

Appendix C - Technical review of alternative bridge refurbishment methodology

Technical Review of Alternative Refurbishment Methodology of Windsor Bridge



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Prepared for

SKM

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1.0 Introduction

1.1 Project

Roads and Maritime Services NSW is planning to replace the existing bridge (No 415 on MR182) over the Hawkesbury River at Windsor (known as Windsor Bridge). The 143m long bridge is over 100 years old and is deteriorating with age and is no longer considered cost effective to maintain.

The proposed Windsor bridge replacement project involves

- Construction of a new bridge over the Hawkesbury River at Windsor, around 35 metres downstream of the existing Windsor bridge
- Construction of new approach roads and intersections to connect the new bridge to existing road network.
- Modifications to local roads and access arrangements, including changes to the Macquarie Park access and connection of The Terrace.
- Construction of pedestrian and cycling facilities, including a shared pedestrian/cycle pathway for access to and across the new bridge.
- Removal and backfilling of the existing bridge approach roads.
- Demolition of the existing Windsor Bridge.
- Urban design and landscaping works, including within the parkland area of Thompson Square and adjacent to the northern intersection of Wilberforce Road, Freemans Reach Road and the Macquarie Park access road.
- Ancillary works such as public utility adjustments, water management measures and scour protection works

The Environmental Impact Statement (EIS) for the Windsor Bridge Replacement project has recently been on public display.

1.2 Proposal to Rehabilitate

One of the public submissions received from the EIS display proposes to rehabilitate the existing bridge for light traffic and provide an alternate heavy vehicle high level bridge bypassing the town.

The rehabilitation proposal consists of providing steel jackets to strengthen the aging cast iron piers and extensive patch repairs to the concrete deck units.

1.3 Scope of Technical Review

The scope of this report is to undertake a technical review of the proposed bridge rehabilitation presented by Messrs Wedgwood and Pearson in their submission related to the EIS display. The scope excludes review of the cost estimate. The proposal by Messrs Wedgwood and Pearson also includes an alternate location for a new crossing of the Hawkesbury River, the review of this proposal is beyond the scope of work of this report.

The scope covers a review of the available historical drawings, available condition assessments and load rating reports to understand the form and condition of the existing bridge.

This report reviews the proposed rehabilitation methods and comments on the

- technical merits of the solutions
- constructability of the rehabilitation with respect to impact on traffic, environment and work health and safety (WHS)
- completeness of the proposed rehabilitation with respect to the items and quantities identified in the bill of quantities
- durability of the proposed rehabilitation

2.0 Available Information

2.1 Reports

Inspection & Structural Assessment Report, Report No. C10-69-000-A, Prof. Bijan Samali, Access:UTS, (April 2011).

Graphitisation Investigation Windsor Bridge, Report No. C11231, Rev: Original, CTI Consultants Pty Ltd, (July 2011).

UW Bridge Inspection Contract 10.2535.0889 B 415 Windsor Bridge 9.5.2011 to 20.5.2011, Commercial Diving Solutions Pty Ltd (June 2011).

UW Bridge Inspection Contract 10.2535.0889 B 415 Windsor Bridge Follow-Up Inspection 15.6.2011, Commercial Diving Solutions Pty Ltd, (June 2011).

Bridge of Hawkesbury River at Windsor Load Testing — Test Report, Document No. E208R1, Rev 0, Endurance Consulting Pty Ltd, (June 2011).

Performance Load Testing of Bridge over Hawkesbury River at Windsor (BN 415), Regional Operations and Engineering Services Directorate (August 2006).

Rehabilitation of Bridge over Hawkesbury River at Windsor (BN 415 on MR 182), RTA Operations (May 2005).

Underwater Graphitisation Survey Bridge at Windsor (BN 415), Report No. C10174, CTI Consultants Pty Ltd, (April 2005).

Windsor Bridge over Hawkesbury River – Repair Cost Estimate and Life Cycle Cost Analysis, GHD (February 2005).

Inspection and Assessment Report, Bridge over Hawkesbury River on Bridge St/Wilberforce Road (MR182) at Windsor, Windsor Bridge (BN 415), RTA Bridge Section — RTA Operations, (December 2003).

