for COI1 and COI2, paragraph 3.2.2.
[33] Prof Yin's Expert Reports for COI1 and COI2, paragraph 3.2.5.
[34] Paragraph 4.27 and Table 1 of the Expert Report of Dr Wells for the Original Inquiry.
[35] Paragraph 4.27 and Table 1 of the Expert Report of Dr Wells for the Original Inquiry.

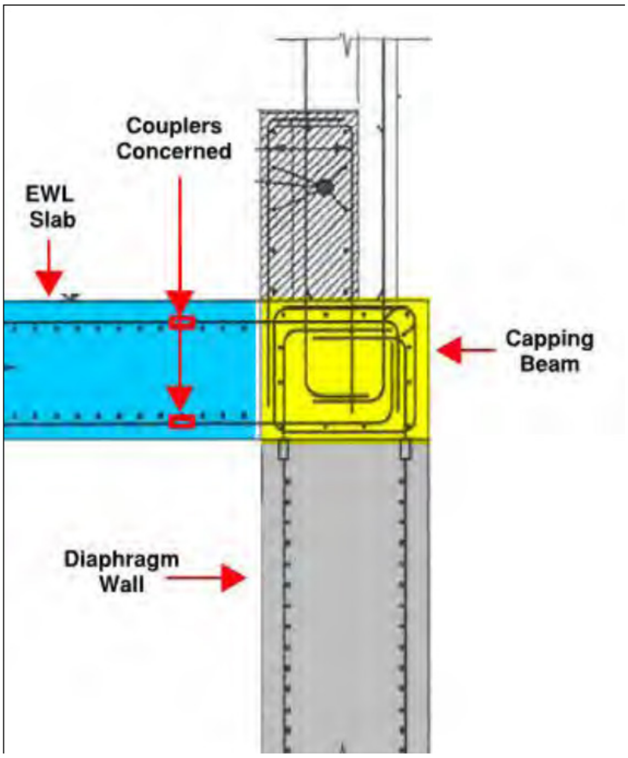


Figure 4 – Layout of Couplers in Area A EWL Slab.

As a result of the opening up investigations and the statistical analysis reflected in the Holistic Report, a higher reduction factor has been applied to the strength of the coupled connections which means that, according to Atkins' calculations, the section is understrength and therefore needs to be strengthened. This is despite no physical investigation work being carried out by the MTRCL in the relevant area, nor any evidence being obtained to show that there are any defective couplers in this area.

The manner in which the statistical reduction factor has been calculated and applied is outside my expertise. However, I note that Dr Wells has commented that the figure has been derived using a very small sample size.[36] Further, as I have noted, there was no sampling or testing of couplers in Area A.

[redacted]

[redacted]

[36] Paragraph 4.40 of the Expert Report of Dr Wells for the Original Inquiry.

7. Shear link reinforcement

7.1. MTRCL Stage 2 opening up investigation

I have not been able to inspect the 18 areas in the EWL slab that were opened up by the MTRCL as part of the Stage 2 investigation works.

I have however reviewed a photographic record of the opening up and inspection of the HZ01 to HZ18 locations on the soffit of the EWL slab, which was provided by LCAL to MTRCL.[37] The report contains photos of each location showing the exposed bars of the slab and records measurements taken for those bars.

Paragraph 3.5.26 of the Holistic Report states that the 18 locations were opened up "each with 1m by 1m patch size". This is not the case. All of the 18 locations are shown to be opened up with an L shape, as shown below in Figure 5 below for location HZ01:

Figure 5 – HZZ01 Opening Up to investigate presence of shear links.

One can infer from this photograph that HZ01 was opened up with 2 right angle slots, each 1m by 150mm. This is not a "1m by 1m patch size" as inferred by the Holistic Report and infact considerably reduces the opened up area.

The findings of MTRCL for HZ01 are that, as no shear links are visible, then no shear links are present at this location. I disagree with this finding and suggest that the reason no shear links are visible is because the location of the right angle slots are not positioned correctly in order to pick up the shear links.

[37] See letter from LCAL to MTRCL dated 10 October 2019

7.2. LCAL Opening Up

This is easily demonstrated using the 1m by 1m square opening up made by LCAL in Area A of the EWL soffit shown in Figure 6 below. [38]

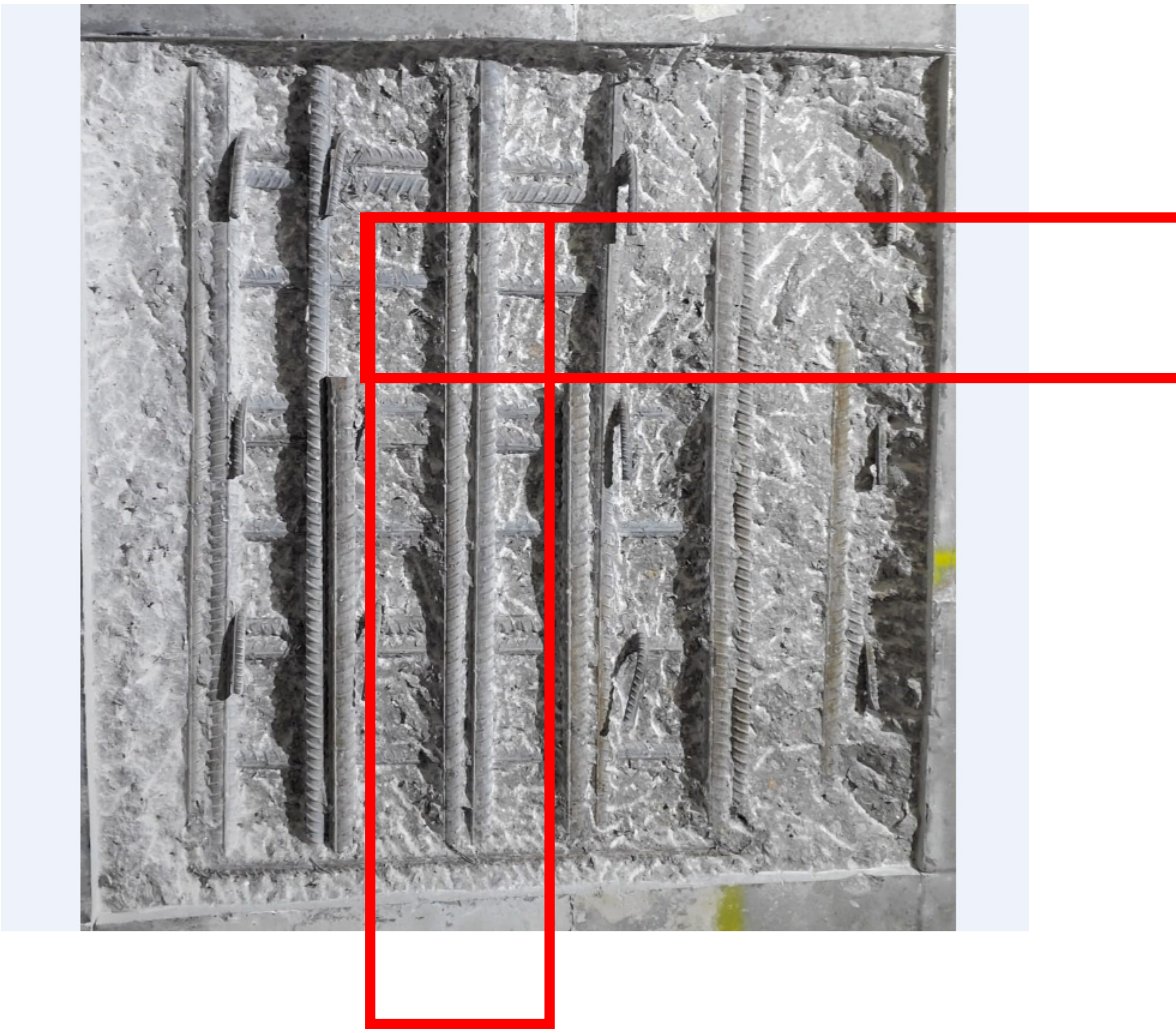


Figure 6 – Superposition of right angle slot opening up onto square opening up.

