Concrete repair challenges in the Middle East

*Rene Brueckner**

Mott MacDonald, 10 Temple Back, Bristol, BS1 6FL, UK

Abstract. The root cause analysis, determination of the extent of concrete defects and identification of an appropriate repair strategy can be straightforward, but it also often provides significant challenges to both contractors and engineers. The challenges can be due to a lack of QA/QC documentation, locally available investigation equipment, repair material and techniques and accepting actually feasible solutions.

The paper presents a case study of a bridge in the Middle East where the root cause analysis was found to be straightforward but the determination of the extent of non-visible defects and the implementation of a feasible repair strategy proved to be much more complicated. The root cause analysis of the defects was carried out by visual inspection, representative intrusive investigations and a comprehensive document review. The determination of the extent of the defects using GPR techniques proved to be challenging under the local conditions. Even more difficulties were encountered by implementing the developed repair strategies to address the various defects to ensure that the required 120 years design life in a very aggressive environment can be achieved. Several revisions to the repair strategy were necessary to identify the most appropriate solution and to accelerate the programme.

1 Introduction

The design of a new 56km highway in the Middle East comprised the construction of 5 main interchanges with approximately 7km of bridges and elevated ramps. The majority of the structures consisted of multiple-span post-tensioned concrete box girders with span lengths varying from 30m to 45m that were constructed span by span using precast segments. The precast design was beneficial for a rapid and efficient construction with a high-quality durable product. The contractor, who was experienced in the construction of these types of structures, started with the precast bridges and elevated ramps and decided to undertake the widening of a bridge at an existing junction using in-situ construction methods towards the end of the project.

The existing main bridge to be widened consisted of two symmetrical bridge decks with a 1.6m deep post-tensioned multi-celled box girder with fours spans each and a length of 28 to 30m per span. The demand for the new highway and upgrade of the existing bridge required the bridge to be widened by 9.1m to increase the total width of each bridge deck to 27.15m. The new widening bridges were designed to be a four span post-tensioned cast in-situ voided slab deck with the same span configuration as the existing bridges. Furthermore, the design required the widening to be fully integrated with the existing structure along its cantilevered slabs of the existing structures. A general cross-section of the new widening section is shown in Figure 1.

Due to the inexperience of the contractor in the construction of in-situ bridges and the decision to undertake the works towards the end of the project it was apparent at an early stage that problems were likely to occur. Upon the completion of the first two spans of the bridge widening, several areas affected by honeycombing were observed on the soffit of the widening decks. Following initial breakouts and repairs it was found that the defects extended over a wider area and were not only limited to the visually identified honeycombed areas.

2 Defects

Honeycombing on the soffits of two spans was observed upon the removal of the formwork which initiated a more detailed investigation and assessment of the extent of the defects. This resulted in the identification of the following defects:

- Honeycombing resulting in non-embedment / lack of bond of the lowest reinforcement in localised areas;
- Voids below lapping bars – lack of bond;
- Cold joints within the lowest reinforcement layer;
- Gap between void formers and internal concrete cover to the lowest reinforcement layer which reduces the internal concrete cover;