Bridge Over Hawkesbury River at Windsor – Durability Condition Assessment GHD (October 2003).

Developing a Cost Effective Assessment Technique for Bridge Assets Management Phase 2, Field Testing and Assessment of Windsor Bridge, Windsor NSW, Prof B Samali, Dr J Li and A/Prof K Crews UTS, (September 2003).

2.2 Drawings

Existing drawings, 70 scanned images cover original bridge, raising, concrete deck conversion, cantilever walkway, new traffic barrier

2.3 EIS Submission

Document titled "Bridge Over Hawkesbury River at Windsor – Proposal to Repair the Existing Bridge as Part of a Scheme Involving a New Road Linking to Hawkesbury Valley Way, the Flood Evacuation Route, undated, Brian Pearson and Ray Wedgwood

Letter to RMS, Subject: Bridge over the Hawkesbury River at Windsor, 4 May 2012, Brian Pearson and Ray Wedgwood.

Transcript of Presentation to RMS Heritage Committee, 5 Sept 2012 Brian Pearson and Ray Wedgwood.

Untitled document – submission to EIS public display, undated, BJ Pearson, RJL Wedgwood.

CAWB EIS Submission – version 3.1 undated

3.0 Existing Bridge

3.1 Description and Geometry

The Windsor Bridge is a 11 span, 143m long bridge. The bridge carries two traffic lanes with a width between barriers of 6.10m and a 1.2m wide walkway on the downstream side. The original bridge was constructed in 1874 consisted of a timber deck on cast iron piers. The deck was raised by 2.4m in 1897 by bolting extensions to the cast iron piers and reconstructing the timber deck. In 1922 the timber deck was replaced with a precast concrete deck. In 1968 a cantilevered footpath supported off the piers was added to provide separated pedestrian access and accommodate utility services. The traffic barriers were upgraded in 1986.

The piers are made up of 6 foot long internally bolted 1067mm diameter segments the lower segments have been filled with bricks and gravel and upper segments with rubble. The extension to the piers in 1897 consisted of an 8 foot long segment with externally bolted flanges. The two columns of the pier extensions are joined by deep diaphragm consisting of steel I section cross bracing encased in reinforced concrete. The headstock consists of two reinforced concrete beams meeting in the middle of the bridge supported off the pier diaphragm in the centre of the bridge and support off the pier columns at the edges. The headstock was constructed in two parts to satisfy staged reconstruction.

The reinforced concrete beam and slab deck units were precast in two segments per span with a longitudinal central joint. The deck units are supported off the pier headstocks and are doweled at both ends.

3.2 Condition

3.2.1 Superstructure

Inspection reports have identified cracking, spalling and corrosion of the reinforcement on the underside of the deck units. Condition inspections have identified approximately 250m² of the bridge surface area exhibiting spalls, delamination or cracking. Exposed longitudinal reinforcement exhibits significant section loss in the external beams near the scupper locations. Reinforcement in other areas exhibits minor section loss.

The cracking and spalling has also been identified in the pier headstocks and diaphragms beams.

Durability assessment has identified extensive carbonation of the concrete deck units, with the mean measured carbonation depth exceeding the minimum cover the reinforcement at numerous locations. The durability assessment states "*General wide spread carbonation induced deterioration is likely in the near future. The damage, therefore is likely to increase with time as the carbonation front advances*"

Dynamic Frequency Analysis (DFA) undertaken on span 1 in 2003 and spans 1 to 4 in 2010 indicated a 16% reduction in the first mode natural frequency of the span 1 indicating a deterioration of condition of the span 1 over the 7 year period. The 2010 recorded natural frequency of spans 3 and 4 were lower than the results for spans 1 and 2, the lower natural frequency recording was consistent with the visual inspections that show spans 1 and 2 are in better condition than spans 3 and 4

3.2.2 Substructure

Underwater inspection reports have identified significant section loss due to graphitisation, with measured effective wall thickness ranging from 27mm to as low as 2mm. There is evidence of graphitisation along the full length of the submerged sections of the piers down to the river bed level. Circumferential cracking has also been identified in both columns of pier 5 and the downstream column of pier 6.