This opening up was done "correctly" in that it shows the full extent of the 1m by 1m square. If a 150mm x 1m right angle slot is superimposed on this photo, it is possible to see that the right angle slots can be positioned so that no exposed shear links are visible, even though we can plainly see that they are present, but just not inside the right angle slots.

[38] See [OU7/9848]. I was shown the same photograph during a presentation to the experts at the site inspection on 21 September 2019.

I have been advised by the MTRCL at the site visit held on 21 September 2019 that the size of the opening up right angle slots were determined for expediency. Apparently there was not enough time to open up the full 1m by 1m square and therefore a right angle slot was chosen to be more expedient.

Whilst my example above is somewhat simplified, I do not believe there is any legitimacy in the MTRCL's finding that that at locations HZ01, HZ05, HZ08 and HZ10 that there are no shear links present.

7.3. MTRCL conclusions from Stage 2 investigation and Stage 3 assessments

On the basis of the observations made on the 18 opening up locations, MTRCL have unilaterally disregarded the presence of all shear links in the EWL and NSL slabs.

With these shear links disregarded, the Atkins calculations show that in some isolated areas, strengthening of the EWL and NSL slabs is required in order to satisfy the HKCOP requirement of minimum shear links.

In these areas, these calculations typically show that 300 mm^2/m are to be provided to satisfy the minimum shear steel requirement.

Both Arup and CEEK have presented a calculation method to justify the effectiveness of the anchorage. I do not have any particular comments on these calculations methods, both of which are different, but report the same result which is that the as constructed condition of the shear links are adequate.

[redacted]

7.4. Where the shear links required to be extended to the bottom layer of reinforcement?

In some locations, shear links were not observed to be visible in the exposed bottom layer. This may have been because of the reason outlined in 7.2 above, but equally it could be that the reinforcement was stopped in the upper layers of the bottom mat of the reinforcement.

Volume 7 of the Arup Stage 3 report explains the difficulties of installation of the shear links through the highly congested areas of reinforcement in the slabs, which typically had 4 layers of reinforcement in each direction. I agree with these observations and the difficulties LCAL were faced with during the construction.

Arup also explain that it would be reasonable to assume the links could have been installed between the inner layers of the upper and lower mats of reinforcement, this being the reason no shear links were visible on exposure of the bottom layer of the reinforcement mat.

I agree with this statement. It is not necessary for minimum shear steel to extend all the way to the bottom mat of the reinforcement, especially in this instance where the Atkins design has not considered the presence of all of the tension reinforcement in the slab in their calculation of the concrete component of the shear capacity of concrete.

It is important however that the shear link is long enough that the 90 degree bend in the link occurs above or below (depending on the location of the tension face of the concrete) the centroid of the tension reinforcement. This is necessary in order that the shear strut and tie model is valid – this model being the standard manner in which shear analysis and design is carried out in the design codes. Figure 7 below shows the layout of a typical strut and tie system to represent the manner in which shear is modelled in the design codes:

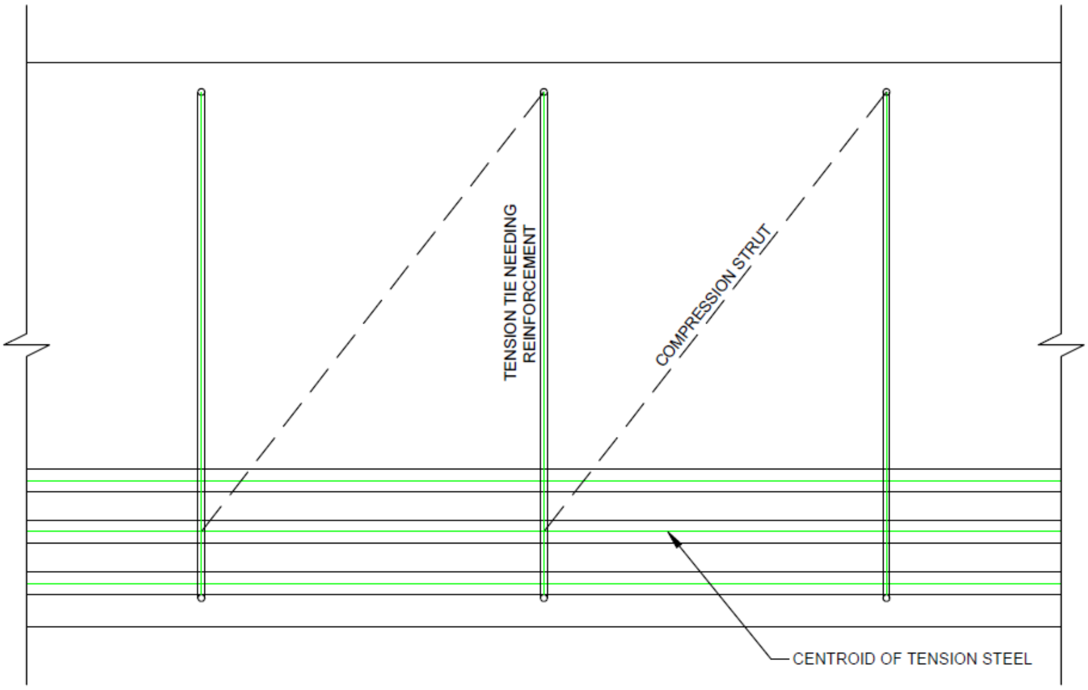


Figure 7 – Strut and Tie shear representation for shear used in design codes.

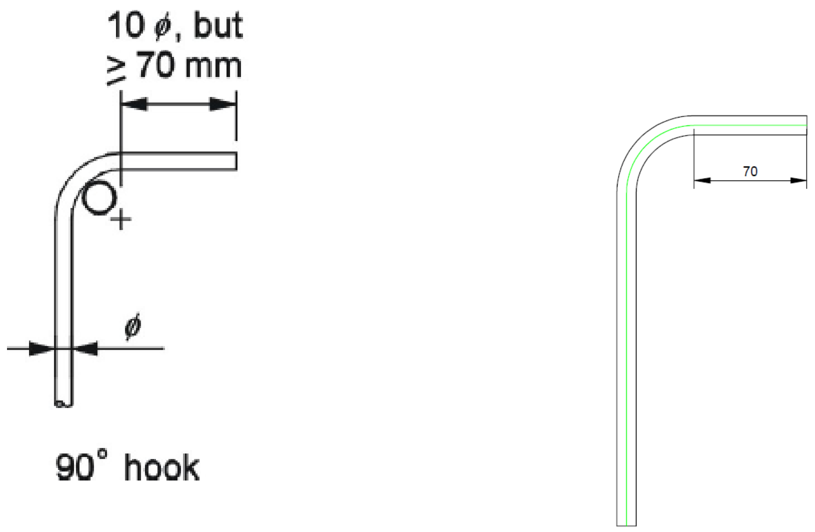
7.5. Review of HKCOP requirements

The chief concern is that the as constructed shear links are not code compliant and this is the reason for them being disregarded in the Stage 3 calculations. This concern is misplaced and I will demonstrate below why the as constructed shear links can be considered to be code compliant.

The code requires a minimum area of shear links to be provided. This is typically 300 mm^2/m and the original design satisfies this by providing T12 bars at 300 centres in both directions. The area provided is therefore 113 / 0.3 = 377 mm^2/m.