* Corresponding author: rene.brueckner@mottmac.com

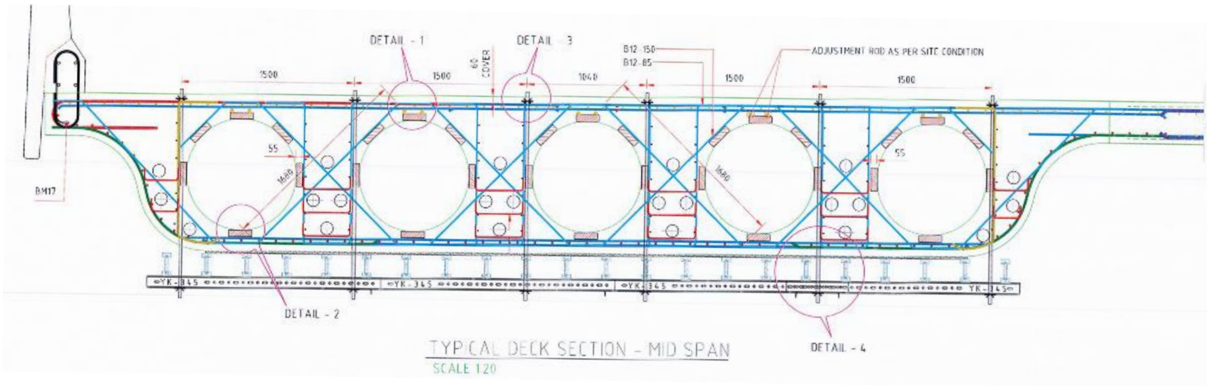

Fig.1. Cross-section of bridge widening

- Reduced level of compaction of the concrete between void formers and lowest reinforcement;
- Reduced localised cover;
- Cracking on top of slab;
- Excessive debris, such as reinforcement caps, tie-wires, reinforcement pieces in soffit reducing the effective cover;
- Polyethylene sheets above bottom reinforcement layer preventing encapsulation of the reinforcement.

The defects were mainly located below the five void formers of the voided post-tensioned widening slabs. Figure 2 shows a general view of the most-affected area of one span where the lines of the five. void formers are marked on the soffit (yellow line).

Fig.2. General condition of soffit

Honeycombing was identified over a large area but the extent of the honeycombing into the slab varied significantly. Near-surface honeycombing defects were observed on the majority of the soffit whereas honeycombing extending beyond the lowest reinforcement layer was mainly observed below the void formers (Figure 2 to Figure 4). The maximum extent of honeycombing into the slab was found to be approx. 180mm, i.e. up to the bottom of the void formers. The honeycombing resulted locally in a lack of bond around the reinforcement but there was no honeycombing below the post-tensioning ducts away from the void formers. The soffit also suffered from an excessive amount of debris that was not cleaned before the concrete pour and included reinforcement pieces, caps and large polyethylene sheets.

Cold joints were found between the 1st and 2nd pour layer which is located approximately between the two lower layers of transverse reinforcement or between bottom reinforcement and void former. A gap was observed between the void formers and the lowest reinforcement in several locations, reducing the internal concrete cover to the lowest reinforcement. The gap was mainly encountered where the bottom spacers for the void former (concrete blocks: 200x200x55mm^3) were located at a distance of 100mm from the joints of void formers.

It was observed that the size of the gap varied and that there was a small gap potentially along the entire length of the void formers. The latter does not reduce the protection by the internal concrete cover as it was mainly a result of the void former floating up during concreting and the specified internal cover was obtained.

Cracking was identified on top of the slab and appeared to be a result of early thermal cracking and drying shrinkage. No cracking was observed on the soffit due to the post-tensioning.

