

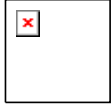
Roads and Traffic Authority

Bridge over Hawkesbury River at Windsor

Durability Condition Assessment

Report





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Contents

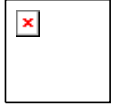
Executive Summary	i
1. Introduction	1
1.1 Background	1
2. Scope of Work	1
2.1 Visual and Delamination Survey	1
2.2 Diagnostic Testing	1
3. Results and Discussion	2
3.1 Visual and Delamination Survey	2
3.2 Diagnostic Testing	3
3.3 Discussion	7
4. Repair Options	8
4.1 Repair Options	8
5. Conclusions and Recommendations	10
5.1 Conclusions	10
5.2 Recommendations	11

Table Index

Table 1	Summary of Diagnostic Testing Results	4
Table 2	Reinforcement Section Thicknesses	6
Table 3	Compressive Strength of Retrieved Concrete core Samples	7

Appendices

Defect Mapping Survey Drawings
Photos
Background to Diagnostic Testing
Laboratory Test Certificates



Executive Summary

GHD Pty Ltd (GHD) was commissioned by the Roads and Traffic Authority (RTA) to undertake a durability condition investigation of the bridge over the Hawkesbury River at Windsor.

The scope of work was to carry out a defect mapping survey of the bridge, perform a limited delamination survey and diagnostic testing of the bridge elements and provide a technical report presenting the findings of the investigation and outlining the remedial options.

The condition investigation was performed in August 2003.

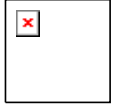
A visual survey of the visible above ground concrete elements was performed using boat access. The delamination survey was performed over approximately 30% of the bridges beams and headstocks using a cherry picker mounted on a floating barge.

Prior to this, the RTA undertook a separate detailed visual inspection of the entire bridge using the cherry picker and floating barge. The RTA's visual inspection records were provided to GHD for inclusion in the report.

Diagnostic testing was undertaken at three details areas and included a covermeter survey, depth of carbonation measurements, chloride content analysis, alkali silica reactivity (ASR) testing, reinforcement section thickness and compressive strength testing.

The condition investigation concluded that:

- ▶ Significant visible spalling was evident on the soffits of the external beams (B1 and B8) with a majority of the spalling/delamination located adjacent to drainage hole (scupper) locations (refer to drawings in Appendix A).
- ▶ Regular vertical cracking at stirrup locations was present on a majority of beams.
- ▶ Significant quantity of horizontal cracking on both external and internal beams (B1-B8) at the level of longitudinal reinforcement was evident.
- ▶ Ends of each beams generally cracked horizontally from base dowelled fixing to headstocks.
- ▶ Approximately 250 m² of the bridge surface area was spalled/delaminated/cracked.
- ▶ The mechanism responsible for the corrosion is carbonation of the cover concrete. The mean (as measured) carbonation depth exceeded the minimum cover and approached/exceeded the mean cover at numerous locations. General widespread carbonation induced deterioration is likely in the near future. The damage, therefore, is likely to increase with time as the carbonation front advances.
- ▶ Exposed reinforcement exhibits significant section loss on the internal main longitudinal reinforcement in the external beams (i.e. at scupper locations). Reinforcement in other areas exhibits only minor section loss.



- ▶ Chloride contents are low to negligible and chloride induced reinforcement is not likely to be a concern for the remaining service life of the bridge.
- ▶ Although ASR gel was present in two of the three cores tested, we do not consider that ASR is a deterioration mechanism of the bridge.

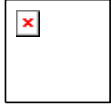
Three repair options have been reviewed for their applicability/appropriateness to the bridge:

- ▶ Option 1: Conventional Patch Repair, application of a protective surface coating and installation of a Sacrificial Anode System;
- ▶ Option 2: Realkalisation;
- ▶ Option 3: Cathodic Protection.

Realkalisation is recommended as the most technically appropriate repair method. It is also considered that Realkalisation will be a more cost effective repair over a future service life of 25 years.

The preferred remedial option should be implemented as soon as practicable. Significant delays in repair may result in increased costs due to higher levels of deterioration.

Prior to selection of the repair option(s), it may be beneficial for the RTA to undertake an engineering cost estimate and a life cycle cost analysis for the above-mentioned repair options (including sensitivity variations). The most appropriate repair option may then be chosen from both technical and economical considerations.



