AtkinsRéalis

BROADMEAD ROAD VIADUCT

Concrete Condition Assessment -Test Result Interpretation Report

Kenson Highways Ltd.

BRVTP-ATK-SBR-B4-RP-CB-000005
Revision: C02

Structure Key: O51 / B4

October 2024

Notice

This document and its contents have been prepared and are intended solely as information for Kenson Highways Ltd and use in relation to this Assignment.

AtkinsRéalis assumes no responsibility to any other party in respect of or arising out of or in connection with this document and/or its contents.

Document History

Revision	Purpose description	Originated	Checked	Reviewed	Authorised	Date
C02	Appendix A removed to reduce file size.	SMS	HSP	KD	DS	30th October 2024
C01	For Client review / comment	SMS	HSP	KD	DS	30th October 2024

Issue Record

Client	Kenson Highways Ltd
Project	Broadmead Road Viaduct Investigation & Assessment
Document Title	Concrete Condition Assessment – Test Result Interpretation Report
Job number	5224707
Version No.	C02
Document Reference	BRVTP-ATK-SBR-B4-RP-CB-000005

Contents

1. Introduction

1.1. Broadmead Road Viaduct information

Asset No.	B4
Name:	Broadmead Road Viaduct
OS Grid Ref:	TQ 408 913
Postcode:	IG8 0AR
What 3 Words	Unity. Matter. Army
Date of Construction:	1937
Structure Type:	31 span viaduct with 8 continuous spanning RC slabs on transverse RC crossheads and RC columns founded on RC spread footings.
Under:	Woodford to South Woodford London Underground Railway Line (Beneath Span 6).
Over:	Broadmead Road (A1009)
Bridge Width:	19m
Bridge Length:	227m

The viaduct was tested at multiple locations across the span on the support structures below the deck comprising crossheads and columns, the deck soffit and the deck topside via trial pits through the surfacing.

The layout of the test areas can be seen in the extract drawing below.

1.2. Abbreviations

PI- Principal Inspection

GI- General Inspection

CP- Cathodic Protection

ICCP- Impressed Current Cathodic Protection

AAR- Alkali aggregate reaction

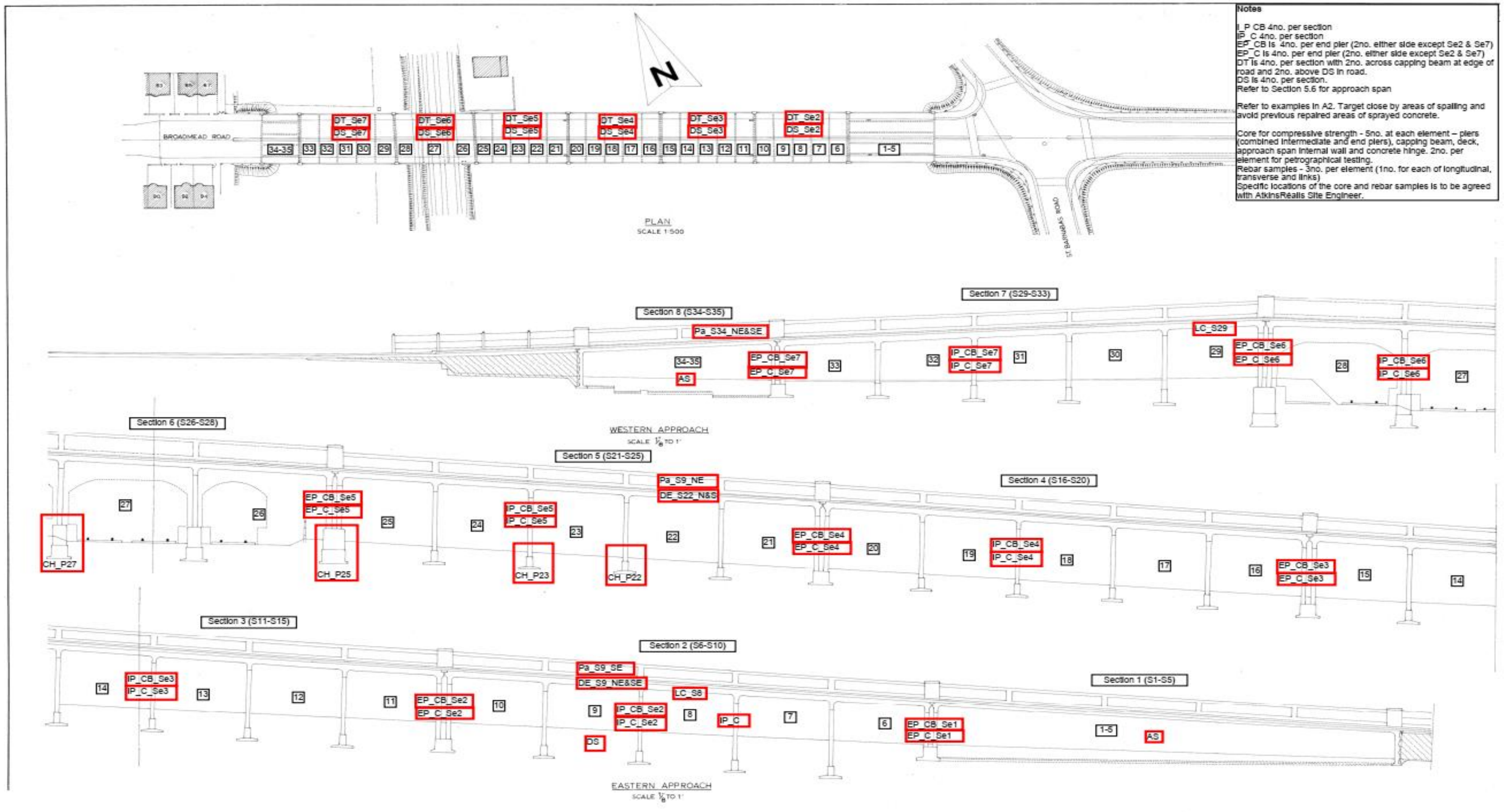
ASR- Alkali silica reaction (most common type of AAR)

DEF- Delayed Ettringite Formation.

