Suitable Intervention Strategies for Structures Affected by Alkali-Silica Reaction (ASR)

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SYNOPSIS

The purpose of this paper is to provide a review of the issues facing the Maintenance Engineer in relation to repairing bridge structures affected by alkali-silica reaction cracking. The bridge structures chosen cover problems with a superstructure and also a substructure. The superstructure example consists of an original section of 8 prestressed T girders and was subsequently widened in 1990 with 11 deck units on the Western side and 9 deck units on the Eastern side. The deck units have been identified by number for each widening starting from the upstream unit in each case. All of the cracking previously identified by others was reported as being confined to the new deck units in both widening sections. This bridge consists of 6/16 m spans supported on prestressed concrete piles. Field inspection indicates the soffit of the deck units is very close to the high water mark in the tidal creek (approx. 1000 mm clearance). This means that the deck units reside in a very aggressive environment in relation to concrete durability and the environment would be classified as Type C in relation to the Austroads Bridge Code requirements. As a result of the work performed in this report, Alkali-Silica Reaction (ASR) was determined as the primary mechanism causing the observed cracking in the prestressed deck units used in the widening of this bridge in 1990. An approach to the rehabilitation of this structure is outlined which had a degree of urgency in relation to the satisfactory long-term performance of this structure. The substructure example chosen consists of significant vertical cracking (up to 8mm in width) in prestressed piles in a marine environment. The vertical cracking was proven to have been initiated by ASR distress and subsequently widened due to corrosion of the underlying reinforcement and prestressing strands. A combination of concrete encasement below water and fibre composite encasement above water was chosen to arrest the rapid deterioration of these major supporting elements of this structure. Further developments in the repair of piles underwater using fibre composite systems will be discussed during the presentation of this paper.

1 BRIDGE WITH SUPERSTRUCTURE CRACKING IN BEAMS DUE TO ASR

1.1 General

In the 1980's, Carse et al (1) found over 100 bridges suffering ASR distress in Queensland. One structure consisted of an original section of 8 prestressed T girders and was subsequently widened in 1990 with 11 deck units on the Western side and 9 deck units on the eastern side. The deck units have been identified by number for each widening starting from the upstream unit in each case. All of the cracking previously identified by others was reported as being confined to the new deck units in both widening sections. This bridge consists of 6/16 m spans supported on prestressed concrete piles. Field inspection indicates the soffit of the deck units is very close to the high water mark in the tidal creek (approx. 1000 mm clearance). This means that the deck units reside in a very aggressive environment in relation to concrete durability and hence I would classify the environment Type C in relation to the Code requirements.

1.2 Deck Units Visual Condition

Inspection of the deck units indicated cracking was severe at each pier location and generally for 1500 mm each side of the piers (see Fig's 1 and 2). This is presumably due to the flow of water through the footpath and leakage at the pier joints. Subsequently the water will travel along the unit soffits and discharge from the units into the creek over the measured distribution length. Table 1 summarises my broad assessment of the crack widths along the Western and Eastern sides.

Table 1: Broad Assessment of Deck Unit Crack Widths at Exposed Vertical Faces

Exposed Vertical Face	Location and Crack Width (mm)				
	P5	**P4**	**P3**	**P2**	**P1**
Eastern	1.5	1.3	0.5	0.8	2.5
Western	1.5	0.9	0.3	0.7	2

Examination of the data in Table 1 shows the maximum crack width on the vertical faces of the units increases away from the central pier in each direction. The largest cracks were identified in the vicinity of the Pier 1 at the Southern end of the bridge. This structure runs in a North - South direction. The type of cracking observed was map cracking at the ends of the units and longitudinal away from the transmission length of the tendons (approx. 600 mm). This type of cracking is typical of alkali-silica cracking however the size of the maximum crack width of 2.5 mm is very severe in this case.

1.3 Previous Records

Review of previous records indicate the deck units were manufactured by Rocla Pty Ltd using Mt. Marrow basalt as the coarse aggregate source and Pioneer Jindalee medium sand as the fine aggregate. From existing knowledge this combination of aggregates would not be considered to have the potential to deliver the level of observed cracking in the deck units. A detailed assessment of the cores is required to determine the cause/causes of the observed severe cracking in these deck units.