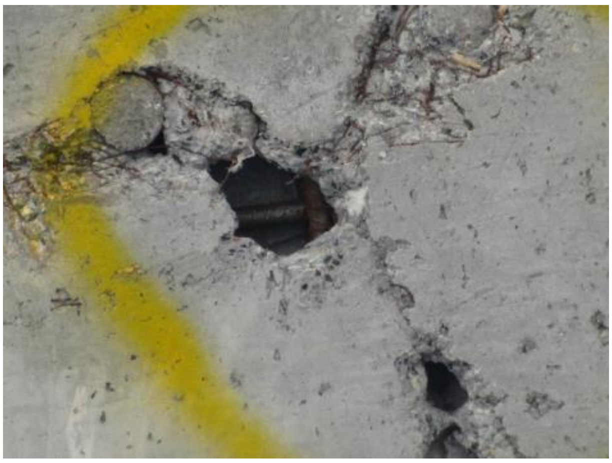

Fig. 3. Honeycombing with visible reinforcement

Fig.4. Void below void former and non-embedded rebar

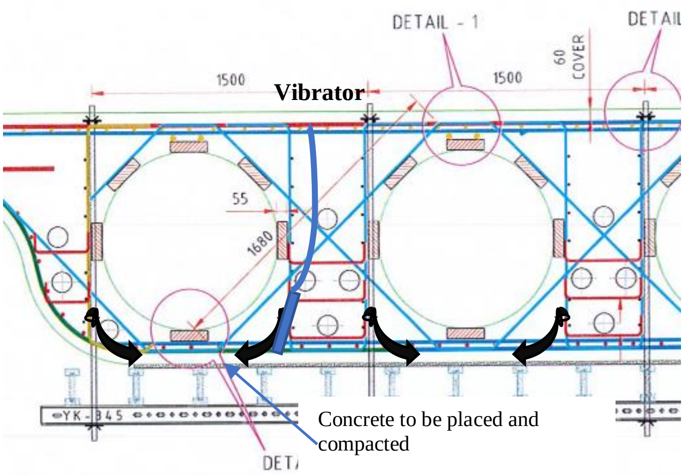

Fig.6. Cross-sectional arrangement

3 Root cause analysis

The root cause of the defects was identified to be a combination of varying concrete consistency and concreting / compaction operations. The consistency of the C45/55-10mm concrete (OPC+35% PFA+7.5% MS), as delivered to site, ranged from 160mm to 210mm with periods of lower slumps (130-140m^3 of 490m^3), as was observed through compaction issues at the time of the pour and can be seen in Figure 5. Such consistency may be considered suitable provided that the concrete is fully compactable in all locations and the pour layers are interlocked during compaction without the risk of cold joint formation.

It was also reported that the concreting sequence consisted of pouring an initial layer of concrete that only covered the bottom transverse reinforcement before proceeding to build up the full height of the section that included the bottom longitudinal and second transverse reinforcement layer. This pour method was implemented to reduce the uplift of the void formers. The time between finishing the bottom layer and continuation of the full height concrete pour was reported to have resulted in total loss of workability of the 1st layer of concrete so that the 2nd layer could not be poured 'fresh on fresh'. Compaction of the 1st layer was also inadequate.