CEMI- Grade of cement (Portland cement)

Ag/AgCl/0.5M KCL- Common type of Silver Chloride reference Electrode.

Cu/CuSO4 – Copper Sulphate reference Electrode



Notes

I_P CB 4no. per section
IP_C 4no. per section
EP_CB is 4no. per end pier (2no. either side except Se2 & Se7)
EP_C is 4no. per end pier (2no. either side except Se2 & Se7)
DT is 4no. per section with 2no. across capping beam at edge of road and 2no. above DS in road.
DS is 4no. per section.
Refer to Section 5.6 for approach span

Refer to examples in A2. Target close by areas of spalling and avoid previous repaired areas of sprayed concrete.

Core for compressive strength - 5no. at each element – piers (combined intermediate and end piers), capping beam, deck, approach span internal wall and concrete hinge. 2no. per element for petrographical testing.
Rebar samples - 3no. per element (1no. for each of longitudinal, transverse and links)
Specific locations of the core and rebar samples is to be agreed with AtkinsRéalis Site Engineer.

Abbreviations

A – Abutment
AS – Approach Span
CB – Capping Beam
CH – Concrete Hinge
DE – Deck Edge
DT – Deck Top
DS – Deck Soffit
EP – End Pier
IP – Intermediate Pier
LC – Lighting Column
NE – North Elevation
P– Pier
Pa – Parapet
S – Span
Se – Section
SE – South Elevation

Broadmead Road Viaduct - Testing Locations
Sketch Ref. BRVTP-ATK-SBR-B4-SK-CB-000002 P01
L Moran 29/02/24

These testing works were commissioned following a 2023 Principal Inspection of the structure by Enfield Council Engineers (working on behalf of Redbridge Council) which identified structurally significant defects in the structure, particularly at the end legs of each section of the structure and the deck soffit throughout. The nature and severity of these defects necessitated closure of the structure to vehicles in 2023. The report starts by summarising the existing condition of the structure, the tests undertaken thus far, and results obtained for the same. The report includes a review of the likelihood of corrosion to the structural steel and its influencing factors.

In the conclusion section of this document the findings of the testing are analysed and potential solutions for maintaining or extending the residual life of the structure are proposed.

1.3. Aim of investigation

This investigation aims to obtain information about condition and mechanical properties of the concrete and reinforcement to assess the suitability of the structure for an extended service life and what remedial / protective measures may be required. In the conclusion section of this document the findings of the testing are analysed and potential solutions for maintaining or extending the residual life of the structure are proposed.

2. Schedule of investigations

The Contractor undertook the investigations set out in Table 1.

The factual results and locations of sampling are presented in the factual report by CPL Ltd document reference; 693/REP/001.

Table 1 – Schedule of investigations

Structural element	Number of test areas	Total
Intermediate pier	2no. capping beams and 2no. columns per section	24
End pier	2no. capping beams and 2no. columns both either side of joint per section (except at Section 2 and Section 7)	48
Deck soffit	4no. per section	24
Deck top	4no. per section	24
Deck edge	2no. per elevation (non-rail)	4
Parapet	2no. per elevation (non-rail)	4
Approach span deck top	1no. per approach span	2
Approach span soffit	2no. per approach span	4
Approach span cross wall	2no. per approach span	4
Abutment	4no. per abutment	8
		146

3. Site condition / Visual inspection

A principal inspection (PI) of the structure was carried out by Enfield Council in 2023 prior to the testing and investigation in 2024. A visual assessment of the viaduct made whist carrying out the concrete testing revealed:

- The end piers were in generally poor condition, with extensive areas of concrete delamination and spalling at areas of leakage. Extensive temporary repairs had been undertaken prior to the concrete testing.
- The intermediate piers were typically in good condition with only a small number of visible defects.
- The deck soffit condition was variable, with large patches of delamination and or poor compaction previously removed, and patch repaired. The deck soffit away from these areas looked to be in fair condition. The testing subsequently proved that the deck soffit concrete was heavily carbonated (See Figure 1).
- The deck topside was only visible at the trial pit locations opened for the concrete testing. Within these areas the condition was again variable with some areas exhibiting low cover and spalling once the waterproofing was removed. It was clear that previous concrete repair phases had been undertaken on the deck topside. The report states that generally the bar condition within the breakouts was sound, however one of the extended breakouts for tensile strength testing exhibited section loss to 3No. low cover bars that had not been part of any previous repairs (See Figure 2). An aborted core sample also revealed that a concrete repair had not fully removed the defective concrete with poorly compacted parent material below the repair (See Figure 3).
- The hinge details on the end piers were not investigated as planned due to challenges in securing possessions on the Central Line within the timeframe of the testing. The minor exploratory work that was carried out appeared to indicate that the mastic seals around the columns on the tops of the upstands were not effective and the void had become full of water overtime.
- The east and west hollow abutments appear in fair to good condition. Only the west abutment was investigated as access could not be arranged into the east abutment. The west abutment appeared to be in worse condition than the east as it has been disused for many years. However, only a limited number of significant defects were noted during the investigation many of which had been previously patch repaired and were largely confined to the deck soffit, which should be treated separately from the rest of the abutment.
- The investigation of the lamp column fixings was also not completed due to time constraints, with only the corroded top plates partially exposed. The bolt condition and the presence of any section loss was not confirmed (See Figure 4). However, given the condition of the top plates coupled with deterioration around many of the bolt fixings on the deck soffit the lamp column holding down bolts should be assumed to be in poor condition.
- The parapet and edge beams were in variable condition, with frequent previous repairs particularly to the edge beams. The test report states no defects for these elements which is not considered entirely accurate- See Figure 5 for an example defect.