There is therefore an overprovision of the shear link reinforcement of 377 / 300 – 1 = 25.6%. This means that the minimum shear links provided can be up to 25.6% ineffective but still meet the HKCOP criteria.

The as constructed shear links are not fully effective in the eyes of the HKCOP (clause 8.5, Figure 8.2) because the straight length after the end of the bend is not 10 x bar diameter = 120mm long, as shown in Figure 8 below.



HKCOP Requirement As-constructed Shear Link

Figure 8 – Comparison of HKCOP and As-constructed shear links

The straight length after the bend has been measured to be 70mm. The anchorage is therefore not fully effective according to the HKCOP, as the length of the straight portion after the bend should have been 120mm.

7.5.1. Determination of anchor length

In order to demonstrate that the as constructed shear links are code compliant, it is necessary to look beyond the simple code rule for the shape of the bar, which itself is only applicable for 100% utilisation and is not relevant in this case.

The principle behind the design of anchorage lengths in the HKCOP is one which is based on the force in the bar and the bond stress between the reinforcement bar and the adjacent concrete. Clause 8.4.3 of the HKCOP defines the design anchorage bond stress as:

$$f_b = \frac{F_s}{\pi \, \emptyset \, l_b}$$

Where,

f_b = bond stress

F_s = force in the bar

$\emptyset$ = diameter of bar

l_b = length of anchorage

Therefore, if the force in the bar is reduced, the length of the anchorage can also be reduced in proportion by simple mathematics, for the same bar size and bond stress.

The bond stress is defined at the ultimate limit state as a function of the strength of the concrete, i.e.:

$$f_{bu} = \beta \sqrt{f_{cu}}$$

Where,

f_{bu}= design ultimate anchorage bond strength

β = coefficient dependent upon the bar type and how it is loaded – ribbed or smooth surface

f_{cu}= characteristic compressive cube strength of the concrete.

This bond strength is used to compute the length of the anchorage of the bar at the ultimate limit state as follows:

$$l_b \geq \frac{f_s}{4\, \emptyset\, f_{bu}}$$

Where,

l_{b_s} = length of anchorage

f_s = ultimate force in the bar, which is 0.87 f_y

$\emptyset$ = diameter of bar

f_{bu}= design ultimate anchorage bond strength

Therefore, the anchorage lengths to be used are based on the ultimate bond stress and the ultimate strength of the bars.

7.5.2. Why anchor lengths can be varied for the same diameter bar

It necessarily follows that if the force in the bar is less than the ultimate strength of the bar, then the anchorage length can be reduced in proportion by simple mathematics.

In our case, the overprovision of the minimum shear steel links by 26% means that the anchorage lengths of the shear steel links can be reduced by 26%. This is because by over providing the shear links, the force in the shear links at the ultimate limit state is reduced by 26%.

7.5.3. Anchorage of shear links

The anchorage length of a straight T12 bar is specified by the HKCOP as 32 bar diameters for the project design materials of grade 40 concrete and grade 460 reinforcement. For a 12mm bar this is an anchorage length of 384mm. This anchorage is based upon the assumption that the bar is fully loaded to the design ultimate loads.

Shear links however are not straight bars, so by necessity they have to be anchored by a 90 degree bend around the longitudinal steel, with the bar extending 10 bar diameters beyond the end of the bend.

The shear link anchorage "starts" at the start of the bend of the bar around the longitudinal steel. The total length of the anchorage of the bar is the length of the bar around its curved section and then the 10 diameters beyond the end of the bend. For a 12mm bar, this is 66mm + 120mm = 186mm, as per Figure 9 below.

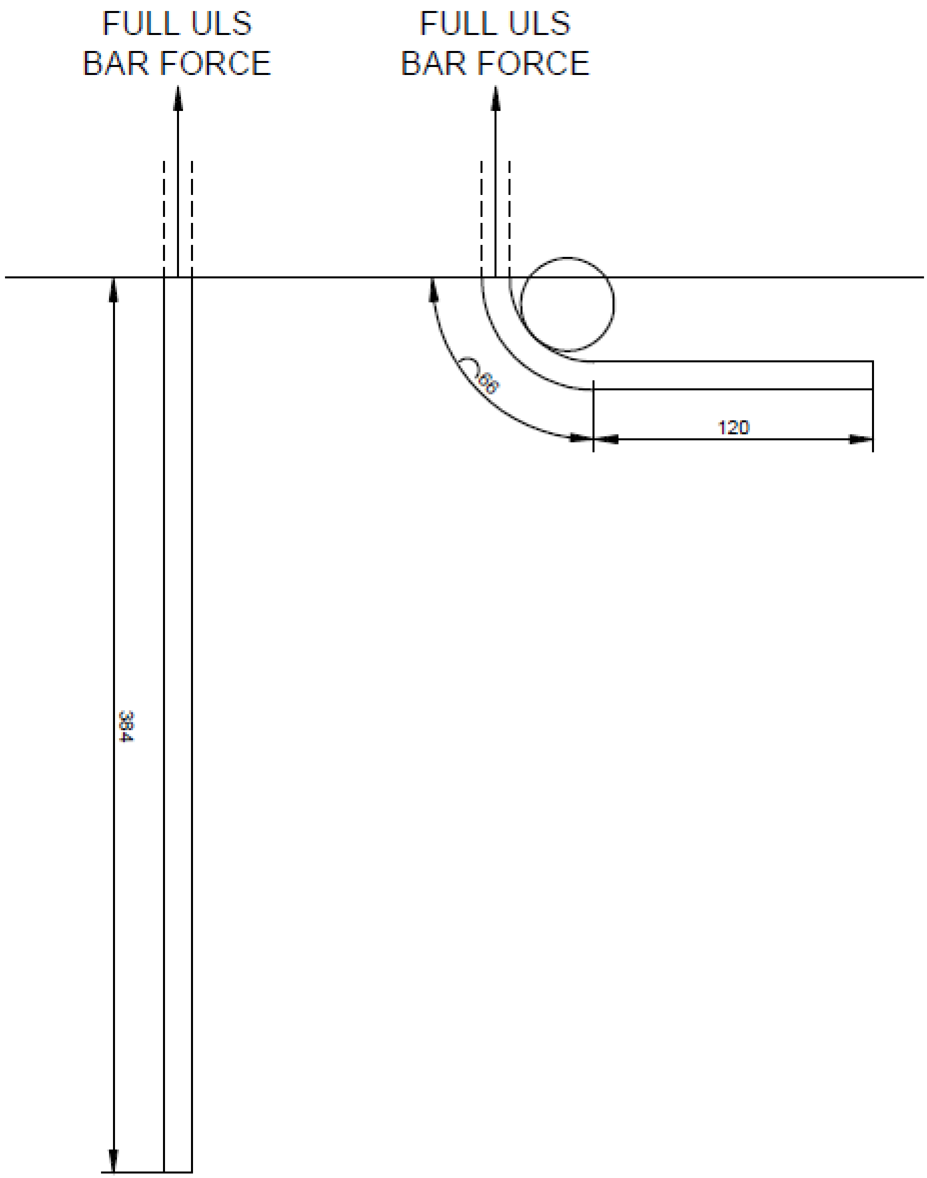


Figure 9 – Comparison of anchorage of straight bar and bent bar around another bar

Therefore, as a straight 12mm diameter bar needs to be 384mm to be fully anchored and a 90 degree bend bar needs to be a total of 186mm long to be fully anchored, we can compute the effective contribution of the bend to the anchorage of the shear link as follows:

384 – 120 = 264mm of equivalent straight bar length.