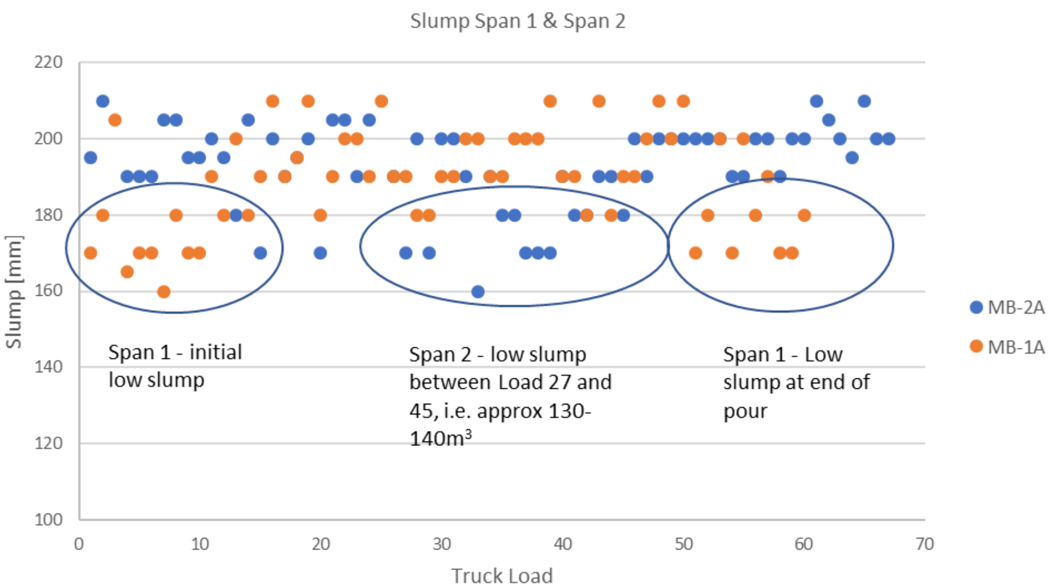

Fig.5. Slump of Slab 1 and Slab 2 pours

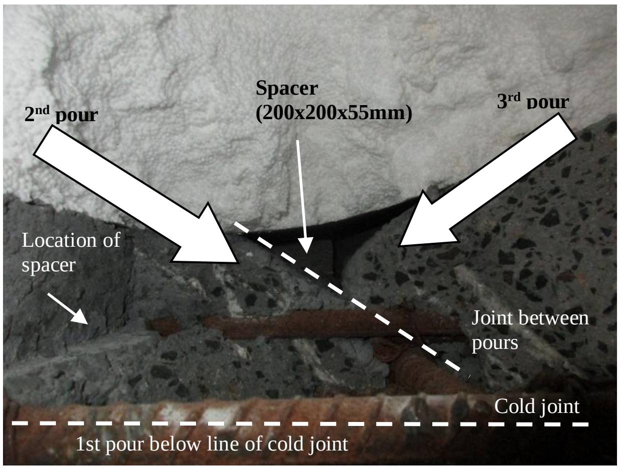

Fig. 7. Root cause schematic

The formation of a cold joint between the two layers was prevented below the ducts; however, cold joints were identified under the void formers. This was a result of the arrangement of the void formers, spacers, post-tensioning ducts, reinforcement bars that were spaced at 125mm and 150mm and the delayed time between the two layers in addition to the partially reduced consistency of the concrete. The arrangement made it difficult to insert vibrating pokers and compact the concrete below the void formers whereas compaction around the ducts was achievable, as indicated in Figure 6 and shown in Figure 7, and also discussed by Grantham [1].

There was also a high degree of debris contained within the concrete cover zone which was attributed to poor housekeeping, cleaning and workmanship.

4 Determination of the extent of defects

The extent of the defects on the two slabs was determined using both intrusive and non-intrusive testing methods. The latter comprised Impact Echo and GPR scanning to determine sub-surface defects but both techniques proved unsuitable due to interferences from vibrations from the traffic on the existing neighbouring bridge and the extent of reinforcement, respectively. The verification of negative anomalies in the GPR scan data with intrusive break-outs provided unreliable results. Based on this the extent of repairs was determined by a combination of visual inspection, investigation holes and representative break-outs.

Investigation break-out holes of approx. 100mm diameter at a spacing of 3m along the line below each void former and representative break-outs were used to determine the extent of gaps, sub-surface honeycombing and loss of bond between concrete and reinforcement that could have effects on the durability and structural integrity of the deck.

5 Remediation strategy

The environmental exposure conditions in the Middle East are very aggressive in particular with regards to chloride induced reinforcement corrosion when located in proximity to the Gulf. In addition to the exposure conditions, the design life requirements of the new highway structures for this project was specified to be 120 years.

The effects of the various defects identified on the soffit of the decks were assessed to be loss of resistance to chloride ingress from external sources due to lack and loss of cover as a result of sub-surface and near surface honeycombing in combination with localised lack of reinforcement embedment below the void formers. As the specification did not require a waterproofing system to be installed on the deck of the bridge and existence of cracks there was a high risk that moisture and chlorides may enter the voids resulting in chloride ingress from the internal face. The resulting reinforcement corrosion of the bottom layer and associated spalling was considered to have a high potential to develop into a hidden defect.

The lack of reinforcement embedment below the void formers and its effects on the structural integrity were not significant but had to be repaired to ensure the as-design conditions.