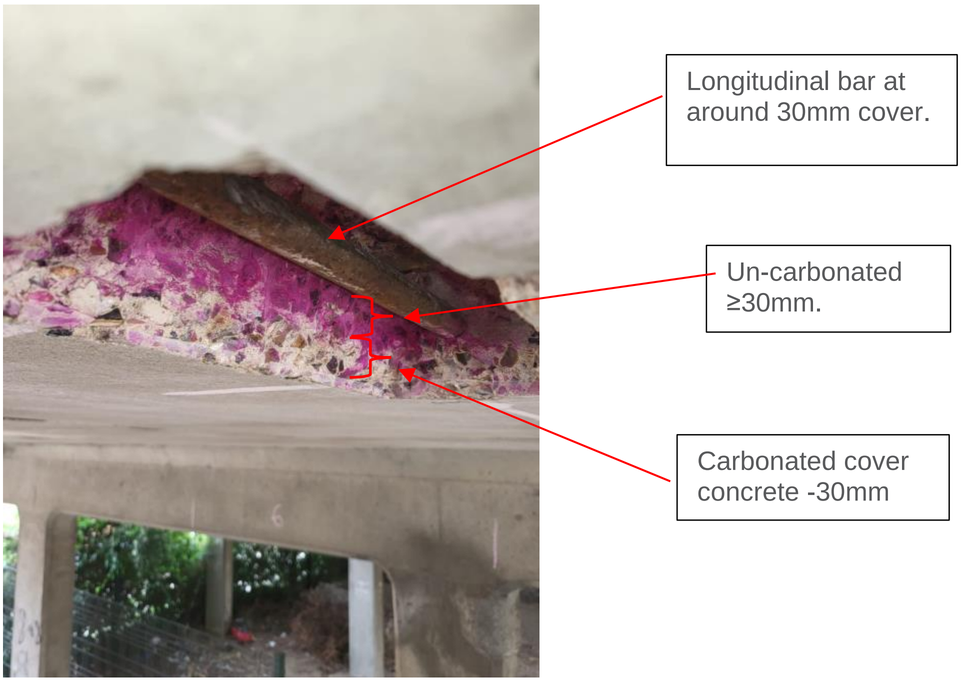


Figure 1- Carbonated concrete on deck soffit- Location TS1.

Corroded low cover bars. Top of bars are flat-possibly from historic planning operation.

Figure 2- Corroded bars with section loss, possibly due to historic planning, on deck topside -location TS6.

Figure 3- Core from deck topside showing poorly compacted parent concrete below repair.

Figure 4- Corroded base for lamp column in Span 4- bolt not extracted

Figure 5 -Defect on parapet at northwest end

4. REVIEW AND INTERPRETATION OF CONCRETE TEST DATA

The results of concrete testing on the support structures and bridge deck together with the defects identified by visual inspection have been analysed. The summary of results is presented in table 4.2 with principal findings presented below.

4.1. Concrete cores and petrographic examination

The findings of compressive strength tests and petrographic analysis of core samples are summarised below:

a) Compressive strength of concrete was assessed on a number of 100mm core samples, Cylinder L/D ratio 1:1. Results are presented in Table 4.2. The results of compressive strengths on cores samples indicate that concrete used in the deck, end piers and intermediate piers was of good quality and the mechanical properties are still maintained after 87 years of use.

b) Macroscopic features of core samples indicated crushed flint aggregates were used in concrete. Visual inspection revealed that core samples were of generally coherent concrete, well compacted with occasional air voids.

c) Core samples taken from the elements were examined petrographically. The water to cement ratio of concrete has been estimated petrographically. The paste in concrete of all elements is based on Portland cement only. On the basis of current level of porosity of the paste in concrete of the elements the water/cement ratio is estimated to be in the range of 0.5−0.6. The core samples showed cement content in the order of 340-440 kg/m^3. The estimated W/C ratio and cement content of concrete was found to be consistent with measured compressive strength. However, a number of samples retuned high cement content values ranging from 20% to 29%. The highest values were found on the deck topside which were contrasted by the low, more typical values on the deck soffit 10% to 15%. The support structures generally return typical values in the region of 14 -15% with occasional anomalies up to 22%. The anomalies appear to be related to sampling and the amount of aggregate in the samples.

d) The concrete core samples could be categorised into a single group based upon the main coarse aggregate type: CEMI. (100% cement but can contain 5% other materials).

e) No evidence of ASR was observed in any of the samples.

f) Alkali Silica Reaction (ASR) is the most common alkali-aggregate reaction occurring in concrete. AAR occurs to due reactive aggregate particles reacting with the alkalinity in the concrete. The formation of a gel which increases in volume by taking up water resulting in extensive map cracking and break-up of the concrete. Even when reactive particles are present in the concrete the reaction requires water to initiate, so typically only occurs on elements which are exposed to frequent wetting.

g) No evidence of DEF was observed in any of the samples. Delayed ettringite formation (DEF) results in the cracking and expansion of concrete due to the delayed formation of ettringite crystals. This typically occurs due to high early temperatures (up to 80ºC) preventing the formation of normal ettringite. DEF is often prevented by controlling the early concrete temperatures and through specific mix design.