Significant corrosion of the bottom flange of the horizontal element of the pier bracing has also been identified.

3.3 Load Rating

The structure has been load rated, load tested and instrumented, a summary of the results are presented below.

3.3.1 Superstructure

The results of desk top load rating for “as new” and “as is condition” of the superstructure identified in the RTA Inspection and Assessment Report December 2003 are as follows

- T44 design loading – Load factor 1.48 for as new
- 42.5 Tonne Semi Trailer (ST42.5) – load factor 1.87 for “as new” and 1.45 for “as is condition”
- 62.5 Tonne B-Double Vehicle (BD 62.5) – load factor 1.87 for “as new” and 1.45 for “as is condition”
- Load testing (Aug 2006) concluded – 42.5t GVM Semi trailer be allowed to continue to cross the bridge
- Load testing (Aug 2006) of the deck units measured dynamic load allowances of 17%, this is less than code minimum values of 25% and less than the code value of 40% for short spans used in the desktop load rating of the superstructure.
- Dynamic Frequency Analysis (April 2011) concluded “the bridge in its present condition will be safe for some time” implying it will be safe until the planned replacement of the bridge.

The load ratings undertaken to date have not assessed the capacity of the bridges for High Mass Limit (HML) Vehicles or the current SM1600 design vehicles.

3.3.2 Substructure

Load rating of the piers for “assumed condition” of the substructure identified in the RTA Rehabilitation Report (2005) is as follows

- Current legal loads – Load Factor marginally above 2.0
- T44 design loading – Load Factor was less than 2.0

The load rating of the piers was based on a residual 25mm wall thickness allowing for general graphitisation minus eight, 30mm diameter x 13mm deep per recesses in the cross section to account for localised graphitisation.

Traffic plus braking was found to be the critical loading on the piers. A review of the flooding case found flooding does not govern the pile load rating, and there was no uplift on the upstream pile for the flood case.

CTI undertook additional work in 2011 which indicated the graphitisation was more extensive than had been indicated by the limited assessment prior to the 2005 report. The effective remaining section was found to be as low as 12mm compared to the 25mm used in the 2005 assessment.

Therefore in 2011 RTA undertook load testing with strain gauging of the piers. The instrumentation found good correlation between the stresses in the cast iron under vertical loading with the theoretical values from calculations. Under the braking test the change in strain was a lot less than calculated indicating that the braking forces were being transferred through the deck to the abutments rather than down the piers. The cracking around the dowels into the deck units would support the case that the deck units are restrained at the piers and are prevented from sliding freely under braking and thermal movement. Even allowing for the halving of the section thickness and hence capacity, the load factor of 2 from the 2005 work was still considered to be acceptable for current loading based on the strain gauge results indicating the horizontal braking forces from the deck are being transferred through the deck back to the abutments with the existing articulation of the bridge. However, if the bridge is rehabilitated to its original “as designed” state the articulation could change, resulting in the horizontal braking forces being transferred to the piers and not to abutments. It will then be necessary to determine whether the piers could carry these forces or if another solution to releasing the dowels is required. The flood case was not revisited on the basis the bridge was scheduled for demolition.

RTA also undertook survey of the deck to check for any evidence that the deck was settling due to deterioration of the piers. The survey results indicated that there was no evidence of settlement occurring. Based on these findings and the assessments of the superstructure no load limits were imposed on the bridge.

3.3.3 Load Rating Factors

Based on the analysis results available, the overall rating of the bridge in its current condition is around 1.45 for current loadings.

In accordance with the rating section of the bridge code (AS5100.7), bridge structures are required to have a 2.0 load factor for T44 design loading. As the load factor for this bridge for T44 and current legal loadings (ST42.5 and BD62.5) are less than the required 2.0, RMS have put in place an intensive inspection program (including weekly inspections) to manage the risks associated with a bridge with a load factor less than 2.0.

4.0 Proposed Rehabilitation

4.1 Understanding of the Proposed Rehabilitation

4.1.1 Substructure

Extract from EIS submission by Messrs' Wedgwood and Pearson.