1.4 Analysis of Concrete Cores

1.4.1 Location of Cores

Table 2 defines the location of the cores determined during the site visit as providing a representative set of cores of the range of observed distress.

Figure 1: View of severe ASR cracking in deck units

Figure 2: General view of structure and proximity to salt water

Table 2: Summary of Core Locations in Deck Units

Core No.	Widening	Deck Unit No.	Core Location
1	Left Span 1	9	Unit centreline 335 mm from Pier face (V)
2	Left Span 1	1	475 mm from end of unit and 335 mm above bottom flange (H)
3	Left Span 3	1	875 mm from end of unit and 335 mm above bottom flange (H)
4	Left Span 4	1	475 mm from end of unit and 335 mm above bottom flange (H)
5	Left Span 4	9	Unit centreline 300 mm from Pier face (V)
6	Left Span 6	1	475 mm from end of unit and 330 mm above bottom flange (H)
7	Left Span 6	1	Unit centreline 675 mm from Pier face (V)
8	Right Span 4	9	Unit centreline 650 mm from Pier face (V)
9	Right Span 2	9	475 mm from end of unit and 330 mm above bottom flange (H)
10	Right Span 2	9	Unit centreline 330 mm from Pier face (V)

Table 3 identifies the range of testing to be performed on each core.

Table 3: Scope of Testing for Cores

Core No.	Widening	Deck Unit No.	Test Description
1	Left Span 1	9	3/30 mm slices for chloride ion profile
2	Left Span 1	1	3/30 mm slices for chloride ion profile
3	Left Span 3	1	Petrographic analysis
4	Left Span 4	1	3/30 mm slices for chloride ion profile
5	Left Span 4	9	Density, carbonation and UCS
6	Left Span 6	1	Kept as a spare
7	Left Span 6	1	Petrographic analysis
8	Right Span 4	9	Spare
9	Right Span 2	9	3/30 mm slices for chloride ion profile
10	Right Span 2	9	Not taken in initial extraction

Due to the nature of the delivered cores it was not possible to obtain compressive strength results from most of the cores. However the main aims of the core analysis was to examine the penetration of chloride ions and check for alkali- silica reaction.

1.4.2 *Density, Carbonation and Compressive Strength*

Core No. 9 yielded a density of 2360 and a compressive strength of 63.0 MPa. The carbonation depth was measured as 1 mm. Hence the intact properties of the uncracked sections of deck unit indicate the correct Class of concrete was supplied to this Project at the time of construction.

1.4.3 *Chloride Ion Content*

Table 4 contains a summary of the acid soluble chloride ion contents of the measured cores.

Table 4: Chloride Ion Profiles of Cores

Core No.	**Widening**	**Deck Unit No.**	**Chloride Ion Content (kg/m^3)**		
			Depth (mm)		
			15	**45**	**75**
1	Left Span 1	9	5.13	3.4	4.3
2	Left Span 1	1	22.6	8.1	5.1
4	Left Span 4	1	6.0	4.3	3.8
9	Right Span 2	9	13.2	6.0	4.7

The allowable level of chloride ions in concrete is 0.8 kg/m^3 at the time of construction (see AS 2758.1). The threshold value for corrosion in this type of concrete is approximately 2.0 kg/m^3. At the level of the prestressing steel which is close to the 45 mm depth level given in Table 4 it can be seen that the concentration of chloride ions is 2 to 4 times the threshold value for corrosion to initiate. Hence the penetration of chloride ions into this structure has been significant over the last 10 years and their concentration at the level of the prestressing strands is now critical. Examination of Fig. 3 shows the chloride ion concentration is highest on the vertical faces of the deck units, which is consistent with the associated large crack widths.