4.2. Chloride content and Depth of Carbonation

The findings of chloride content and depth of carbonation tests on samples are presented in table 4.2 and summarised below:

a) Concrete dust samples collected from two locations at each test panel sample were tested for the chloride ion content. These samples were taken at the depth of 5 to 100 mm, except for an additional set of horizontal samples taken at the joint location these were taken at approximately 150-250mm depth to enable sampling of the inside face of the crossbeam on the end piers. The level of chloride ion content per mass of cement in the concrete cover layers for the various elements was as summarized below.

- Deck topside: 0.02- 0.96%
- Deck Soffit: 0.02 -0.68%
- Intermediate Pier Crosshead: 0.02 -0.68%
- Intermediate Pier Columns: 0.02 – 0.48%
- End Pier Crosshead: 0.02 -2.48%
- End Pier Crosshead (inside face): 0.02 to 1.42%
- End Pier Columns: 0.02 – 3.6%
- Parapet: 0.02 -0.06%
- Edge Beams: 0.02 – 0.16%
- West Abutment: 0.02 -0.56%

The range of chloride content over the structure varies widely from 0.02 to 3.6% (by mass of cement at the depth of reinforcement). Which ranges from below the threshold of 0.4% indicating low risk, to significantly above the critical threshold value of 1.0% by mass of cement indicating a very high risk with intervention required in accordance with BRE Digest 444. In general, the data confirms negligible to light chloride contamination of concrete on the intermediate piers, parapets, edge beams and the west abutment and heavy chloride contamination at the end piers and to a limited degree on the deck topside, although this appears localised to areas of waterproofing damage. The chloride profiles indicate the source of chloride is leakage through the joints carrying chloride laden water from winter gritting activities. The elements subject to chloride ingress are at significant risk of reinforcement corrosion. This was evidenced by the significant amounts of concrete delamination and spalling seen at numerous locations on the end piers.

b) The maximum depth of carbonation determined at the concrete dust sampling locations and / or breakouts was generally found to be at the level of the reinforcement with carbonation frequently down to 30mm in the majority of elements. The exception to this was the deck topside, as would be expected due to its protection via the waterproofing and road surfacing, here the carbonation was consistently below 3 mm. As a result, there is a risk of reinforcement corrosion due to carbonation across much of the structure with the exception of the deck topside.

c) With respect to the corrosion mechanisms and risk to durability of structure posed by chloride attack and carbonation the following points can be made:

- The ingress of chlorides into reinforced concrete can lead to the corrosion of steel. Due to the high alkalinity of cement, reinforcing steel forms a protective iron oxide layer (passive film) providing the steel with protection against mechanical damage and corrosion (Bertolini, L et al., 2013). In this state, with very low electric potentials present, the iron in steel is considered to be thermodynamically stable and passive i.e. not reactive and will not dissolve into solution nor react with water to form passive oxides. It should be noted that the formation of the passive layer requires the sufficient compaction of local concrete.

- The presence of chloride ions at the steel surface can disrupt this balance leading to the breaking down of the passive film and thus the creation of a corrosive environment at the steel surface (Bertolini, L et al., 2013). As the structural integrity of the structure is provided by the steel reinforcement, steel corrosion poses the most significant threat to the durability of the structure. As a result of chloride induced steel corrosion the durability of the concrete is also put at risk. The increased volume of corroded steel (rust products) will lead to further cracking of concrete cover. Chloride corrosion tends to be localised in nature and tends to cause pitting of the steel, these pits can penetrate deep into the reinforcing bars.

- Carbonation is the reaction of carbon dioxide in the environment and calcium hydroxide in the cement paste. The result is the production of calcium carbonate and a lowering of the pH to around 9. At this pH the protective oxide layer breaks down and corrosion can initiate. It is likely that condensation forming on the deck soffit provided sufficient moisture to allow the reaction to occur and given the age of the structure there has been sufficient time for the carbonation front to migrate to or close to the reinforcement. Carbonation tends to affect large areas of reinforcement with resultant flattening of the reinforcement rather than deep pits.

4.3. Concrete Cover

The range of cover for each element is presented in table 4.2. The cover meter survey was carried out on all test locations and actual depth of cover was also physically measured at breakout locations. The range of minimum cover to the reinforcement for each element is summarised below.

- Deck topside: 8-60mm
- Deck Soffit: 10-39mm
- Intermediate Pier Crosshead: 2-32mm
- Intermediate Pier Columns: 12-35mm
- End Pier Crosshead: 10-69mm
- End Pier Columns: 3-30mm
- Parapet: 18-38mm
- Edge Beams: 18-38mm
- West Abutment: 10-38mm

The cover stated above is from the cover survey (scanning) which was frequently below the minimum cover specified in BS 8500-1 for XC3/4 and XD3 exposure classes i.e. 40-55mm applicable to this structure when concrete is subjected to carbonation and chloride contamination. Actual cover should have been recorded at the concrete breakout, but no data is present in the report.

4.4. Potential Survey Data

The range of steel/concrete half cell potentails for each element are presented in table 4.2. The steel/concrete reference electrode survey was undertaken in accdance with the Concrete Society Technical Report 60 ; Electrochemical tests for reinforcment Corrosion, 2004.

The survey was undertaken using a Ag/AgCl/0.5M KCL type portable reference electrode.