This is 68.7% of the full anchorage length.

Hence, the bend provides most of the anchorage in bearing. Alternatively, this can be considered as the full bar force required to be anchored – the bar force anchored by 120mm straight section of bar equals the bar force anchored by the bend.

The bend provides its anchorage because it is bent around another bar – it is that bar which is providing the physical interlock to anchor the shear link.

The HKCOP provides some rules about bends in bars in its section 8.3, and allows a standard minimum internal radius bend of 3 x bar diameter provided one of three criteria are met:

- the anchorage of the bar does not require a length more than 4ϕ past the end of the bend;
- where the bar is assumed not to be stressed beyond a point 4ϕ past the end of the bend at ultimate limit state; or
- there is a cross bar of diameter $\geq \phi$ inside the bend.

In our case, the last bullet point is satisfied as the shear links are anchored around the transverse mat of steel, as shown in the Figure 10 below, which is an extract of the LCAL photograph in Figure 6 above.

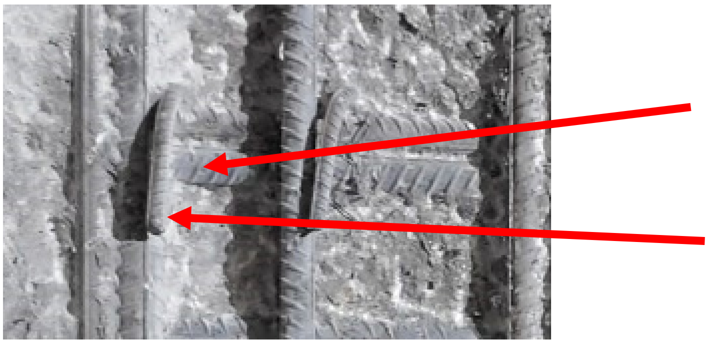


Figure 10 – As constructed shear link anchored around transverse steel

Therefore the HKCOP rules imply the 90 degree bend of the bar to provide 264mm of equivalent anchorage. Its derivation is mathematically valid and nothing in its derivation is contrary to any rules within the HKCOP, but it is however an approximatation. The reality is that this approach has been determinded by testing and successful use.

7.5.4. Anchorage of as constructed shear links

The as constructed shear links only extend 70mm beyond the end of the bend. Therefore, the effective anchorage of the shear link is reduced and is computed as follows:

Bar force taken by the bend, 264mm + bar force taken by the 70mm straight = 334mm

So the reduction in the anchorage capacity of the shear link is computed as follows:

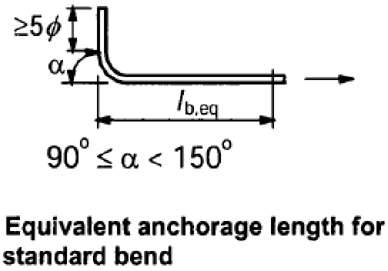
$$1 - \frac{334}{384} = 13\%$$

As the design of the shear links over provides by 26%, the as constructed shear links are adequate for the design and remain in compliance with the HKCOP.

7.6. Approach by other international design codes to anchorage

7.6.1. Eurocode approach to anchorage of bars

As Arup showed in their report (Volume 7, Appendix B, Section B2.2), the anchorage length of the bar, $l_{bd.rqd}$, is 30.3 x bar diameter. A bar which has a 90 degree bend is considered anchored when the distance from the effective start of the anchor to the perpendicular of the far face of the bend, $l_{b.eq}$, is as per figure 11 below.



$$l_{b.eq} = \alpha_1 \, l_{bd.rqd}$$

Figure 11 – Eurocode anchorage for right angle bent bar

where

α_1 = 0.7 due to shape of bar and cover

$l_{bd.rqd} = 30.3$ x bar diameter as computed by Arup.

Therefore $l_{b.eq}$ = 21.2 x bar diameter.

In Figure 12 below, it is possible to see that the shape of the bar defined by the Eurocode that has a full tension anchorage (on the left) is shorter than the as constructed shape of the bar incorporated into the relevant part of the Works (on the right).

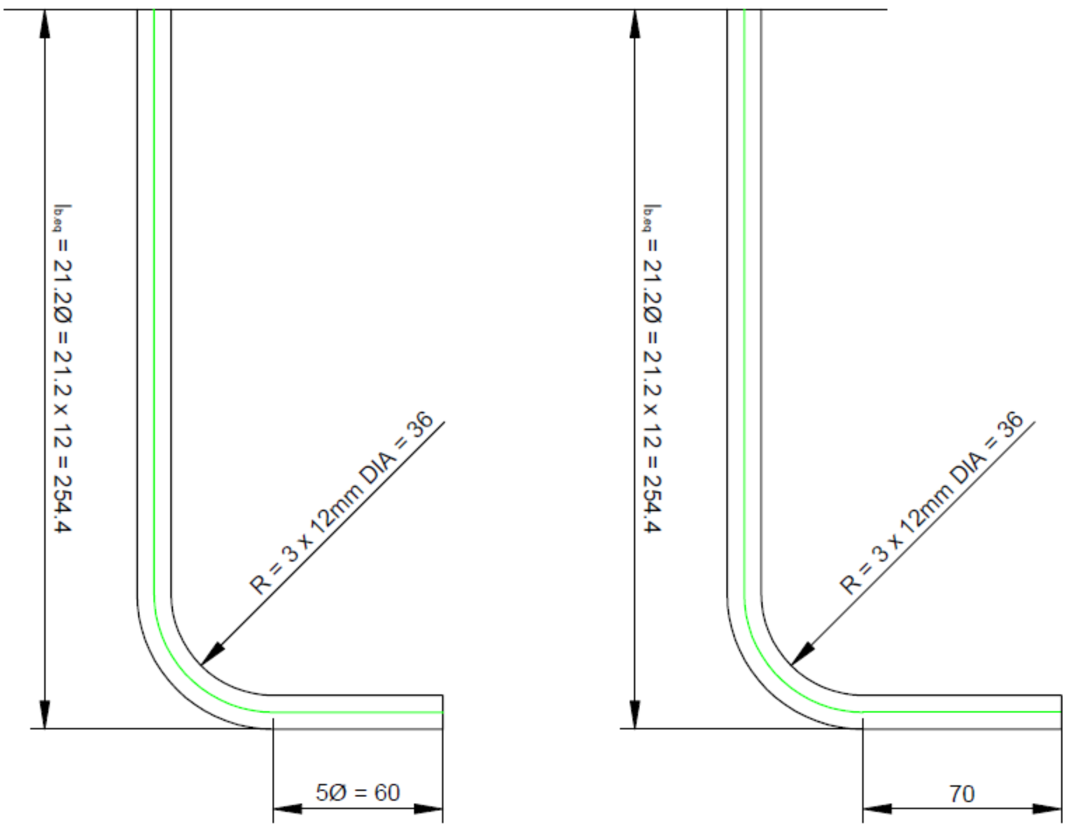


Figure 12 – Comparison of Eurocode anchorage for right angle bent bar and as constructed shear link

Therefore, the as constructed shear link bars can carry a full tension anchorage in accordance with the Eurocode requirements.

Eurocode shear links themselves are detailed in clause 8.5 (2) b) as per Figure 13 below:

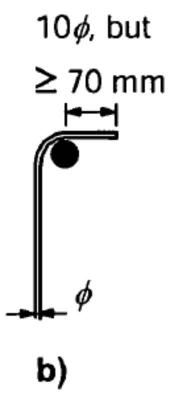


Figure 13 –Details of Eurocode shear links

These are exactly the same shape as that defined by the HKCOP, so therefore the same reasoning as 7.5.4 above can be used to demonstrate that the as constructed shear links are compliant with Eurocodes in terms of ability to carry design loads.