Introduction

GHD Pty Ltd (GHD) was commissioned by the Roads and Traffic Authority (RTA) to undertake a durability condition investigation of the bridge over the Hawkesbury River at Windsor.

The scope of work was to carry out a defect mapping survey of the bridge, perform a limited delamination survey and diagnostic testing of the bridge elements and provide a technical report presenting the findings of the investigation and outlining the remedial options.

The condition investigation was performed in August 2003.

Background

The Bridge over the Hawkesbury River at Windsor was built in the early 1900's and is an 11 span bridge with simply supported reinforced concrete beams, with headstocks (cross beams) supported on steel piles.

The drawings in Appendix A show the general arrangement of the bridge.

A site inspection by RTA revealed that the bridge is suffering from reinforcement corrosion-related deterioration.

Scope of Work

Visual and Delamination Survey

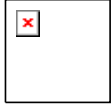
Visible deterioration such as cracking, rust staining and spalling was recorded in the visual survey.

The delamination survey involved 'sounding' the surface of the concrete with a hand-held hammer. The purpose of a delamination survey is to detect sections of delaminated concrete that are visibly "sound". Typically, delaminated concrete is caused by corrosion of reinforcement.

Diagnostic Testing

The primary purpose of diagnostic testing is to determine the mechanism(s) responsible for deterioration. Only after determining the deterioration mechanisms can appropriate remedial strategies be developed.

Diagnostic testing also indicates the extent of corrosion activity, which typically, is considerably greater than the extent of visible deterioration. Furthermore, from an analysis of the test results within undamaged areas it is possible to estimate the likely extent of future deterioration.



The diagnostic testing was carried out in Span 5 and Span 6 of the bridge at a total of three detail areas. The areas were selected to be representative of the different element types.

The following diagnostic testing was carried out:

- ▶ Covermeter survey (at selected locations);
- ▶ Depth of carbonation measurements (12 No of locations);
- ▶ Chloride content analysis (3 Number of locations at 3 depth increments per location);
- ▶ Alkali Silica Reactivity (ASR) testing (3 No of cores);
- ▶ Reinforcement section thickness (5 No of locations, 9 measurements); and
- ▶ Compressive strength testing (9 No of cores).

The drawings in Appendix A show the areas of visible deterioration and delamination including test locations on various elements.

The results of the diagnostic testing are provided in Section 3. Background to diagnostic testing is provided in Appendix C.

Results and Discussion

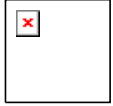
Visual and Delamination Survey

GHD undertook a visual survey of the visible above ground concrete elements using boat access. Prior to this, the RTA undertook a separate detailed visual inspection of the entire bridge using a cherry picker mounted on a floating barge. The RTA's visual inspection records were provided to GHD for inclusion in the report.

The delamination survey was performed over approximately 30% of the bridges beams and headstocks using the cherry picker mounted and floating barge. Drawings of the defect locations are provided in Appendix A. The results of the visual and delamination survey were used as a basis for the estimation of repair quantities.

The defect records suggest that:

- ▶ Significant visible spalling was present on the soffits of the external beams (B1 and B8).
- ▶ A majority of the spalling/delamination was located adjacent to drainage hole (scupper) locations.
- ▶ The extent of deterioration appears to be greater in the middle spans.
- ▶ Regular vertical cracking at stirrup locations on all beams.



- ▶ Significant quantity of horizontal cracking on both external and internal beams (B1-B8) at the level of longitudinal reinforcement, possibly indicating corrosion of reinforcement.
- ▶ Exposed reinforcement exhibits varying degrees of corrosion.
- ▶ Ends of each beams generally cracked horizontally from base dowelled fixing to headstocks.

Photos in Appendix B shows the defects observed on the bridge's concrete elements.

Diagnostic Testing

Table 1 summarises the diagnostic testing results for each of the three detail areas for the bridge. Discussion of the results is included in the following sections.