"It is envisaged that the external plates would be in half section semi-circles with flanges for making a bolted connection between the half sections. Neoprene packing would give a uniform tight fit to the cylinders. The strengthening would only need to go for the depth of the cylinder that is severely affected by graphitization. The depth required for strengthening the cylinders would need to be determined (assumed 3.35m).

It is envisaged that 16mm thick plates would be satisfactory for the strengthening covers"

4.1.2 Superstructure

Extract from EIS submission by Messrs' Wedgwood and Pearson.

"It is envisaged that the underside of the reinforced concrete deck could be restored by:

- a) Using high pressure water blasting of the underside surfaces from barges under the deck to ensure traffic using the deck is not disrupted;*
- b) Inspecting the reinforcement for possible loss of cross sectional area and determining if supplementation of the reinforcement is required;*
- c) If supplementation of the reinforcement is required it can be readily achieved by using carbon fibre epoxy bonded to the final concrete surface;*
- d) Replace the blasted concrete using "gunniting" or "shotcreting" process;*
- e) Provide a protective coating to the repaired and/or strengthened concrete"*

5.0 Technical Review

5.1 Substructure

5.1.1 Structural

Jacketing of the piers to reinstate the structural capacity of the piers to their original capacity can be achieved by the proposed bolting of semi-circular steel shells around the existing pier columns. As the original column segments are internally jointed, the proposed sleeve solution would not be fouled by the original column joints.

The steel jackets will provide structural strengthening to compensate for the loss of cast iron due to graphitisation. We are of the opinion that it will be necessary to grout the annulus between the irregular surface of the existing piers and the inside face of steel jacket to achieve effective load transfer into the steel jacket. The steel jacket should be taken, say one times the pile diameter beyond the point of need to achieve load transfer, this would need to be confirmed during detailed design. The provision of a grout annulus also has the added benefits of protecting the inside face of the steel jacket and preventing further graphitisation of the cast iron.

To minimise the visual impact it would be preferable to start the casing at the low water line and extend down to just below (say 0.5m) the river bed level. The jackets may need to extend above the low level line depending on the extent of the graphitisation in this area, measurements taken approximately 0.7m below the water level at the time of the underwater inspection show significant section loss. Extending the jackets one diameter above this level would put the top of the casing around 0.3m above the low water mark. Extending the jackets above the low water level will impact on the pier cross bracing that would need to be modified. There were no thickness measurements recorded in the tidal zone, so additional testing would be required to establish the termination level of the jackets. Inspections of the casting above the high water mark indicate the cast iron is in excellent condition with minimal section loss, so jacking above the high water mark is unlikely to be required. The river bed is around 5m below the low tide level. One set of cast iron wall thickness measurements taken at the river bed level at pier 5 indicate a residual wall thickness of 21mm, this result is significantly thicker than the results higher up the pier. There was evidence that the softer graphitised material has been eroded off the pier at the river bed level. Therefore it is recommended that the jacket is taken a minimum of 0.5m below the river bed level to limit future graphitisation and abrading of the cast iron. Additional investigations would be required to establish the extent of section loss below the river bed to determine the final termination level of the jackets. It is anticipated a jacket length required will be in the range of 5 to 6m, with an average of say 5.5m. A 16mm thick jacket is considered sufficient for structural capacity; an allowance for corrosion needs to be added to this thickness, taking the thickness to 20mm.

5.1.2 Durability

The proposed jacketing solution will provide a long term solution to the repair of the piers. Water sampling results contained in the underwater inspection report indicate the water has a low chloride content and is considered fresh at this location. It is understood in periods of drought, brackish water may be found at the bridge site. An allowance of 0.025mm/year corrosion allowance for the external face of the steel jacket would require a 2.5mm additional wall thickness over a 100 year design life. Grouting of the annulus between the jacket and the cast iron will enhance passivation of the steel and cast iron surfaces and prevent further graphitisation of the cast iron. The outer surface of the jackets should be coated with a protective coating to enhance durability.

Repainting of the cast iron piers above the jacket level and the steel cross bracing will be required to preserve the cast iron and steel work. Repainting would be required approximately every 15 - 20 years.