The effects on the specified design life of 120 years were required to be mitigated by reinstating the resistance against chloride ingress and carbonation of external and internal faces. As the original durability design only relied upon the resistance of an uncracked concrete cover zone around the bottom reinforcement layer by post-tensioning a protective coating system was not specified. At the time the defects were observed and repairs were planned to be carried out, the structure had already been post-tensioned and therefore the removal of the entire soffit was not desirable as the status of the cover zone would change from compression into tension following repair and the time required to complete the demolition and reinstatement was unacceptable. However, initial discussions included the removal of the entire soffit of the worst affected deck in combination with the application of a protective coating system.

Taking into account the various defects and project durability requirements a repair methodology was developed and proposed to the contractor which comprised:

- Repair of the gaps below the void former and reinstatement of the internal concrete cover by injection of cementitious grout which also addressed the lack of reinforcement embedment in interconnected voids.
- Repair of small-scale honeycombing and inspection break-outs by using a cementitious shrinkage compensated concrete repair mortar with a minimum equivalent compressive strength of a C45/55 concrete.
- Repair of large defect areas by adopting one of three specific reinstatement methods as in the following paragraph.
- Application of a protective coating system to mitigate the risk of deterioration from any unidentified and hidden defects.

Method 1 of the large area repairs comprised gravity pouring of high flow mortar from the top of the bridge whereas Method 2 was suggested to repair the defects on the soffit by pumping a repair mortar from the soffit, see Figure 8. The third and considered most suitable repair method (Method 3) was sprayed concrete application.

In addition to the three repair methods a preplaced aggregate method in combination with self-compacting mortar pumped from the bottom was also considered but excluded due to the increased labour cost, the required skills and levels of control. The most likely outcome would have been incomplete grouting due to inexperience and dry and hot aggregate in addition to areas with lack of aggregate as a result of insufficient aggregate placement.

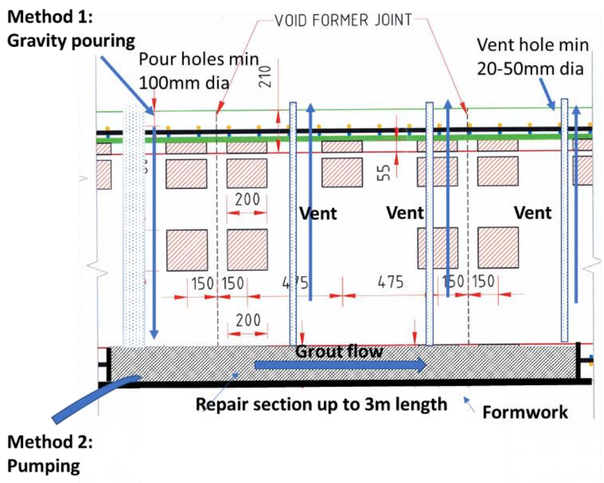

Fig.8. Repair schematic of Method 1 and 2

6 Implementation

6.1 Large defects

Removal of defective areas and repairs commenced at the worst affected span and comprised the removal of the entire length below two of five void former lines as part of the initially considered replacement of the full deck soffit. The methodology consisted of two stages, Stage A comprised the removal of the concrete below void formers 2 and 4 including reinstatement and Stage B was planned to cover void former lines 1, 3 and 5, see Figure 9. The specified demolition method was hydro-demolition but this could not be sourced and the concrete was broken out using hand-held pneumatic breakers.

The reinstatement of the large strips proved to be challenging due to various reasons:

- A competent and experienced sprayed concrete subcontractor could not be identified and Method C had to be discarded;
- A suitable pump for Method B could not be sourced.
- Method A, gravity pouring from the top of the deck, created further difficulties and challenges.