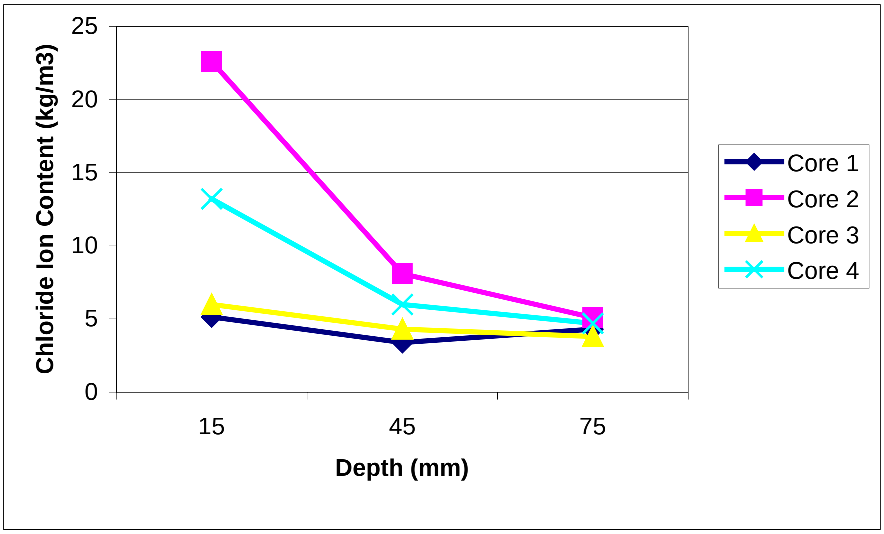

Figure 3: Deck Unit Chloride Ion Profiles

1.4.4 Petrographic Analysis

The petrographic report found that the coarse aggregate used was from a meta-greywacke source and not a basaltic source, as records appeared to indicate. Previous testing of aggregate similar to this meta-greywacke has shown it to be significantly ASR reactive. Recent discussions with other International Researchers in this field has also shown that significant ASR cracking may occur without much evidence of ASR gel in the thin section analysis. Hence it is my conclusion that ASR is the dominant cause of the observed cracking and it will continue to cause further distress in combination with the penetration of chloride ions and subsequent corrosion of the prestressing strands.

1.5 Summary

In relation to the above information the following conclusions were made:

(i) Alkali-Silica Reaction (ASR) is the primary mechanism causing the observed cracking in the prestressed deck units used in the widening of this bridge in 1990.
(ii) The coarse aggregates used were from a meta-greywacke source and not a basaltic source as previously thought. This was an important finding as the observed cracking is consistent with a meta-greywacke source and not the basaltic source.
(iii) A high level of chloride ions was detected at the level of the prestressing strands. This level is between 2 to 4 times the level required to initiate corrosion. The concentration of chloride ions is highest on the vertical exterior faces of the deck units.
(iv) The cracking in the deck units increases in magnitude away from the central pier as shown in Table 1. At each pier the cracking is most severe for approximately 2000 mm each side of the joint in the unit soffits under the footpaths. These footpaths are full of select fill which will enhance the promotion of ASR distress due to the retention of moisture.
(v) The risk of corrosion of the prestressing strands is now at a high level. Once corrosion starts in this type of material it may proceed at a very fast rate due to the phenomenon of stress induced corrosion acceleration.

1.6 Intervention Strategy

Based on the above conclusions the following intervention strategies were advised:

1.6.1 Removal of Fill in Footpaths and Sealing of Footpaths

The select fill in the footpaths needs to be removed and the footpath waterproofed. It would appear that Telstra ducts are already in use with allowance for SEQEB cables as well which may now be in place. A check of the Utilities in place is required to determine a method of fill removal without damage to the existing services. the footpath could be redesigned as a voided section with removable top slabs. The important points are that it is waterproofed to prevent water seeping through to the units at the pier joints and the services are accessible. A suitable liquid applied bitumen modified polyurethane based water proofing membrane was proposed. At the central Pier 3 expansion joint a flexible sealing system was recommended to be installed first and the liquid applied membrane sprayed to overlap with this joint system. The installation of this system will ensure appropriate movement is available at the joint without damaging the applied membrane.

1.6.2 Sealing of Large Cracks in Deck Units

The large cracks in the deck units (0.5 mm and above) should be sealed with a flexible silicone sealant prior to coating with Silane as defined in (c) below. The sealant should be gunned into the cracks and pressed home with a spatula etc. The object is not to fill all of the depth of the crack but provide a surface barrier to the macro- penetration of chloride ions. Note this crack filling should not be done until after item (a) sealing of the footpaths.