5.1.3 Constructability

The jacketing of the piers can be undertaken without any impact on the traffic using the bridge.

5.2 Superstructure

5.2.1 Structural

The Wedgewood / Pearson repair methodology for the deck units to maintain “as is” condition providing a load factor of approximately 1.5 on current traffic loadings (42.5 tonne semi trailers and 62.5 tonne b-double vehicles). Repair of the deck to achieve an “as new” condition would increase the load factor on current loading to 1.87. Strengthening of the deck units for flexure would be required to bring the bridge up to a load factor of 2.0 for current loading and T44 design vehicle for code compliance. The refurbishment in combination with the proposed bypass would provide an alternate heavy vehicle route and a load restriction could be placed on the refurbished bridge for light vehicles only. However to reduce current inspection and monitoring requirements, the refurbishment of the bridge would need to achieve a higher load factor for current and T44 design loading. Acceptance of a load factor of less than 2.0 for the refurbished structure would require appropriate risk assessment and risk management procedures.

The load rating of the superstructure undertaken by RMS has indicated strengthening for flexure is required to achieve a load factor of 2.0. The beams have a load factor greater than 2.0 for shear and do not require shear strengthening. Strengthening for flexure could be achieved by bonding high strength carbon fibre laminates to the underside of the beams of the deck. The use of carbon fibre would not distract from the current appearance of the bridge. However there is a risk associated with bonding carbon fibre strips to the beam soffit areas not replaced with concrete patch repairs. The strips would be bonded over the top of areas of potential corrosion activity (un-patched areas) and these areas may over a period of time crack, spall or delaminate losing the effectiveness of carbon fibre. Therefore if carbon fibre strengthening is to be undertaken it is recommended that it be undertaken in conjunction with re-alkalisation and patch repairs.

The connection between the deck units and the pier headstocks consists of a doweled connection. The drawings call up a 48mm diameter anchor at each end of each girder of the deck unit. There is no detail to show how this was achieved with the precast deck units. The girders of the deck units sit directly on the headstock concrete without bearings. It appears the dowels are locked up with diagonal cracking identified at the support locations, these cracks may have been caused by the restraint of thermal expansion and contraction movements. The braking load test undertaken on the bridge supports the theory that the bridge is locked up with negligible bending stresses recorded in the piers under the tests indicating the horizontal forces are being shared by all the piers or being transferred back to the abutments. It would be difficult to remove and replace the dowels, as this would involve coring out the existing dowels and casting new dowels into the headstocks fitted with rubber o-rings and conduits that are grouted into the deck unit to provide rotation and translation capacity to prevent the build up of stresses. The difficulty with replacing the dowels is preventing damage to the existing reinforcement in the deck units and headstocks. A detailed assessment of the bridge articulation would need to be undertaken to determine the most appropriate solution to address the current diagonal cracking. Alternative solutions that provide a stiffer alternative load path for longitudinal and transverse forces to reduce the stresses imposed on the dowels could be developed as part of the detailed design of the strengthening works. The cost estimate for the rehabilitation should provide some allowance for the cost to solve this issue.

Geometrically the width between barriers of 6.1m for the two traffic lanes remains unchanged under the proposed rehabilitation, This width is less than current standards.

5.2.2 Durability

Repairs to the spalled concrete and corroded reinforcement using pressure blasting and scrubbing tools is an established method to remove defective concrete. Wire brushing and grit blasting can be used to remove corrosion product off reinforcement. Cementitious patch repair mortars can be effectively applied by hand for small areas and using methods such as shotcreting for large areas.

It is noted that the condition investigation conducted by GHD was performed in August 2003 and therefore the information and conclusions relate to condition of the structure more than 9 years ago. Considerable continued deterioration would be expected to have occurred since the GHD and RTA reports were written. The extent of additional deterioration that has occurred since 2003 would give a good indication of the rate of deterioration and the expected increase in repair quantities, which in turn could lead to an alternative repair solution being more cost effective than re-alkalisation.