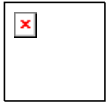
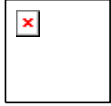


Table 1 Summary of Diagnostic Testing Results

Location	Element	Depth of Concrete Cover to Stirrups (mm)	Depth of Carbonation (mm)	Test Depths (mm)	Chloride Content (% by weight of concrete)	ASR Reactivity	Comments
Span 5	Beam 1	13-20 side face	14-25	0-10	0.01	Positive	Depth of carbonation exceeds minimum cover Corroding rebar present
		20-25 bottom	9-28	10-30	0.01		
			12-22	30-50	0.01		
Span 5	Beam 2	18	24 side	Not tested	Not tested	Not tested	Depth of carbonation exceeds minimum cover Vertical cracks at stirrups
			19 bottom				
Span 6	Beam 1	15-20 side face	13-20	0-10	0.01	Positive	Depth of carbonation exceeds minimum cover Corroding rebar present
		20-25 bottom	20-25	10-30	0.01		
			20-25	30-50	0.01		
Span 6	Beam 2	22	15 side	Not tested	Not tested	Not tested	Carbonation less than cover
Headstock		20-30 side face	6-16	0-10	0.05	Negative	Depth of carbonation at minimum cover Concrete generally sound
			10-24	10-30	0.01		
			8-22	30-50	0.01		



Covermeter Survey

Table 1 summarises the depth of cover results for the bridge.

For the bridge beams the measured cover to the stirrups varied between 13 mm and 25 mm. In general, the measured cover was between 15-20 mm for the sides of the beams and 20-25 mm on the bottom of the beams. The main longitudinal reinforcement was located inside these bars at approximately 25-30 mm cover.

For the bridge headstocks, the measured cover of the stirrups varied between 5 mm and 30 mm, but was generally greater than 25 mm.

Depth of Carbonation

The depths of carbonation measured at breakout and core locations are provided in Table 1.

For the bridge beams, the measured carbonation depth varied between 10 mm and 30 mm. The calculated mean carbonation from 12 measurements is 23 mm. Therefore the mean (as measured) carbonation depth currently exceeds the minimum cover and is at the mean depth of cover. In the test and spalled locations, the maximum carbonation depth exceeds the mean cover. This is supported by visible damage on the bottom corners of the beams.

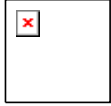
For the bridge headstocks, the measured carbonation depth varied between 6 mm and 24 mm. Therefore, like the beams, the mean (as measured) carbonation depth has reached the minimum depth of cover. This is supported by visible damage on site.

Chloride Content Analysis

Chloride content was determined in accordance with BS1881: Part 124:1988 "Methods for Analysis of Hardened Concrete" Section 10.2. The results of the chloride content analysis are given in Certificate of Test No. 4063 in Appendix D.

Generally, uncontaminated concrete has a background chloride content of up to 0.01-0.02% by weight of concrete. At all but one of the locations tested the chloride content was 0.01% by weight of concrete. The results indicate an absence of chlorides in the environment or the original concrete mix.

A typical chloride profile for concrete in a salt -water marine environment is to have a high chloride content in the surface of the concrete, which reduces with depth. The Hawkesbury River at Windsor is a freshwater river and thus the chloride content results from the tests are to be expected.



Alkali Silica Reaction

The results of the alkali silica reaction analysis are given in Certificate of Test No. 4062 in Appendix D. Detection of alkali-silica reaction (ASR) was by UV-florescence staining using uranyl acetate. Note that this test detects if ASR gel is present or not, but does not suggest the likely reactivity of the aggregates.

Two of the three tests indicate the likelihood of ASR product (gel) in the bridge concrete. This was not conclusively supported by visual evidence on site as a majority of the cracking present could be attributed to carbonation and may be shrinkage effects.

Reinforcement Section Thickness

Section loss of the corroded reinforcement was measured at selected locations to obtain a worst-case indication of the loss of section. The most heavily corroded bars were generally immediately below the scuppers. These are specifically the bottom corner bars on the internal face of the external beams. Other reinforcing bars indicated a section loss of up to approximately 1 mm. The original diameter of the main steel is likely to be 32 mm and that of the stirrup is 12.5 mm.

Table 2 Reinforcement Section Thicknesses

Location on Beams 1	Element	Remaining Reinforcement Section Thickness (Worst Case) (mm)
1: Span 5	Main Bar	29.5, 28, 29
2: Span 5	Main Bar	29.5, 28
3: Span 6	Main Bar	28, 30
4: Span 5	Stirrup	10
5: Span 6	Stirrup	11

Compressive Strength Testing

Compressive strength testing was performed on retrieved concrete core samples taken from the beams and headstocks. Six samples were taken from the external beams (3 from Span 5 and 3 from Span 6) and three samples taken from the headstock on Pier No. 5.