The criteria noted in ASTM C876-22 ‘Standard test method for half-cell ptentials of uncoated reinforcing steel in concrete’ are presented below.

Potentials more postive than -200mV represent areas with a low risk of corrosion (≤10%).

Potentials between -200mV and -350mV have an indeterminate risk of corrosion

Potentials more negative than -350mV have a high risk of corrosion (≥90%)

The results of the survey are presented in accordance with the standard to Cu/CuSO4 type reference electrode using a conversion factor of -50mV to account for the use of the Ag/AgCl electrode.

Testing was undertaken on a nominally 250mm grid acorss the test panel, with some variation due to site conditions and structure geometery.

The range of potentials are summarised below-

- Deck topside: +87 to -605mV
- Deck Soffit: +311 to -409mV
- Intermediate Pier Crosshead: +280 to -258mV
- Intermediate Pier Columns: +275 to -305mV
- End Pier Crosshead: +277 to -571mV
- End Pier Columns: +164 to -508mV
- Parapet& Edge beams: +258 to -80mV
- West Abutment: +118 to -487mV

The potential values varied with each element highlighting the variable nature of the contamination. The highest potentials recorded indicate a significant corrosion risk which can be seen particularly on the End Piers where leakage through the joints was significant and in some areas on the deck topside possibly indicating areas of historic damage to the waterproof system.

A large number of potential readings were found to be highly positive this is thought to be due to two possible factors; discontinuity within the structure and more signifcantly the generally high level of carbonation in the concrete which is known to cause a siginficant postive shift in potentals as per Concrete Society Technical Report TR60.

4.5. Cement Content Analysis

The cement content taken at various locations was reported as being variable. A number of high to very high cement contents were reported particularly on the deck topside, this unusually was contrasted by much lower cement content of the deck soffit. As this is the same element it would be assumed that the cement content on both the deck topside and deck soffit would be similar. Given how high some of the cement content percentages are (up to 29%) the samples may be erroneous. Outside of the anomalous readings the cement contents typically fall within the normal range.

Comparison of cement content and compressive strength, in the same general location, does not appear to suggest any significant variation in strength as a result. Analysis of the laboratory data does indicate that the high cement contents correspond to a reduction in the amount of aggregate found in the respective sample, see example in table 4.1 below. This poor distribution of aggregate may be the result of inadequate mixing at the time of construction, this backed up by evidence of poor compaction found during the coring on the deck topside.

Table 4.1- Aggregate/cement ratio

Calculated Values	**Sample DT21**	**Sample DT22**	**Sample DT23**	**Sample DT24**
Cement Content (%)	26	24	12	12
Aggregate Content (%)	68.1	70.7	85.2	85.3
Aggregate/cement ratio	2.6	3.0	7.1	7.2

Table 4.2. Summary of concrete test data from CPL report.

Location/ Element	Minimum depth of cover (Range)	Chloride at rebar depth	Corrosion risk vs chloride content	Range of Half-cell potential (wrt $Cu/CuSO_4$)	Corrosion risk vs half-cell potential	Range of carbonation depth	Corrosion risk vs carbonation	Comp Strength (fck)/ Cement content	Summary of petrographic analysis	Number and % of test panels with overall high corrosion risk	Number and % of defects
	mm	% per mass of cement		mV		mm		N/mm^2 %			
Deck topside (24 No. TP)	8-60mm	0.02 – 0.96. (6No. locations above 0.4).	79.2% low risk 20.8% med risk 0% high risk	+87 to -605mV.	37.5% low risk 25% med risk 27.5% high risk	≤3mm	Low Carbonation not at, or close to the depth of reinforcement	31.4 to 43.7N/m m² 11.0 to 29.0 %	Core D14 Deck topside Span 4- No cracking, or ASR, well compacted with minor voids. Core D24 Deck topside Span 6– Longitudinal crack, no ASR. Moderate compaction with frequent voids.	0%	46%- 11 TP with defects (mainly repaired)
Deck Soffits (19No. TP)	10-39mm	0.02 -0.68 (2no. locations above 0.4).	89.5% low risk 10.5% med risk 0% high risk	+311 to -409mV.	84.2% low risk 10.5% med risk 5.3% high risk	10-35mm	High Carbonation front at or within 10mm of reinforcement	NA 11.0 to 14.0%	NA	68%	70% - 15 TP with defects

Location/ Element	Minimum depth of cover (Range)	Chloride at rebar depth	Corrosion risk vs chloride content	Range of Half-cell potential (wrt $Cu/CuSO_4$)	Corrosion risk vs half-cell potential	Range of carbonation depth	Corrosion risk vs carbonation	Comp Strength (fck)/ Cement content	Summary of petrographic analysis	Number and % of test panels with overall high corrosion risk	Number and % of defects
	mm	% per mass of cement		mV		mm		N/mm² %			
Intermediate Pier Crossheads (10No. TP)	2-32mm	0.02 -0.68 (1no location above 0.4)	90% low risk 10%med risk 0% high risk	+280 to -258mV.	90% low risk 10% med risk 0% high risk	0-30mm	High Carbonation front at or within 10mm of reinforcement	30.8 – 38.8 N/mm² 13.0 to 18.0%	Core 1.4.2 Cro. No cracking, or ASR. Good compaction with sporadic voids. Core 4.2.2 Cro. No cracking, or ASR. Good compaction with sporadic voids.	20%	70% - 7 TP with defects
Intermediate Pier Columns (10No. TP)	12-35mm	0.02-0.48 (2No. locations 0.4 or above)	90% low risk 10%med risk 0% high risk	+275 to - 305mV.	90% low risk 10% med risk 0% high risk	5-22mm	Med-High Carbonation front at or within 10mm of reinforcement	27.5-30.3 N/mm² 13.0 to 21.0%	Core 1.2.3 Col. No cracking, or ASR. Good to moderate compaction with frequent small voids.	30%	40% - 4 TP with defects