The effectiveness of any repair is influenced by the basic quality of the materials within the structure. The core data indicates the beam and headstock concrete have average compressive strengths of 37 MPa and 50 MPa respectively. These results would suggest that the strength indicates a high quality of concrete considering it was cast in 1922 which is encouraging for the continued performance of the bridge after repair. Under dry conditions, it would be expected such a concrete to have a relatively high resistivity which would also limit the rate of corrosion.

The observation that the majority of the spalling and delamination on the external beams was adjacent to drainage holes demonstrates that the presence of water is a key component in the observed deterioration on this particular structure. The corrosion rate of reinforcing steel within carbonated concrete is often very low and the propagation phase before cracking occurs can be very long. This is usually because of the limited moisture available to promote corrosion. Damp environments lead to saturated concrete which would not be expected to have significant carbonation. The environmental conditions on the Windsor Bridge allow carbonation with intermittent saturation to accelerate corrosion of the depassivated reinforcement due to periodic flooding and surface runoff during rain events.

Installing properly sealed downpipes into the existing drainage holes in the deck to a new longitudinal stormwater drainage pipe suspended on the underside of the slab between the beams draining back to a collection point at each abutment is required. This would prevent drainage water spilling onto the beams and soffit of slab, slowing the rate of corrosion damage adjacent to the existing deck drainage holes. Waterproofing of the wearing surface is also recommended to reduce moisture ingress into the deck units. This would involve removing the existing asphalt surface, applying a sprayed waterproofing membrane and relaying a new asphalt surfacing. As part of the waterproofing of the deck, repairs to the joints between the deck units at the piers will be required. This will involve breaking out defective concrete and repairing. It is also required to create a recess 20mm x 20mm at the top of the deck units to accept a poured joint sealant. A bituminous tape is placed over the joint before reinstating the asphalt surfacing.

Two approaches to the repairing of the concrete elements of the superstructure are discussed in the GHD report. The first option is a conventional patch repair solution; the second option is a patch repair plus re-alkalisation solution.

5.2.2.1 Conventional Patch Repair

For a patch repair and coating option, GHD report states that conventional patch repair “entails removal of all delaminated and loose concrete behind reinforcement and along reinforcement until it is not corroded. This is likely to result in removal of approximately 2-3 times the concrete surface currently identified as cracked/spalled or delaminated. This option is deemed to require cyclic patch repair every 3-5 years” which is based on the assumption that the anti-carbonation coating would not stop the reinforcement corrosion in areas where the carbonation exceeds the cover depth”.

The assumed limited longevity of such repairs is the main reason that the patch repair option is not considered appropriate. If the patch repair option were to include a sacrificial anode to limit the incipient anode effect and a silane treatment prior to application of the anti-carbonation coating, we are of the opinion that a patch repair system could provide an appropriate cost-effective repair option provided the coating system prevents water ingress. Migrating corrosion inhibitors could also be used to further enhance the durability of such an option.

It is unlikely that such an option would not have local defects that may require future repair. A visual survey should be conducted say 10 years after repair to determine any additional areas of cracking/spalling. These should be repaired before reapplication of the anti-carbonation coating.

The patch repair option would not return the reinforcement to a passive state as anticipated in the re-alkalisation option. Nonetheless we are of the opinion that a properly conducted repair and coating system should provide acceptable performance of the structure for an additional service life of 25 years with limited maintenance.

The scope of works for a patch repair solution would include.

- Contain and remove defective concrete in accordance with environmental and WHS requirements
- Breakout of defective concrete to a depth of approximately 80mm to allow for the patch repair to envelope the longitudinal reinforcement. Clean and treat corroded reinforcement including the installation of sacrificial anodes
- Apply repair mortar
- Apply silane coating
- Apply anti-carbonation coating

Periodic maintenance (every 10 to 12 years) would include

- Breakout and repair defective / spalled concrete including treating of reinforcement
- Reapply silane treatment to new repairs
- Reapply anti-carbonation coating to entire surface