7.6.2. AASHTO approach for anchorage of shear links

The American Association of State Highway and Transportation Officials ("AASHTO") organisation is the publisher of the AASHTO LRFD design code, which is used for the design of infrastructure in the United States of Americanand many other American influenced countries in the world.

This code also has requirements for anchorage of reinforcement bars and shear links. A 12mm diameter shear link bar is considered anchored if the bar has a 90 degree bend and a 6 x bar diameter extension at the end of the bend. Refer to article 5.10.2.1. of the 2017 LRFD version, as extracted below.

- For transverse reinforcement:
  - (a) No. 5 bar and smaller—90-degree bend, plus a $6.0d_b$ extension at the free end of the bar;
  - (b) No. 6, No. 7 and No. 8 bars—90-degree bend, plus a $12.0d_b$ extension at the free end of the bar; and
  - (c) No. 8 bar and smaller—135-degree bend, plus a $6.0\ d_b$ extension at the free end of the bar.

A 12mm diameter bar is the above category of "No 5 bar or smaller" and 6 x bar diameter = 72mm, which is the same (to within the tolerances of bar bending of +/5mm) as the as-constructed shear link.

Therefore, the as constructed bars in the Works comply with the AASHTO codes.

7.6.3. BS8110 approach to anchorage of shear links

Although now superseded by Eurocodes, the British Standard 8110 part 1 1997 specifies the anchorage of links to have a 8 x bar diameter extension after the bend.

3.12.8.6 *Anchorage of links*

A link may be considered to be fully anchored if it satisfies the following:

a) it passes round another bar of at least its own size, through an angle of 90°, and continues beyond for a minimum length of eight times its own size; or

8 x bar diameter is larger than the 70mm extension on the as constructed links. However, it is less than the 10 x bar diameter requirement of the HKCOP and Eurocode, so therefore the same reasoning used in 7.5.4 above can be used to demonstrate the as constructed shear links are compliant.

7.7. First Alternative Method to consider anchorage of shear links

All the codes consider their shear models to use a strut and tie system. This system provides engineers with a simple and readily understandable model of how to analyse and design for the effects of shear. The model assumes that at any one particular cross section of the beam or slab, the distribution of shear force is evenly spread out over the depth of that cross section, i.e. they are designing for the average shear stress in the cross section.

The reality is however different. All of the Consultants have used a linear elastic finite element analysis to determine the level of shear force in the slabs. Such analysis methods assume that the slab is solid and uncracked.

In a solid and uncracked slab, shear stress is distributed parabolically across a rectangular section such as the slabs in the Works, as shown in Figure 14 below.

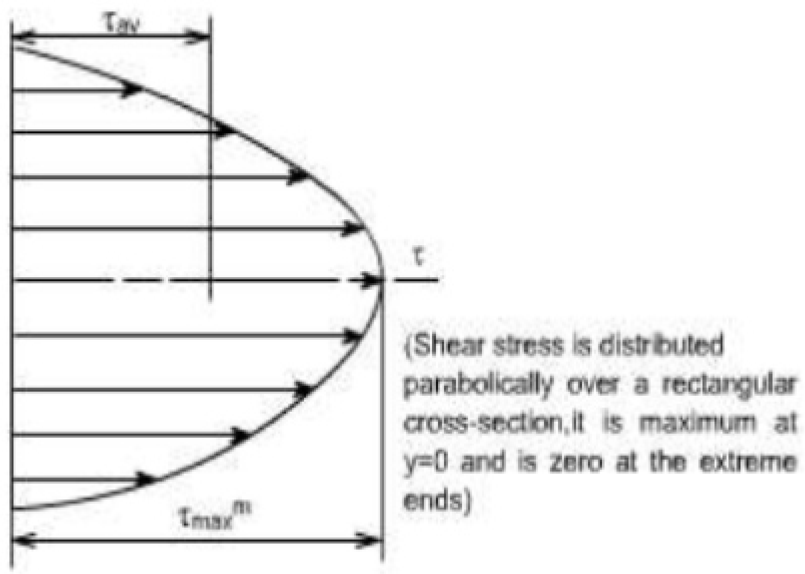


Figure 14 – Distribution of shear stress within the depth of a rectangular section.

Therefore, the actual distribution of shear force assumed by the design analysis in the shear links will have a similar profile to that above, i.e. at the ends of the links, the forces in the links will be very much smaller than the max shear force in the bar.

EIC has reviewed this matter and prepared an analysis that is set out in Appendix E.[39] This analsyis demonstrates that the load to be carried in the bar at the location of the start of the anchorage bend is small and does not require a full shear link anchor length.

[39] I understand that LCAL will submit this analysis by EIC to MTRCL shortly and it will be disclosed to the COI in due course.

7.8. Second Alternative Method to consider anchorage of shear links

In Appendix D of the EIC report contained in LCAL's letter to the MTRCL of 30 August 2019,[40] there was a paper prepared by Professor Stephen Foster concerning the shear strength of the slabs.

EIC had engaged Professor Foster to review the shear behaviour of the slabs, particularly the partial engagement of the shear reinforcement. Professor Stephen Foster is an internationally recognised expert in the shear behaviour of reinforced concrete who was involved in drafting both the Australian code and fib Model International Code rules on shear design.

His paper notes that any critical diagonal shear crack will cross a number of vertical shear reinforcement bars. If it is assumed that the bars have an incorrect anchorage at the bottom, then the following can be considered:

- Where the bar has a full anchorage below the critical shear plane, then it can be considered to be fully effective.
- Towards the bottom of the slab, where the bar is not fully anchored, then a reduced effectiveness should be taken.
- The total shear capacity can then be calculated over the length of the critical shear crack.
- Figure 15 below shows two different shear cracks developing through a sample 1m deep slab and the percentage of shear reinforcement which can be included.

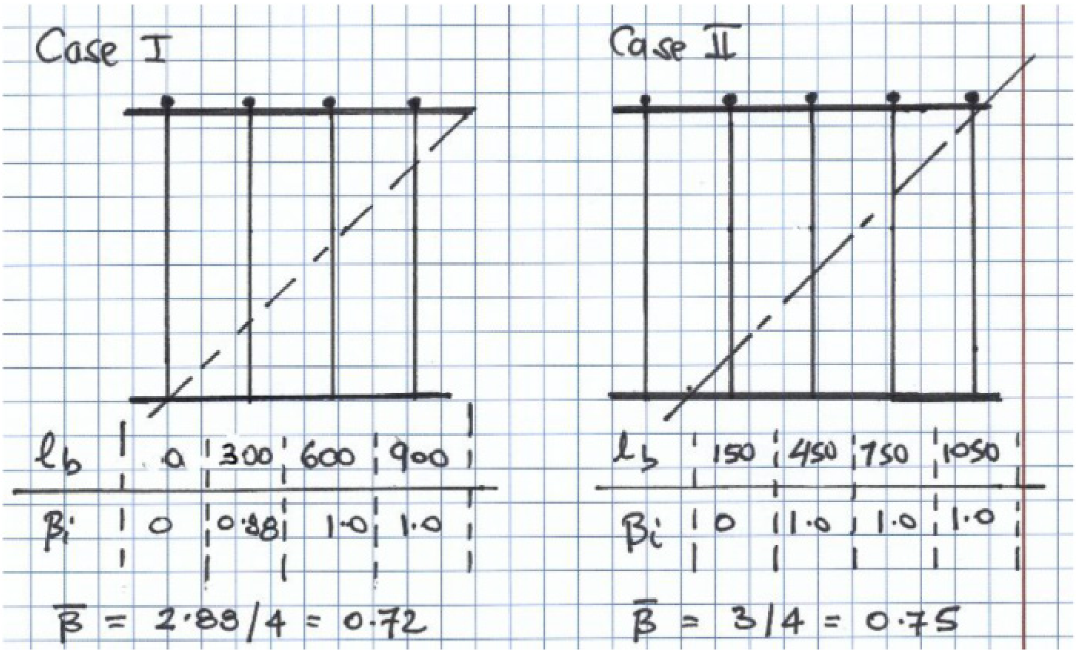


Figure 15 – Anchorage of bars passing through diagonal shear crack.