Location/ Element	Minimum depth of cover (Range)	Chloride at rebar depth	Corrosion risk vs chloride content	Range of Half-cell potential (wrt $Cu/CuSO_4$)	Corrosion risk vs half-cell potential	Range of carbonation depth	Corrosion risk vs carbonation	Comp Strength (fck)/ Cement content	Summary of petrographic analysis	Number and % of test panels with overall high corrosion risk	Number and % of defects
	mm	% per mass of cement		mV		mm		N/mm^2 %			
End Pier Crossheads (16No. TP)	10-69mm	0.02- 2.48. (5No. locations 0.4 or above). (4No. locations above 1.0).	62.4% low risk 18.8%med risk 18.8% high risk	+277 to - 571mV.	18.8% low risk 25% med risk 56.2% high risk	5-20mm	High Carbonation front at or within 10mm of reinforcement	36.2 – 41.1 N/mm^2 16.0 to 19.0%	No core sample taken?	69%	94% - 15 TP with defects
End Pier Columns (13No.)	3-30mm	0.04-3.60. 12No. locations above 0.4. 12No. locations above 1.0.	18.8% low risk 25%med risk 56.2% high risk	+164 to - 508mV	6.2% low risk 25% med risk 68.8% high risk	5-30mm	High Carbonation front at or within 10mm of reinforcement	34.4 – 43.1 N/mm^2 17.0 to 22.0%	Core E3.3.W col. No cracking, or ASR. Good compaction with sporadic voids.	81%	94% - 15 TP with defects
End Pier Crossheads- Deep chloride- %No joints tested	NA	0.02 to 1.42	40%-Low 20% -Med 40% High	NA	NA	NA	NA	NA	NA	NA	Dust samples only.
Parapets Edge Beams (8No.TP)	18-38mm 18-36mm	0.02-0.06 0.02- 0.16	100% low risk	+258 to - 80mV	100% low risk	10-30mm 10-30mm	High- Carbonation front at or within 10mm	NA	NA	8%	No defects reported; however, some

							of reinforcement.				defects do exist.
Location/ Element	**Minimum depth of cover (Range)**	**Chloride at rebar depth**	**Corrosion risk vs chloride content**	**Range of Half-cell potential (wrt $Cu/CuSO_4$)**	**Corrosion risk vs half-cell potential**	**Range of carbonation depth**	**Corrosion risk vs carbonation**	**Comp Strength (fck)/ Cement content**	**Summary of petrographic analysis**	**Number and % of test panels with overall high corrosion risk**	**Number and % of defects**
	mm	**% per mass of cement**		**mV**		**mm**		**N/mm²** **%**			
West Abutment (5No. TP)	10-38mm	0.02- 0.56 1No. location above 0.4	80% low risk 20%med risk 0% high risk	+118 to -487mV	20% low risk 20% med risk 60% high risk	5-30mm	Med-High- Carbonation front at or within 10mm of reinforcement. High carbonation confined to deck soffit.	NA	Core -West Abut. No cracking, or ASR. Good compaction with sporadic small voids.	40%	40% - 2 TP with defects

5. Rehabilitation Options

The possible repair options are considered in principle in this section and will be explored/developed further in a future options report. A general overview is presented below:

i) Option 1 – Do nothing.

ii) Option 2 – Patch repair with galvanic anodes.

iii) Option 3 – Concrete replacement with galvanic anodes.

iv) Option 4 – Concrete repair & Impressed Current Cathodic Protection (ICCP).

v) Option 5 – Partial replacement (End Piers) and ICCP

vi) Option 6 – Demolish Structure.

5.1. Option 1 – Do nothing

The concrete testing has shown significant chloride contamination has already taken place on the end piers; carbonation was also found to be frequently at the level of the reinforcement on the majority of structures. As such the structure generally has a high-risk category of corrosion. The large area of delamination on the end piers and surface corrosion found on bars in what was assumed to be sound concrete shows the reinforcement is actively corroding. The continued loss of reinforcement cross section would compromise the structural integrity further and therefore doing nothing is not viable.

5.2. Option 2 – Patch repair with galvanic anodes

Localised patch repairs are used when the delamination is limited to small areas of the structure. The end piers have extensive delamination and therefore localised repair is not viable. To prevent incipient anode effect at the repair, which is where the concrete adjacent to the repair becomes an anode and its reinforcement corrodes, galvanic anodes at the repair boundary are required. The anodes could be installed by either tying to the reinforcement within the repaired concrete (Type 1A) or installed in holes drilled into the existing concrete substrate and connected to the reinforcement within the repaired area (Type 1B). Galvanic anodes have a limited-service life (up to 10-15 years) depending on level of chloride contamination, concrete moisture content and environmental condition and would need to be replaced once spent. Furthermore, galvanic anodes will have limited zone of influence typically 150-300mm away from the anodes and therefore reinforcement within areas of concrete away from the repair patches with chloride contamination will not be protected. Given the extensive nature of carbonation on the structure, coupled with high chlorides on the end piers this is not considered a viable option for this structure.

Hybrid anodes, which combine an initial powered phase (via batteries) with long term galvanic protection, are also not considered suitable, given the potential extent of the CP system from a practical perspective during installation, the extent of chloride contamination and having unproven performance at present.