5.2.2.2 Re-alkalisation Plus Patch Repair

The second option to repair the deck concrete involves a patch repair plus re-alkalisation. The patch repair area would be similar to the first option however the depth of the concrete to be removed would be less as there is no requirement to envelope the existing reinforcement. The re-alkalisation should restore the reinforcement to a passive state and therefore would be a technically superior option albeit more expensive solution. An important concern for the use of this technique is the possibility of slowly reactive aggregate indicated by the uranyl acetate test. The benign reactivity of the aggregates used in the concrete should be confirmed by petrographic examination. Alternatively more expensive lithium hydroxide or similar may be used as the electrolyte which would not cause any Alkali Silica Reactivity (ASR) problems. A potential side effect of re-alkalisation is that residual electrolyte compounds can crystallise out and cause debonding of the coating. Care needs to be taken to ensure effective surface preparation to ensure the longevity of the coating.

The scope of works for a re-alkalisation plus patch repair solution would include:

- Contain and remove defective concrete in accordance with environmental and WHS requirements
- Breakout of defective concrete to a depth of approximately 50mm depth to expose the corroded longitudinal reinforcement
- Clean and treat corroded reinforcement
- Apply repair mortar
- Apply re-alkalisation
- Apply anti-carbonation coating

Periodic maintenance (every 12 to 15 years) would include:

- Breakout and repair defective / spalled concrete including treating of reinforcement (expected to be minor under this option)
- Reapply anti-carbonation coating to entire surface

Both the patch repair and coating option or patch repair plus re-alkalisation and coating option would provide suitable solutions to extend the life of the Windsor Bridge for 25 or more years. The former would be expected to require future repair of localised areas of cracking/spalling. We would suggest that this should be conducted after 10 years and prior to the reapplication of the anti-carbonation coating. Reapplication of the anti-carbonation coating would be required every 10-12 years.

Re-alkalisation would be expected to provide a long-term solution greater than 50 years that would not require reapplication of the re-alkalisation provided the anti-carbonation coating was properly maintained (i.e. reapplication of the anti-carbonation coating every 10-12 years).

5.2.3 Constructability

The majority of the defects identified in the bridge superstructure have been identified on the underside of the bridge. The majority of the work to clean and repair these defects could be carried out under traffic from the underside of the bridge in accordance with environmental and WHS requirements. To provide access to the underside of the deck and contain the waste a platform could be provided to span between the piers to avoid placing additional loads on the deck units. The platform would need to incorporate an enclosure, drainage system and sump to contain the waste from the high pressure water blasting used to remove the defective concrete for the repairs to the deck soffit.

Depending on the properties of the patch repair material some restriction on traffic movements over the deck may be required while the repair mortars are setting. Traffic restrictions could include reduced speed limits and overnight lane closures.

Repairs to the joints at the ends of the deck units would need to be undertaken during night lane closures. High early strength repair mortars and possibly road plates would be required to enable early trafficking of the repairs to minimise traffic impacts.

Waterproofing of the deck could be undertaken as part of general maintenance during planned resurfacing of the asphaltic wearing surface.

6.0 Quantities

Basic quantities for the refurbishment are presented in the table below. Items identified with an asterisk indicate the item was not specifically included in the cost estimate provided in the Messrs Wedgewood and Pearson submission.

Substructure	Qty	Unit
Preparation - Water blast clean existing piers		
Area	369	m2
Steel Jackets		
Total Weight	75	tonne
Protective Treatment Exterior Surface of Steel Jackets		
Area of Protective Coating	565	m2
* Protective Treatment Exterior Surface of CI Pier above Jackets		
Area of Protective Coating	670	m2
* Replacement of Horizontal Bracing Element		
Total Weight including clamps (galvanised)	2.4	tonnes
* Grout Annulus in Steel Jackets		
Total Volume Grout	11.4	m3
* Pier Diaphragms		
Repair Defective Diaphragm Concrete	75	m2
Repair / treat defective reo	75	m2
Silane treatment - Deck Units	277	m2
Anti carbonation coating - Deck Units	277	m2
Articulation		
Rectification of bridge articulation		Item
Superstructure		
Underbridge access and containment platforms		Item
Repair Deck Units - Patch Repair Option		
Repair Defective Deck Concrete (80mm depth)	750	m2
Repair / treat defective reo	750	m2
* Silane treatment - Deck Units	2611	m2
* Anti carbonation coating - Deck Units	2611	m2
Repair Deck Units - Re-alkalisation Option		
Repair Defective Deck Concrete (50mm depth)	750	m2
Repair / treat defective reo	750	m2
* Re-alkalisation - Deck Units	2611	m2
* Anti carbonation coating - Deck Units	2611	m2
* Repair Deck Joints		
Breakout Defective Deck Concrete	4	m3
Repair / treat defective reo	40	m2
Joint Sealant	73.2	m
* Repair Headstocks		
Repair Defective Headstock Concrete	50	m2
Repair / treat defective reo	50	m2
Silane treatment - Deck Units	195	m2
Anti carbonation coating - Deck Units	195	m2
* Drainage		
Install scupper downpipes	44	each
Longitudinal drainage pipe	280	m
* Deck Waterproofing		
Remove existing surfacing	897	m2
Waterproofing layer plus AC	897	m2