1.6.3 Coating of Deck Units

The deck units should be coated with Silane Cream product recently made available in the market place to act as a desiccator and as a chloride ion barrier. I believe the Cream is more appropriate due to environmental concerns with the close proximity of the Creek. Note the structure should not be cleaned by any water blasting methods as this will act to accelerate the ASR expansion.

1.6.4 Long Term Monitoring

After completion of the above work, the structure needs constant monitoring for assessment of the potential onset of strand corrosion and the associated rate of change of the existing crack pattern. Monitoring should be on a maximum 6 monthly basis in the first instance. Note the repair work needs to be done in the driest time of the year typically Winter/Spring and completed before the next wet season starts.

2 BRIDGE WITH SUBSTRUCTURE CRACKING IN PILES DUE TO ASR

2.1 General

This structure consists of 99/27.4m spans of prestressed I girders supported on 500/560 mm octagonal prestressed piles and lives in a marine environment. The bridge was constructed in 1979 and the fist signs of significant vertical cracking in the prestressed piles were detected in 1992. Subsequently it was determined that the distress was due to the primary mechanism of ASR followed by the subsequent ingress of chloride ions into the ASR crack network. A detailed report on the analysis of cores extracted from this structure has been reported by Carse previously (2).

2.2 Intervention Strategy

Queensland Main Roads is finding a range of bridge structures with prestressed piles cracked due to ASR distress. The intervention strategy depends on:

(i) The amount of distress that has already occurred
(ii) The environment the piles reside in
(iii) The level of the high water mark in relation to the top of the piles

In general, the target intervention time is usually early in the life of the deterioration cycle rather than after an extended level of distress has occurred (some Interstate bridges have had nearly the whole cross section eroded prior to intervention). In this particular structure, the overall level of distress was low but would build (without intervention) within 5 years of discovery to be catastrophic. Hence, the intervention strategy adopted for this structure consisted of:

(a) Concrete encase all piles below water and to 500 mm above high water level
(b) Composite fibre encase all piles to the underside of the headstocks
(c) Insert linear polarization probes in selected piles under the composite fibre encasements for monitoring of corrosion currents

The main purpose of the wrapping carried out in (a) and (b) above was to provide a durability wrap rather than additional structural capacity. The concrete encasement was chosen for the section underwater as it would stay continuously wet and not be prone to excessive restrained shrinkage cracking. The composite fibre wrap was chosen for the section above the high water mark as it would be in a wind tunnel environment and needed to remain crack free. In addition, the appearance of the repair would be enhanced by having a slender top section leading into a thicker concrete repair in the water. Figures 4 and 5 show views of the repair system and final product where the desired affect has been achieved.

Figure 4: View of Concrete and Fibre Composite Encasement of Piles

2.3 Summary

In summary, the distress in the substructure of this bridge was detected in 1992 and repaired during 1993 to 1999. The total cost of the investigation, repairs and disruption to traffic was approximately $5,000,000. Today the bridge has a replacement value of $50,000,000 and hence the maintenance expenditure of 10% of the replacement value at an intervention age of 14 years was essential and effective.