Testing was undertaken in accordance with AS1012.14 after dry conditioning for 7 days.



Table 3 Compressive Strength of Retrieved Concrete core Samples

Location	Element	Compressive Strength (MPa)
1: Span 5	Beams	35.0
2: Span 5	Beams	37.0
3: Span 5	Beams	37.0
4: Span 6	Beams	39.5
5: Span 6	Beams	34.0
6: Span 6	Beams	38.5
7: Pier 5	Headstock	49.5
8: Pier 5	Headstock	53.0
9: Pier 5	Headstock	47.0

The compressive strength results presented in the above were corrected for length/diameter ratio and AS3600 recommendations.

The beam concrete strength varied between 34 and 39.5 MPa (mean 37 MPa).

The headstock concrete strength varied between 47 and 53 MPa (mean 50 MPa).

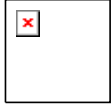
Compressive strength results are considered to be reasonable for age of the bridge.

Discussion

The diagnostic test results confirm that the spalled and delaminated areas identified in the visual and delamination survey of the soffit of the beams are the result of carbonation-induced corrosion of reinforcement. Although ASR gel was present in two of the three cores tested, we do not consider that ASR is a deterioration mechanism of the bridge. Had ASR been a deterioration mechanism of the 100 or so years old bridge, map cracking characteristic of ASR would have been evident.

For the bridge beams, the measured cover to stirrups varied between 13 and 25 mm. At most of the locations tested, the mean depth of carbonation exceeded the mean measured cover. Therefore, though the current visible spalling is relatively limited, the RTA's defect records indicate that the total amount of damage is extensive. Therefore, a large proportion of the bridge is likely to be suffering from corrosion-induced corrosion due to carbonation of the cover concrete.

The greater relative quantity of damage on the external beams (B1 and B8) is most likely due to increased exposure to moisture from runoff over the sides of the bridge and drainage through scuppers. The most severe corrosion and spalling is generally located on the bottom of the external beams either side of the scupper. The dirt and debris, which has built up immediately below the scuppers, seems to have 'protected'



the concrete slightly at this location, shifting the greater corrosion activity to either side of this area.

Notwithstanding the majority of the visual damage being on the external beams, the internal beams (B2 to B7) are showing signs of significant corrosion-induced cracking which is likely to result in spalling in the future. It is likely that the propagation of damage has been restricted by a lower corrosion rate due to the relative lack of moisture (as compared to the external beams).

Quantity of Cracked, Spalled or Delaminated Concrete

Based on the general visual survey and 30% delamination survey by GHD and detailed visual by RTA:

- ▶ The existing damage noted in GHD's visual survey on the beam soffits and sides is estimated at approximately 125 m². The additional damage/cracking noted by RTA's 'arms-length' visual is estimated at approximately 100 m².
- ▶ The existing damage noted by GHD on the headstocks is estimated at approximately 25 m².
- ▶ Therefore, the total estimated area of cracked/spalled or delaminated concrete at the time of the survey is approximately 250 m².

The damage is likely to increase with time as the carbonation front advances. The beams are also likely to exhibit damage at a faster rate compared to the headstocks.

Repair Options

Repair Options

The bridge requires repair to achieve a future service life of say 25 years.

The following three (3) repair techniques have been considered:

- ▶ Option 1: Conventional Patch Repair, application of a protective surface coating;
- ▶ Option 2: Realkalisation; and
- ▶ Option 3: Cathodic Protection.

Note: A "Do Nothing" option is not considered appropriate for a 25 year service life. It is likely that within a 25 year period, the reinforcement section loss due to corrosion could be significant enough to cause structural problems. A structural assessment of the bridge is required to estimate the allowable steel section loss.



Option 1: Patch Repair and Coat

The remedial measure of conventional patch repair and coat 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 area currently cracked/spalled or delaminated.

Removal of unsound concrete is followed by application of a reinforcement coating system, shotcrete or hand applied application of a cementitious repair mortar, followed by coating with an anti-carbonation coating system.

Conventional patch repair is not a lasting solution to concrete deterioration caused by carbonation induced reinforcement corrosion owing to its inability of reducing the reinforcement corrosion rate at locations where corrosion has initiated but spalling/delamination has not occurred as yet. For the bridge beams, the maximum carbonation depth has exceeded the mean cover at a number of locations.