7.0 Conclusions

The proposal to rehabilitate the Windsor Bridge using steel jackets and deck concrete patch repairs is considered technically viable provided future maintenance is undertaken to maintain the integrity of the repairs.

Key points of the review are summarised below.

- To provide an effective load transfer between the steel jackets and the existing cast iron piers it is recommended that a grout annulus be included in the jacket design. This will also provide protection to the remaining cast iron material and protection to the inside surface of the steel jacket.
- The steel jackets should start 0.5m below the river bed and extend to just above the low water level, the estimated average length of the jacket is 5.5m. Additional testing of the remaining cast iron thickness in the tidal zone and just below the river bed will be required to establish the final length of the jacket.
- An allowance for long term corrosion should be incorporated into the design of the wall thickness of the jackets in addition to providing a protective coating to the exterior surface of the jackets.
- The cast iron piers above the jackets and the steel cross bracing will need to be painted on a regular basis approximately every 15 years.
- A patch and repair approach to the concrete defects of the bridge could provide a short to medium solution, (greater than 25 years) if combined with a silane and anti-carbonation coatings, plus deck water proofing. Additional measure such as sacrificial anodes, migrating corrosion inhibitors could be used to improve durability and reduce future maintenance of the concrete elements. Anti-carbonation coatings will need to be re-applied every 10 to 12 years.
- A patch and repair plus re-alkalisation approach to the concrete defects of the bridge if proven to be cost effective would significantly reduce future patch repairs of the structure and would be expected to extend the life of the structure to beyond 50 years. The anti-carbonation coating would need to be reapplied every 10 to 12 year.
- Modification to the current scupper arrangement is required to prevent scupper water splashing on the deck units; this will include the installation of droppers and a longitudinal underdeck piped drainage system.
- Joint repairs, waterproofing and resurfacing of the deck would require night lane closures
- In terms of traffic related impacts, the majority of the repairs to the structure can be undertaken with little impact on traffic using the bridge. Surface repairs to the deck joints will require night lane closures and road plates while the repairs are undertaken. An appropriately designed containment system would be required to prevent the waste products from the repairs entering the river.
- The load factor of the rehabilitated bridge assuming the refurbishment brought the bridge back to a “as new” condition would be 1.87 for current legal loadings (42.5 tonne semi-trailer and 62.5 tonne B-double vehicles). This is less than 2.0 required by current bridge design standards. Acceptance of a load factor of less than 2.0 for the refurbished structure would require appropriate risk assessment and risk management procedures.
- Strengthening of the bridge to achieve load factors of 2.0 for current legal loadings and T44 design vehicles could be achieved through the use of carbon fibre strengthening of the deck beams. Carbon fibre strengthening is only recommended in conjunction with the re-alkalisation repair option.
- The load ratings undertaken to date have not assessed the capacity of the bridges for High Mass Limit (HML) Vehicles.
- A load factor of 2.0 on T44 design vehicle may not be considered necessary if a heavy vehicle bypass bridge was construction and the bridge restricted for use for light traffic only.
- The existing 3.05m traffic lanes are less than current road design standards, the lane width remains unchanged under this refurbishment proposal.