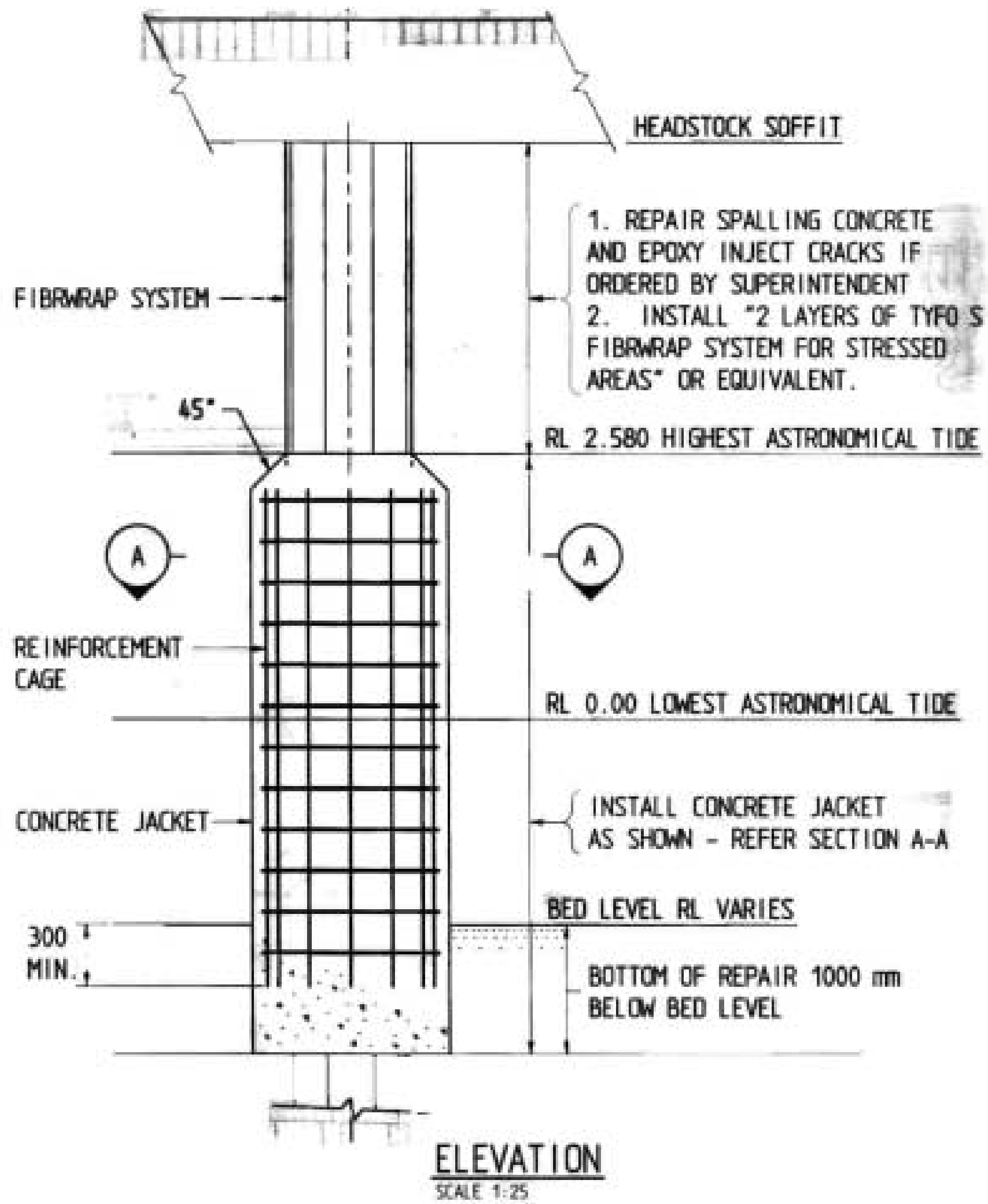

Figure 5: View of Concrete and Fibre Composite Encasement System

3 CONCLUSIONS

In relation to the information presented in this paper the following conclusions are made:

(i) Where ASR distress has occurred in superstructure beams, decks etc it is very important that an effective waterproofing membrane is applied to the top surface of the members. The earlier in the life of the affected elements this membrane is applied the better. It is detrimental to carry out waterproofing on the soffits of elements unless the coatings can breathe ie not trap water behind the coating. Water should not be allowed to pond on the superstructure and hence a free draining system is essential.

(ii) Where ASR distress has occurred in the supporting piles of a bridge early intervention is vital to ensure the long-term integrity of the supporting system.

Options include concrete encasement and/or composite fibre encasement. Some existing installations are being monitored underneath the composite fibre wraps for the potential build up of corrosion currents. The composite fibre wraps have only been used above water however; trials are currently underway with QDMR, RTA and the University of Southern Queensland investigating the use of a more substantial thickness of wrapping for underwater applications. The main advantage of this system will be the elimination of the reinforcement required in the concrete encasement solution. The composite fibre skin can be designed to deliver nominal, medium or full confinement to the deteriorated piles.

4 REFERENCES

1 CARSE A. and DUX. P. F., "Alkali-Silica Reaction in Concrete Structures", Univ. of Qld Res. Rep. No. CE88, 1988.

2 CARSE A, " The asset management of a long bridge structure affected by alkali-silica reaction", 10th Int. Conf. on Alkali-Aggregate Reaction, Melbourne, Aug. 1996, Proc. pp's 1025-1032.