The challenges comprised the opening up of the pour and vent holes and removal of all polystyrene debris from the void formers, sealing and support of the formwork and manpower for mixing and pouring of the concrete repair mortar. The repair of the first 3m long by 1.25m wide section resulted in severe leakage from the formwork, missing of pour holes and insufficient manpower for mixing to fill the repair within the workability time of the mortar. This resulted in opening up of an additional pour hole during the pour causing polystyrene debris to fall into the formwork. Hence, a section of the trial repair had to be repaired.

The repair mortar was a shrinkage compensated, high flow and high strength pre-packaged concrete repair mortar that may be used with additional aggregate for larger thicknesses. The incorporation of aggregate was assessed and found to cause additional problems as there was a high risk that the aggregate may not be stored in an air-conditioned environment. Pre-bagged aggregate stored in the shade with ambient temperatures of 35 to 45ºC, as at the time of the repairs, significantly reduced the workability of the mortar.

The difficulties encountered during the repairs of the first two strips and observations that the concrete was acceptable below the post-tensioning ducts and outside the areas of the worst visible surface defects led to an optimisation of the repair procedure. Further verifications using only smaller strip break-outs of 1.5m x 0.3m at representative locations along void former lines 1, 3 and 5 were carried out. The 1.5m x 0.3m strip break-outs comprised a hit and miss strategy of three break-outs covering a length of 7.5m along one line. Signs of honeycombing around the reinforcement and up to the void former required continuous hit and miss break-outs including removal of all honeycombed concrete under all void former lines of the respective span. The three hit and miss trials, that were undertaken at the worst visually affected areas without obvious full depth honeycombing, confirmed that full removal of the deck soffits and large strip break-outs below each void former could be avoided. However, this method identified the presence of a large cast-in PE sheet above the bottom reinforcement layer, see Figure 10.

Nevertheless, any areas showing visible surface defects and cast-in debris were removed and extended to sound substrate if required. This resulted in both shallow and deep break-outs requiring repair using Method A or hand applied overhead application.

Small-scale deep repairs were carried out by using a hand applied high-strength concrete repair mortar that was suitable for overhead applications.

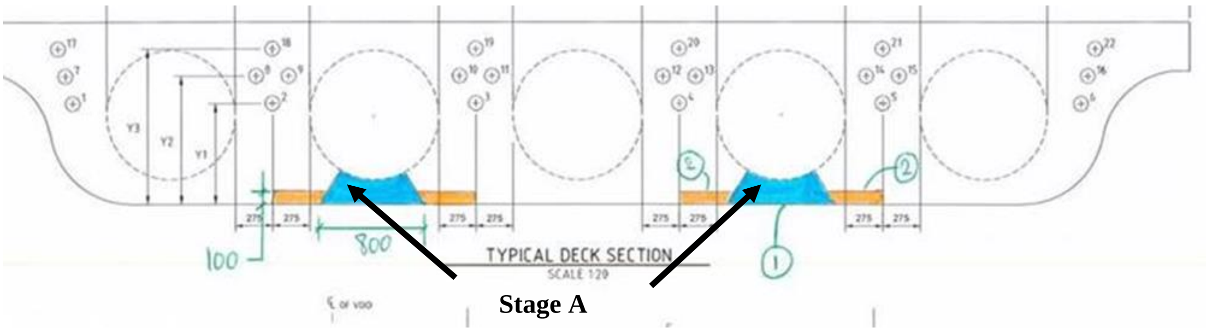

Fig.9. Stages of full soffit demolition and reinstatement

Fig.10. Debris below void former

6.2 Gap below void former

The gaps below the void formers and any interconnected voids within the bottom rebar layer were repaired by injection of a low viscosity cementitious grout that was able to flow below and around the void former.

The injection methodology included an initial investigation to establish a gap and connectivity below the void former by injecting water as initial repairs showed that curing compound egressed at various locations indication sub-surface interconnected voids and honeycombing. Following trials, injection packers were installed at a spacing of 300mm below the centre line of the void formers, sealed and grout was injected by utilising an injection pressure up to 1 bar at the injection port. The effectiveness and pressure limits of the injection were determined prior to commencement of the works using mock-ups to prevent the deformation / compression of the void former. Injection pressures in excess of 2 bar were assessed to compress the polystyrene void formers by more than 10% causing an increase of dead load and affecting the design load assumptions.