Additionally, the application of anti-carbonation coating at this time would not be effective as the majority of areas already show the carbonation front is at the level of the reinforcement.

This option as with all the other repair options will require deck re-waterproofing, joint replacement and deck drainage improvements which will increase the scope and may lead to delivery over two years.

5.3. Option 3 – Partial concrete replacement

The structural elements comprising the structure could be repaired by replacing the contaminated concrete. This option removes the requirement for any cathodic protection system. However, concrete replacement requires that all contaminated concrete with chloride greater than 0.4% Cl per mass of cement and/or actively corroding as indicated by half-cell potentials more negative than -350 mV, be replaced. The level of chloride contamination and half-cell measurements would require full replacement of substantial amounts of the crossheads and columns on the end piers to achieve this. This would require a large amount of concrete removal and either phased repairs and temporary jacking of the deck. Extensive carbonation across the structure would also require removal, this would be particularly extensive on the deck soffit.

Given the extent of concrete defects and contamination (chloride & carbonation) across the structure, the requirement for concrete removal renders this option unsuitable.

This option as with all the other repair options will require deck re-waterproofing, joint replacement and deck drainage improvements which will increase the scope and may lead to delivery over two years. Further to this the works, particularly on the deck soffit, would need to be undertaken in multiple longitudinal strips such that the integrity of the structure is not compromised during the works, this would result in an extended duration for the works. Any traffic or pedestrians will need to be diverted to other parts of the deck not affected by the work to prevent loading during the remediation.

5.4. Option 4 – Concrete Repair & Impressed current cathodic protection (ICCP)

Following extensive concrete repairs to the defective concrete the remaining contaminated, but still sound, concrete could be protected with an impressed current cathodic protection system. To stop the corrosion of the existing reinforcement in chloride contaminated concrete and carbonated concrete ICCP could be installed. ICCP uses a permanent external anode system connected to a low voltage DC electrical supply to provide sufficient current to ensure the reinforcement is cathodically protected. An advantage of ICCP is that structurally sound concrete that is contaminated with chloride does not need to be broken out. The system is well established in the industry. It does require monitoring to ensure durability, which is typically carried out remotely. The anode type can be either drilled discrete anodes or a ribbon mesh, with the discrete anodes protecting deeper reinforcement. The capital cost and cost of ongoing monitoring of the ICCP coupled with the required concrete repairs to remove defective concrete would need to be understood but this is a possible option for the structure.

This option as with all the other repair options will require deck re-waterproofing, joint replacement and deck drainage improvements which will increase the scope and may lead to delivery over two years.

Additionally, it is anticipated that two separate power supplies would be required, one for each approach, to minimise the amount of cabling installed/crossing over the railway.

Consumption rates for the anode system will provide an expected anode life of 60yrs, the control panels and power supplies would have an expected life of 15 yrs. The system is fully monitored quarterly (remotely) with an on-site inspection carried out annually. Typical annual running costs for an ICCP system, including monitoring, maintenance and power are in the region of £16K.

5.5. Option 5 – Replacement of End Piers and Impressed current cathodic protection (ICCP)

Given the condition and levels of contamination, both from chloride and carbonation, the End Piers could benefit from complete replacement. This would enable design of the end piers in conjunction with the repair/ replacement of the deck ends and the essential new expansion joints. This portion of the structure would not require a CP system and provide the maximum possible service life. The remaining elements would require concrete repairs to all defective concrete followed by installation of an ICCP system on the remaining sound concrete. The capital cost and cost of ongoing monitoring of the ICCP coupled with the required concrete repairs to remove defective concrete would need to be understood but this is a possible option for the structure.

This option as with all the other repair options will require deck re-waterproofing, joint replacement and deck drainage improvements which will increase the scope and may lead to delivery over two years.

The replacement of the end piers would require extensive temporary propping to facilitate the demolition, increasing the cost and risk associated with this option. This may be particularly challenging over the LUL rail span (Span 6) due to the approvals required from London Underground/TfL. However, temporary propping may also be required for the other repair options.

5.6. Option 6 – Demolish and Re-build

Demolition of the structure may be required; the outcome of the on-going structural assessment will ultimately determine if this option is to be considered but it should also be considered following a cost analysis of the concrete repair and CP option compared to re-building the entire structure. The potential for a reduced duration of closures to the Central Line with this option may be significant.

Demolition of the structure may be full or partial depending on the findings of the structural assessment and the various cost analysis of the options provided.

5.7. Advantages and Disadvantages

Option	Advantages	Disadvantages
Option 1 – Do nothing	No cost.	Bridge would have to remain closed and potentially be demolished in the short to medium term as on-going deterioration continues to impact the structure. Does not facilitate the re-opening of Broadmead Road (A1009) Loss of steel cross-section due to high rate of corrosion. Accelerated by delamination and spalling exposing reinforcement, eventually leading to failure of the end piers.
Option 2 – Patch repair with galvanic anodes	Delamination, cracks and spalling repaired which will reduce the rate of corrosion. Low to moderate cost.	Ongoing deterioration of sound but chloride contaminated or carbonated concrete and need for future repairs. Anode life span limited to 10-15 years, requiring more works in the medium term.
Option 3 – Partial concrete replacement with galvanic anodes	New concrete free of defects and excessive chloride contamination and or carbonation Low to moderate cost. Long service life. Structural integrity restored if replacing reinforcement.	Replacement of large area of concrete. Ongoing corrosion and deterioration of area areas outside the zone of influence of the anodes. Anode life span limited to 10-15 years, requiring more works in the medium term. Long construction period to remediate carbonated deck soffit.