[40] [OU7/9916-9940].

Shear failure can only occur via a diagonal crack - the part below the diagonal separating and falling away from the section above. So in other words, it is the vertical bars that cross this plane which prevent the shear failure from occurring. Professor Foster demonstrates in his paper how these vertical bars are mobilised.

His example of a 1m slab shows that the effective anchorage of straight vertical bars that have no 90 degree bend is either 0.72 or 0.75 of the full anchorage, depending upon the location of the diagonal plane.

This approach is one of simple engineering, the understanding of the failure mechanism and what prevents it and I agree that this is a valid method to consider.

EIC has considered this approach in their shear assessment of Area A and Area C. In Area A, for the 1m thick slabs with 300 x 300mm T12 shear reinforcement with 70mm long straight lengths after the bend, they calculated that the shear reinforcement provided was 95% effective.

In Area C, for the 3m thick slabs, they considered the shear reinforcement only to be vertical T12 bars with no bend at 300 x 300mm spacing. This arrangement gave an effective anchorage of 86%.

In both cases, the provided shear reinforcement is 25.6% more than required, so the reduction in effective anchorage does not compromise the shear strength of the slab and the shear design remains code compliant.

7.9. [redacted]

[redacted] The shear calculations of all of the Consultants involved demonstrate that the shear demand is considerably less than the shear capacity of the concrete and only the nominal minimum shear links are required in limited areas of the slabs.

As demonstrated above, the demand on these minimum shear links is less than 100% and therefore a reduction in effective anchorage length is possible. In other words, the over provision of the shear reinforcement is such that the as constructed links can still carry the ultimate loads and therefore code compliance is achieved.

In my opinion, there is no justification for completely disregarding the shear links in the design calculations for the following reasons:

- The limited investigation measures of MTRCL do not prove that the shear links were not installed in the relevant parts of the Works.
- In Area A, where MTRCL require most of the suitable measures to be carried out, the only investigation done was by LCAL, one which categorically proves the presence of the shear links.

- The evidence of the as constructed shear links show that links were used that, although do not comply with the detailing rules of the HKCOP, can carry the design loads due to their over-provision and are therefore compliant with the HKCOP.
- The detailing rule for shear links, in terms of the 10 x bar diameter distance required past the end of the bend, is applicable and required for shear links that carry the full ULS design load.
- The HKCOP allows modification of anchorage lengths dependent upon the design load in the bars.
- I have presented the justification for a reasonable extrapolation of the anchorage mechanism through which the shear link works in practice, which means that because of the over-provision of the shear links when compared to the minimum steel requirements, the straight length of the shear link can be reduced to 70mm without compromising the design strength of the structure.

[redacted]

8. D-Wall construction joint

8.1. As constructed design of joint

As referred to in Section 5.8 above, the as constructed design of the joint has been demonstrated by several Consultants to be adequate in a similar manner to the evidence presented by Professor McQuillan, Dr Glover and myself in the hearings in January 2019.

8.2. Condition of horizontal construction joint

Four core holes of the D-Wall slab joint were taken in May 2019 at two of the opening up locations, i.e. D-Wall panels of EH-69 and EM-94. It was reassuring that, as expected, all four core holes showed that there was no sign of distress at the D-Wall / slab interface.

However, two of the four core holes showed signs of workmanship defects. At EM-94, a small gap was observed at the interface. I have not inspected the core, but I understand that this gap consisted of a separation between the concrete of the D-Wall and the slab above, possibly due to contamination of the interface with remanants of loose concrete residue or aggregate, which were not removed prior to casting of the slab.

At EH-69, one of the core holes showed remanants of hessian material.

These two workmanship defects are not of any structural concern. I consider these defects to be minor and would not affect the overall performance of the joint compared to that intended in its design. This is because the construction joint interface is proven by all Consultants to be in an area of very low stress and therefore a change in stress in that area cannot necessarily affect the stress distribution in other more critical areas.

However, because these minor defects have been discovered, I believe they should be remediated by pressure grouting the two core holes in order to seal the interface between the D-Wall and EWL slab.

8.3. If these workmanship issues were prevalent, what would be the effect?

The site photographs which record the condition of the D-Wall interface, and which were presented in the hearings for the first part of the COI, show that the top of the D-Wall was a rough interface. One photograph does not show the interface as being flat, but more of a "v" shape.

I agree with Arup that this surface is a rough surface that would have a coefficient of friction of at least 0.7 and would therefore perform adequately. Although I have not personally inspected the cores taken at the interface, the photographs of the cores shown in Volume 6 of the Arup Report do not display any signs of shearing or tearing of the interface, which would have been a sign that the construction joint was not performing. [41]

[41] OU9595 – OU9598

Although in my opinion unlikely, the effect of the two cores which show workmanship issues would mean that there could be a small gap along the top of the D-Wall between itself and the EWL slab.

The effect of the presence of a small gap has not to my knowledge been assessed by Atkins, Arup or Aecom.

My firm have therefore undertaken a comparative finite element analysis of the joint area.

8.4. Finite element analysis

A finite element model was used to conduct a comparative analysis between a control model and a construction joint that is unable to carry any shear over the unreinforced section. This is a conservative approach for this review as it assumes that no shear can be carried, whereas in reality some proportion of load would be carried by the defective construction joint due to friction.

This analysis is not meant to replicate the detailed non-linear cracked section finite element analysis carried out by Atkins and Arup. Such analysis is not the purpose of this exercise, which is only to demonstrate that if a small gap at the D-Wall interface was present it would make little difference to the overall performance of the joint. The full analysis is shown in Appendix F and is summarised below.

8.4.1. Model description

Model geometry was based on a typical D-Wall joint with a 2.5m section of slab and 4m length of diaphragm wall. The boundary of the slab and diaphragm were restrained by links to enable supports to be applied. A pin was applied at the diaphragm wall and a horizontal restraint at the edge of the modelled slab.

A 2D plane stress analysis of a 1m thick section was conducted with a typical mesh size of 45mm. Where the construction joint overlaps with reinforcement on the diaphragm wall outer face, the concrete was modelled as linear elastic as this zone will be capable of carrying tension and shear (by dowel action across the joint). Where the construction joint is remote from any reinforcement, the construction joint has been modelled with a physical gap between adjacent concrete elements. This gap is then connected by joint elements to allow certain load types to be transferred. The following loads types are transferred:

	Location		
Load type	Remote from rebar (control model)	Remote from rebar (defective joint)	In region of rebar (both models)
Tension	n	n	y
Compression	y	y	y
Shear	y	n	y

Remote from rebar there is no load transfer in tension in either model, as there is no rebar to carry such a load there would be no change if the construction joint were unable to carry tension. Therefore, to avoid a situation where the control model was able to carry a load that would not be present in reality, the load is carried in neither model. Shear can be carried in the control model but not in the defective joint model.

A linear elastic concrete material model was used without explicitly modelling rebar. As this analysis is intended to be a comparative assessment of stress distribution with and without a gap at the construction joint, the exact arrangement and performance of the joint need not be modelled.