The effectiveness of the procedure during injection was verified by visual confirmation to ensure that the grout flowed from the injection port to the adjacent port and the packer was sealed once grout flowed. The challenges encountered during injection were pressure control and appropriate seals at the injection ports to mainly avoid health and safety issues.

The grout injection was also used to mitigate the risk of any honeycombed areas below the void former that could not be visually identified from the soffit. Interconnected voids within the lower reinforcement layers were sealed as well.

6.3 Hidden defects

Any hidden defects that were not identified during the inspection and repair works and the lack of external cover at the curved sections was addressed by the application of a polymer-modified cementitious coating despite initial reluctance of the contractor.

The coating provides an effective barrier to waterborne salts such as chlorides and sulfates and is able to compensate for the lack of external cover and any hidden defects to achieve the required design life of 120 years. Additionally, it provided a mitigation measure in the case of cracking of the shrinkage compensated repair material due to both loss of compressive stresses in the repair areas and inadequate curing and workmanship as had been observed.

7 Conclusions

A remediation strategy was identified with the objective to mitigate the risk of deterioration of the structure caused by reinforcement corrosion due to ingress of aggressive media within the 120 years design life as a result of the various defects observed following construction. Furthermore, it was verified that the bond between concrete and reinforcement was sufficient. Lack of significant bond would have affected the structural integrity of the reinforced concrete below the void formers and the post-tensioning ducts.

The repair methodology comprised the following:

- Removal of all visually identified honeycombed areas and surface-near debris up to sound substrate;
- Reinstatement of the broken-out defective concrete;
- Filling of the gap below the void formers and any hidden interconnected voids;
- Application of a protective coating providing a permanent barrier to airborne chlorides and sulfates to address the lack of cover at the curved sides of the deck and at other unidentified locations at the soffit.

The development of the repair strategy and methodology and its implementation on site provided various significant challenges. Despite the expected high standards in construction and materials availability it became a major challenge to identify experienced specialist contractors to undertake the works and therefore, a simple repair method such as sprayed concrete application could not be undertaken. The methodology had to acknowledge the capability of the main contractor and further simplify the repairs until it was achievable to the expected quality for the available resources. The environmental conditions during the repairs, i.e. summer in the Middle East, also played an important role and required consideration during the design stage.

The main lesson learnt during the repair process was to adjust the repair methods to the capabilities of the contractor and not to expect that novel and common standard repair techniques, that are developed in an office environment, can be applied in all projects. The time constraints imposed by the construction programme and leaving the only in-situ structure to be built after completion of all other structures in combination with the inexperience of the contractor in in-situ works played another main role in cause and rectification. Additionally, the availability of materials and simple equipment also affected the repairs.

A major lesson learnt by the contractor during the construction of the bridges was to not rely on the workmanship to ensure adequate compaction and instead to use self-compacting concrete. The use of self-compacting concrete was initially recommended but dismissed by the contractor at the beginning due to the associated higher costs. The final costs to the main contractor to complete the widening as a result of the the repairs and delays to the project can be considered significantly higher than investing a comparative insignificant amount at the initial stages to prevent costly repairs, as highlighted by Sfikas [2].

Despite the various challenges experienced during the repair, all identified defects were satisfactorily repaired and the risk of any potentially hidden defects affecting the durability within the specified design life were mitigated.

References

1. Grantham, MG.: Sustainable, durable concrete – Are specifications always fit for purpose – A case study ICCRRR2018, Cape Town. MATEC Web of Conferences.
2. Sfikas, Ioannis. (2017). Self-Compacting Concrete: History & Current Trends. Concrete (London). 51. pp. 12-16.