With this option, spalling/delamination is expected to continue which will require cyclic patch repair every 3-5 years.

Application of an anti-carbonation coating should stop the advancement of the carbonation front. Such a coating however will not provide passivity to the already active reinforcement. As indicated earlier, the maximum carbonation depth has exceeded the mean cover at a number of locations on the bridge beams. The anti-carbonation coating will not stop the reinforcement corrosion (and subsequent spalling) at these "active" locations.

Option 2: Realkalisation

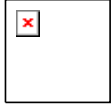
This option involves:

- ▶ Patch repair of existing spalled/delaminated concrete;
- ▶ Apply realkalisation treatment; and
- ▶ Apply decorative or protective coating.

The realkalisation technique is considered to be the most technically appropriate repair option for this bridge.

Realkalisation is an electrochemical technique used for carbonated structures to restore the alkalinity of the concrete and thus reinstate the protective oxide layer around to reinforcement.

The extent of concrete repair prior to realkalisation is substantially less than that required for the conventional patch repair option because there is not the need to breakout concrete to behind reinforcement depth, nor continue concrete break-out until reinforcement shows no signs of visible corrosion. Also, application of reinforcement coating systems to repair areas is not required.



It should be noted that although ASR gel was found to be present in two of the three cores tested, we consider that the ASR has been very slow as ASR induced deterioration was not evident in this 100 or so year old bridge.

Cathodic Protection

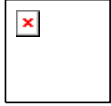
CP is ideally suited for reinforced concrete structures suffering from chloride ion induced corrosion and may be used in carbonated structures also. Therefore, the CP option may be considered as a suitable repair option. We however would not recommend the CP option due to the following reasons:

- ▶ It is realkalisation, not CP, that is normally used for repair of carbonated concrete
- ▶ CP costs could be twice as much as the cost for realkalisation.

Conclusions and Recommendations

Conclusions

- ▶ Significant visible spalling was evident on the soffits of the external beams (B1 and B8) with a majority of the spalling/delamination located adjacent to drainage hole (scupper) locations.
- ▶ Regular vertical cracking at stirrup locations on all beams was recorded.
- ▶ Significant quantity of horizontal cracking on both external and internal beams (B1-B8) at the level of longitudinal reinforcement was evident.
- ▶ Ends of each beams generally cracked horizontally from base doweled fixing to headstocks.
- ▶ Approximately 250 m² of the bridge surface area was spalled/delaminated/cracked.
- ▶ The mechanism responsible for the corrosion is carbonation of the cover concrete. The mean (as measured) carbonation depth exceeded the minimum cover and approached/exceeded the mean cover at numerous locations. General widespread carbonation induced deterioration is likely in the near future. The damage, therefore, is likely to increase with time as the carbonation front advances.
- ▶ Exposed reinforcement exhibits significant section loss on the internal main longitudinal reinforcement in the external beams (i.e. at scupper locations). Reinforcement in other areas exhibits only minor section loss.
- ▶ Chloride contents are low to negligible and chloride induced reinforcement is not likely to be a concern for the remaining service life of the bridge.
- ▶ Although ASR gel was present in two of the three cores tested, we do not consider that ASR is a deterioration mechanism of the bridge.



Recommendations

Three repair options have been reviewed for their applicability/appropriateness to the bridge:

- ▶ Option 1: Conventional Patch Repair, application of a protective surface coating and installation of a Sacrificial Anode System;
- ▶ Option 2: Realkalisation;
- ▶ Option 3: Cathodic Protection.

Notes:

The conventional patch repair and coat option (Option 1) is not a lasting solution to concrete deterioration; this option requires repetitive applications (i.e. 'cyclic patch repair') every ~5 years.

Option 2 (Realkalisation) is considered to be technically appropriate for the bridge and is considered to be technically superior treatment for the bridge.

3. For Option 3 (CP), is not considered to be technically appropriate.

Realkalisation is recommended as the most technically appropriate repair method. It is also considered that Realkalisation will be a more cost effective repair over a future service life of 25 years.

The preferred remedial option should be implemented as soon as practicable. Significant delays in repair may result in increased costs due to higher levels of deterioration.

Prior to selection of the repair option(s), it may be beneficial for the RTA to undertake an engineering cost estimate and a life cycle cost analysis for the above-mentioned repair options (including sensitivity variations). The most appropriate repair option may then be chosen from both technical and economical considerations.