Vertical loading was applied at the end of the slab to generate a ULS slab moment and shear of 7090kNm/m and 1350kN/m respectively. These values are of the magnitude determined during previous FE analysis conducted by Arup and Atkins. A precise value does not need to be used as the analysis is comparative between identical models other than the variable under investigation. Loading to the OTE nib was applied, in conjunction with the main slab load defined above.

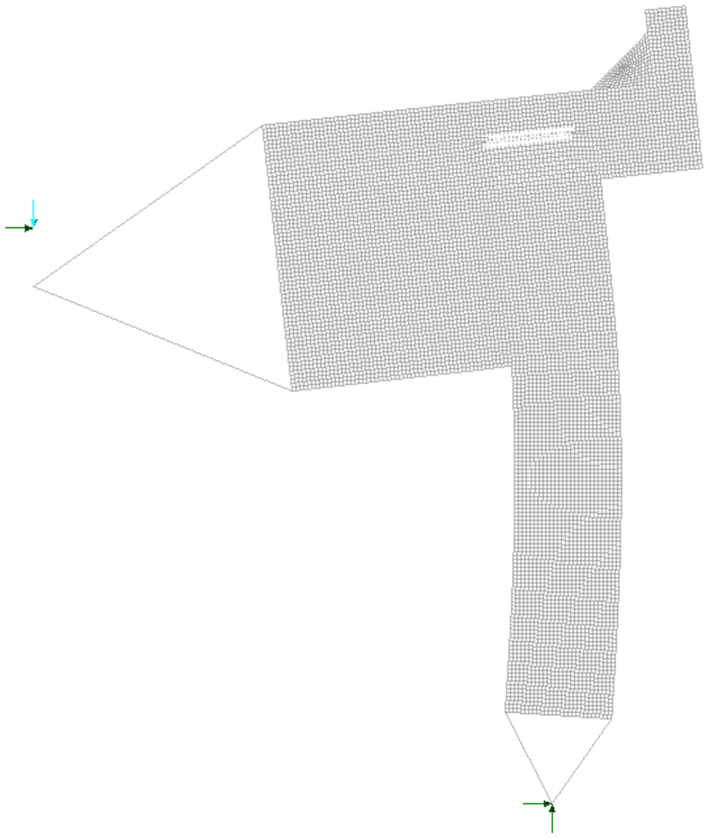


Figure 16 –Model deformed mesh showing supports and applied load

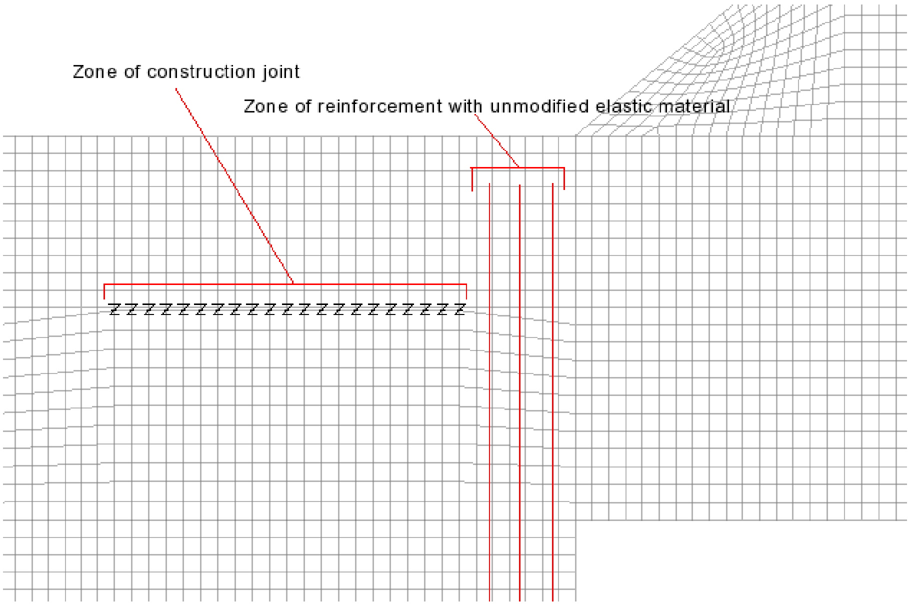


Figure 17 –Zone of reinforcement with an unmodified elastic material (rebar not modelled) and zone remote from reinforcment with variable shear properties

8.4.2. Comparative results

Contour plots of equivalent stress are shown below. These show that the stress distribution for the control and defective joint models are almost identical. A slight variation in stress around the construction joint can be seen in the low stress range.

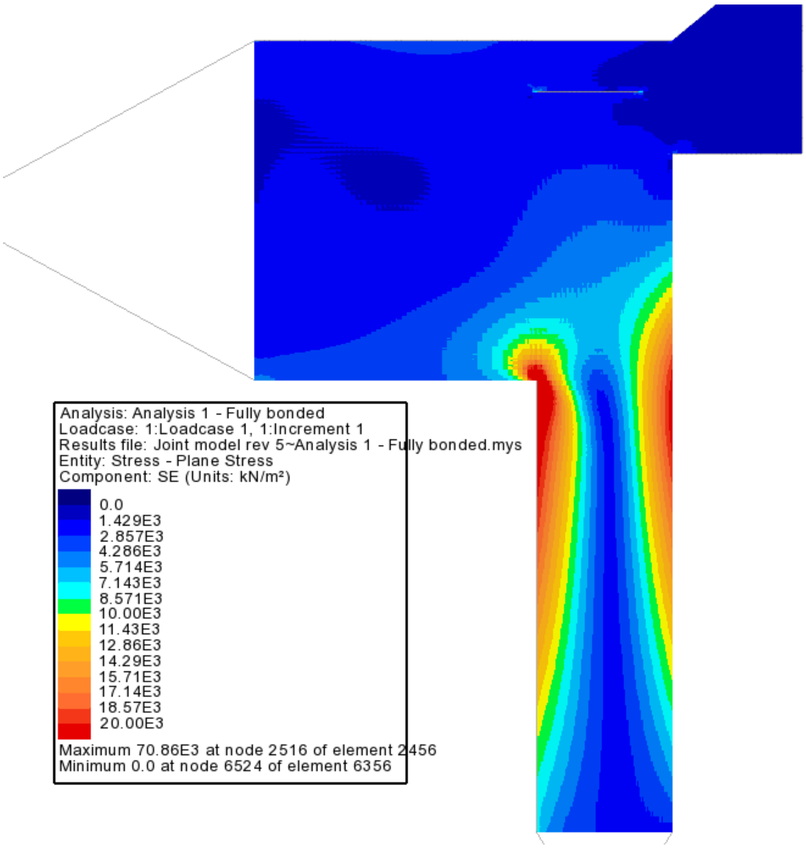


Figure 18 –Contour plot of equivalent stress for control model

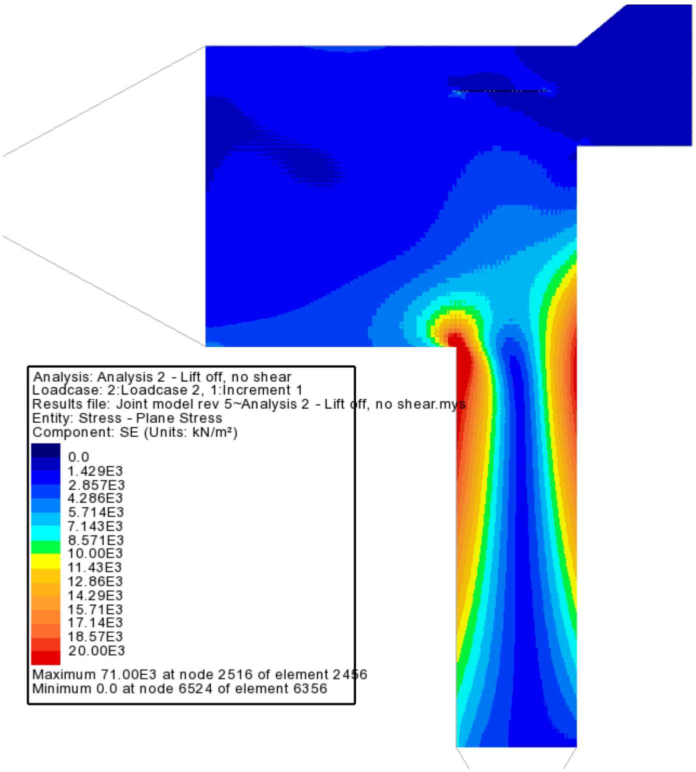


Figure 19 –Contour plot of equivalent stress for defective joint model

To provide an exact comparison of results, a selection of nodes was taken for results output. These comprise the highest loaded areas in compression (1 and 2) and tension (13) up to areas adjacent to the construction joint (6-9).