Option	Advantages	Disadvantages
Option 4 – Concrete repair and Impressed current cathodic protection (ICCP).	Minimal concrete repair in future. Long service life. Proven technology. Remote monitoring/management of CP system to access performance.	High cost. Requires continuous power and monitoring. Replacement of powers supplies/control equipment within design life of system. 60 years' service life for anode system. Annual running costs and on-going monitoring /maintenance of CP system.
Option 5 – Replacement of End Piers and Impressed current cathodic protection (ICCP).	Complete replacement of the End Piers will provide maximum service life without need for CP. Minimal concrete repair in future. Long service life. Proven technology. Remote monitoring/management of CP system to access performance.	High cost. Extensive propping required to replace end piers. Requires continuous power and monitoring. Replacement of powers supplies within design life of system. 60 years' service life for anode system. Annual running costs and on-going monitoring /maintenance of CP system.
Option 6 – Demolish and re-build	New structure Long Service life, with minimal future maintenance. Dependant on design may be a quicker option than repair and CP especially over the rail span. Costs associated with closure of the Central Line will be minimised.	High Cost Requires detailed design. Possible long duration for design and installation.

6. Conclusions & Recommendations

Broadmead Road Viaduct is currently in highly varied condition, the summary condition and recommended remedial options for the individual elements are presented below. The possible repair options are considered in principle in this section and will be explored/developed further in a future options report:

- **Deck topside**- fair condition from trial pits/testing locations but some defects are highly likely elsewhere on the deck topside. Repairs, appear to have been carried out to a fair standard; however, a core sampling into one of the repair areas indicated poorly compacted parent concrete below the repair suggesting that previous repairs have not fully addressed the defects within the slab. A proper understanding of condition can only be assessed once surfacing and waterproofing have been removed and the concrete hammer tapped for delamination.

- **Deck Soffit**- Visual inspection confirmed the soffit was in varied condition with a wide range from poor to good. Numerous defective areas were evident with multiple previous repairs the quality of which are also highly variable. Despite numerous other areas of the deck soffit appearing in fair to good condition the testing indicated an ongoing risk to the elements from carbonation. Testing indicated extensive deep carbonation on the deck soffit which presents a high future corrosion risk, and measures should be put in place to mitigate this in the form an ICCP system. Application of anti-carbonation coating at this time would not be effective as the majority of areas already show the carbonation front is at the level of the reinforcement.

- **End Piers (at deck joints)**- Inspection and testing confirmed the end piers, crossheads and columns, were in poor condition with extensive concrete delamination, cracking and spalling caused by leakage through the deck joints. If retained, these piers will require extensive concrete repairs to replace the defective concrete and an ICCP system to protect the contaminated but still sound remaining concrete. Alternatively, given the poor condition, these piers could be completely replaced removing the need for a protection system on these piers and enabling full repair of the deck ends as part of the essential deck joint replacement.

- **Intermediate Piers –** The intermediate piers were generally in fair to good condition with both the crossheads and the columns exhibiting limited defects. Any deterioration on these structures appeared of limited size and extent. However, the concrete testing indicated that carbonation was a generally high risk to the crosshead and a medium to high risk on the columns, as such intervention is required on these structures to prevent corrosion in the medium to long term. The best option for this would again be an ICCP system.

- **Parapets and Edge beams –** The parapets and edge beams were in generally fair condition with random defects observed across the structure. The testing indicated a high corrosion risk from carbonation on these elements.

    - Edge beams- frequent historic repairs are evident across the spans; however, the testing report states no defects were found but this is most likely due to limitations in where the element could be safely accessed rather than an accurate assessment of condition. This element has a high corrosion risk from carbonation and as such should be treated as part of the deck soffit.

- Parapet -The parapet exhibited defects in places, with the testing report again stating no defects were evident which is not considered accurate; however, it should be noted that the test panels were confined to the base of the parapet directly above the edge beam. Taking into account the nature of the element it may be beneficial to focus on any necessary concrete repairs as required with a monitoring regime to address any on-going issues. If the corrosion risk is seen as being too high, then the parapet would require replacement.

- **West Abutment –** The west abutment was in fair condition, with the exception of the roof which is part of the deck soffit and the front wall which is an end pier with infill between the columns, these elements should be treated separately. See recommendations for End Piers and deck soffit respectively. The longitudinal, internal walls and the rear wall appeared in good condition with the concrete testing indicating generally low levels of carbonation. The floor, rear wall and internal walls of the abutment are in sufficiently sound condition to warrant only minor remedial works.

- **East Abutment –** Not covered by the concrete testing as access was not possible for the duration of the testing works.

7. References

- Bertolini, L et al., Corrosion of Steel in Concrete. Prevention, Diagnosis, Repair, WILEY-VCH, 2013.

- Tuuti, K. Corrosion of steel in concrete. Swedish Cement and Concrete Research Institute, Stockholm, 1982.

- BRE, Corrosion of steel in concrete. Investigation and assessment, Digest 444. Part 2. BRE, 2000.

- British Standard, Concrete. Method of specifying and guidance for the specifier, BS 8500-1:2023.

- Concrete Society Technical Report No. 61, Part 1, Enhancing reinforced concrete durability, Guidance on selecting measures for minimising the risk of corrosion of reinforcement in concrete, 2004.

- Concrete Society Technical Report No. 60, Electrochemical tests for reinforcement 2004.

AtkinsRealis
2 Chamberlain Square
Birmingham
West Mids, UK
B3 3AX