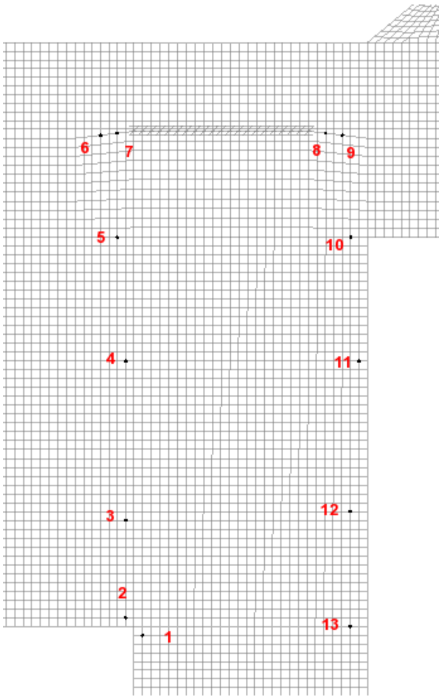


Figure 20 – Results points for comparison

Result point	Control model – horiz stress (N/mm^2)	Defective joint model– horiz stress (N/mm^2)	Change in horiz stress
1	5.8	5.8	100.3%
2	18.5	18.6	100.6%
3	0.89	0.92	104.1%
4	1.04	1.04	100.1%
5	1.08	1.20	111.0%
6	1.67	1.90	113.9%
7	1.77	1.82	103.2%
8	0.53	0.29	54.5%
9	0.52	0.22	43.1%
10	1.42	1.08	76.3%
11	0.01	0.01	110.0%
12	0.01	0.00	80.0%
13	0.10	0.11	101.0%

Figure 21 –Results output for horizontal stress

Within highly stressed areas (1 and 2), the change in horizontal stress is negligible. At points 5, 6, 7 and 11, the stress increases by a small amount but the overall stress is low. The defective joint appears to have little impact on the stress distribution around the connection.

Result point	Control model – vert stress (N/mm^2)	Defective joint model– vert stress (N/mm^2)	Change in vertical stress
1	30.7	30.8	100.2%
2	6.9	6.9	100.2%
3	7.0	7.0	99.9%
4	2.1	2.0	95.7%
5	0.71	0.43	60.5%
6	0.13	0.05	36.2%
7	0.09	0.08	88.0%
8	0.42	0.25	60.8%
9	0.17	0.14	79.3%
10	0.59	0.59	100.5%
11	4.5	4.4	97.5%
12	11.6	11.6	99.9%
13	16.9	16.9	100.1%

Figure 22 – Results output for vertical stress

Within higher stressed areas (1, 2, 3, 12 and 13), the change in vertical stress is negligible. In all other areas, principally where stress is very low, stress is reduced. For vertical stress, the defective joint appears to have no significant impact.

8.4.3. Conclusion

The presence of a completely unbonded construction joint that cannot transfer shear alters local stress immediately adjacent to the joint. However, this is an area of low stress and any increase in stress is not significant in relation to overall material strength. This effect is partly due to the presence of a modelling discontinuity at the end of the joint leading to the formation of a stress concentration. In reality, some shear would be transferred by friction which is not accounted for in the above analysis.

Areas of the connection subject to high stress are unaffected by the ineffective joint such that no impact to the overall joint capacity is anticipated.

8.5. What are the suitable measures proposed?

The Holistic Report proposes that “suitable measures” are carried out at this D-Wall / EWL slab interface in Areas B and C for approximately 60m of the D-Wall in order that the required safety level in the code is achieved.

The report does not define these “suitable measures” in detail, but I understand from discussions with the MTRCL at the Site Visit on 21 September 2019 that the work involves installing 25mm diameter bars, vertically at 600mm centres, to provide reinforcement continuity between the D-Wall and the EWL slab through the construction joint.

8.6. What is the effect of carrying out the suitable measures?

The provision of these dowel bars is clearly meant to provide additional horizontal shear strength across the construction joint. The existing reinforcement across the joint for a typical 3.59m wide D-Wall panel is 20 vertical sets of 2 x T50 + 1 x T32 on the soil face and 1 x T50 on the excavation face. This gives a total cross sectional area of steel of 20 x (3 x 1963 + 804) = 133,860 mm^2 of reinforcement that crosses the horizontal construction joint and which physically locks the EWL slab and the D-Wall together.

The suitable measures involve providing 3.59 / 0.6 = 6 number additional T25 bars per panel, which is 2,946mm^2 of steel reinforcement, which is 2.2% additional reinforcement across the construction joint.

The enchancement to the shear capacity of the section with these additional bars is negligible, because, in the context of engineering design calculation, a provision of an additional 2.2% is insignificant. In other words, the manner in which engineering design analysis and calculations are done means that it is not possible for two different engineers' calculations to be similar enough that one is within 2.2% of the other. This is clearly evidenced by analysis of the results of the calculations of the different Consultants as mentioned in section 5.8 above.

8.7. Is there any justification for carrying out the suitable measures

In short, no. I do not believe there is any justifcation for the "suitable measures" for the following reasons:

- As explained above, the addition of 25mm bar across the interface at 600mm centres will not provide any significant contribution to the shear strength of the section.
- A finite element analysis of the section has shown that the presence of a small horizontal crack at the construction joint makes little difference to the performance of the joint.
- The detailed work of Atkins, Arup and Aecom showed that the shear links in the D-Wall played an important part in the strength capacity of the D-Wall / EWL slab connection. If vertical bars are to be drilled into the top surface of the EWL slab and then downwards into the D-Wall, there is a significant danger that the horizontal shear link bars might be cut by the action of the drilling.
- There is a minimum thickness of 200mm, but typically 450mm of EWL slab above the top surface of the D-Wall. Whilst it should be possible to determine the position of the EWL top slab reinforcement, and thefore locate the drill holes to avoid that reinforcement, it is not possible to detect reinforcement that is 200 to 450mm below the exposed top surface of the EWL slab. There is no possible way to ensure that the shear link bars will not be cut during the drilling and it will be purely down to luck if none are damaged. Therefore, this is a significant risk and one which I do not recommend is taken.

9. Conclusion

[redacted]

[redacted]

- Tests have proven that the partially engaged couplers can withstand the design loads.
- The as constructed shape of the shear links can resist the applied design loads in accordance with the HKCOP requirements.
- The condition of the top of the East D-Wall at its junction with the EWL slab is of no structural concern, even if the worst possible assumption is made that there is a physical gap between these elements.

The Stage 2 investigation and Stage 3 assessments have served their purpose in confirming the opinions of the structural engineering experts who gave evidence to the COI in January 2019, that the station structure is safe [redacted].