Roads and Maritime Services

14.2166.0502-0016

Windsor Bridge Rehabilitation Design of Viewing Platform

Condition Assessment and Rehabilitation Options

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December 2019

Executive Summary

Roads and Maritime Services (RMS) has decided to retain the Southern span (including pier and abutment) of the existing Windsor bridge (BN 415) as a viewing platform for heritage interpretation.

GHD has been engaged by RMS to undertake the rehabilitation design for this designated viewing platform.

There were a few steps envisaged by RMS and GHD in order to select the rehabilitation option(s) and design the selected option:

- Step 1: Existing documentation review & gap analysis. Suggest additional study/investigation required to complete the rehabilitation option selection. This Step 1 work was completed in August 2019
- Step 2: (Earlier version of this Report): Undertake additional testing/study and submit a Draft rehabilitation option recommendation for RMS's review (Earlier version of this Report)
- Step 3: (This Report) Submission of Final Report on the Rehabilitation Option, following incorporation of RMS's comments
- Step 4: Submit a Draft detailed design and Specification for the selected rehabilitation option, for RMS's review
- Step 5: Submission of a final design/specification for the selected rehabilitation option.

The Step 2 (This Report) investigation included:

- Additional Durability Testing. This is to assist RMS in taking an informed decision on the rehabilitation option (s). For example, since realkalisation repair option was considered earlier for longer life, feasibility of this option is required to be assessed.
- There were a few structural information gap that also was obtained in this Step 2 study.

It may be noted that considering the deterioration mechanisms and the extent of physical damage, the following two (2) repair options were considered earlier for the bridge concrete elements.

Option 1 – Undertake conventional concrete patch repairs to the delaminated and spalled areas followed by application of appropriate coating to the concrete elements.

Option 2 – Undertake patch repair of spalled, delaminated and cracked areas of the bridge concrete elements. Re-alkalise concrete elements and apply appropriate coating. This option was recommended earlier.

With Option 2, the re-alkalisation of the more corrosion prone elements is expected to mitigate further corrosion activity and reverse existing carbonation of concrete.

Option 1 does not demand immediate significant costs and provides an estimated remaining service life of less than 5 years. However, it is a reactive asset management approach that does not address the cause of deterioration and simply delays the time to undertake the remedial work. This is not recommended.

The remedial Option 2 is long lasting (greater than 25 years life) and is an effective remedial rehabilitation technique to address the corroding reinforcement in the concrete elements suffering from carbonation.

Based on the testing undertaken in this Step 2, it was further concluded that:

- Concrete carbonation has exceeded the mean cover reinforcement for all elements tested. Realkalisation therefore is considered to be the best solution technically. Note that the earlier testing suggested chloride induced corrosion as not the predominant deterioration mechanism for the reinforcement in this bridge.
- The reinforcement electrical continuity test results generally suggested electrical continuity presence within the elements. Therefore, an electrochemical solution such as Realkalisation is feasible.
- The headstock East section may be considered to be constructed separately from the West section. Reinforcement electrical continuity was absent between the West and East section. This is not considered to be an issue for realkalisation application, since separate reinforcement connections can be added for the East and West sections.
- The steel piers are not considered to be any worse than what was reported around ten years ago by another consultant. It is feasible to maintain the steel piers.

The existing bridge was designed for heavy traffic loading. Due to the proposed conversion of existing Span 1 to a viewing platform, the live load on the structure will be significantly reduced. The maximum live load on the viewing platform will be around 5 kPa. Based on this live load and information on the work-as-executed drawings and additional site inspection and tests, it is possible that Span 1 can be converted as a viewing platform.

Therefore, based on the Step 1 and Step 2 studies, it **can be concluded** that:

- The existing Span 1 can be converted to a viewing platform
- For long life (>25 years), realkalisation would be a suitable rehabilitation solution and this is recommended. Realkalisation shall be followed by application of a coating that is resistive to mainly carbonation and having some chloride resistance.
- The suggested rehabilitation option would include:
  - Install a sacrificial anode cathodic protection (SACP) system for the steel (cast iron) piles below water/mud section.
  - For steel (cast iron) piles above water/mud, apply a suitable coating.
  - For the concrete elements, repair all cracked/spalled/delaminated concrete. Cracks >0.3 mm shall be repaired using crack injection.
  - For concrete elements, apply realkailsation to the reinforced concrete elements. This realkalisation may not be required for the abutment and wing walls.
  - For concrete elements, apply a coating that is resistive to mainly carbonation and having some chloride resistance.
  - When the slab at the Span 1/ Span 2 interface is cut, make adequate durability provision for the reinforcement in the cut section.

We understand that new retaining wall and new barriers and topping slab will be constructed. GHD’s engagement is not to comment on the new structures to be designed and constructed. It is expected that the new elements will have adequate Durability provision for the required design life.

We assume that routine inspection as per RMS Standard procedure will be undertaken even after undertaking all repairs.

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Appendices

1. Introduction

1.1 Background

The existing Windsor Bridge is a two-lane reinforced concrete girder bridge with cast iron piers supporting the super structure over Hawkesbury River in Windsor NSW 2756. There are 11 spans and the approximate total length of 143 m.

Figure 1 - Location of Windsor Bridge (Google Maps 2019)

The original bridge was a timber bridge constructed circa 1874 and it has undergone upgrade works twice in the past, circa 1897 and 1921. In addition, reinforced concrete girders and deck have replaced the timber elements in 1931. However, the bridge has reached the end of its economic life and no longer meets the demands of current traffic volumes of present road standards as well as requiring significant ongoing maintenance.

Roads and Maritime Services (RMS) has decided to replace the bridge with a new bridge being constructed approximately 35 metres downstream from the existing bridge. The single southern span ("Span 1") of the existing bridge (including pier and abutment) is to be retained as a viewing platform for heritage interpretation. There will be new retaining wall, new deck and barriers will be constructed also.

GHD has been engaged by RMS to undertake the rehabilitation design for this designated viewing platform.

There were few steps envisaged by RMS and GHD in order to select the suitable rehabilitation option(s) and design the selected option. The first step (Step 1) was:

- Undertake initial inspection

- Review existing information and undertake an information gap analysis
- Identify additional investigation required.

This Step 1 was completed and report provided to RMS in August 2019

1.2 Information Gap Analysis Summary

The Step 1 deliverables had information gap analysis.

From the information gap analysis, the following structural information/investigation was considered required for further structural appraisal:

- Bridge slab reinforcement size, spacing and cover.
- Confirm if the reinforcement in slab at the joint between span 1 and span 2 is discontinuous.
- Concrete Grade
- Wing wall and abutment wall thickness and reinforcement size and spacing.
- Retained soil height at the abutment and wing wall
- Current cast iron pile wall thickness of pier.
- Headstock dimension.
- Check and confirm whether headstock has a joint at mid span.

The following durability information/investigation was considered missing for undertaking a proper Durability assessment.

  - Reinforcement cover depth and spacing
  - Electrical continuity of reinforcement
  - Confirm the depth of carbonation for Span 1
  - Concrete electrical resistivity.

In addition, RMS suggested (in it's review of GHD's Step 1 Deliverable) that the following testing also be included:

- Carry out a delamination survey of the span to be converted as viewing platform
- Electrical continuity testing should be carried out to assess electrical continuity of all elements that will be part of the proposed viewing platform to ensure that the repair design is suitable. Options previously put forward included electrochemical re-alkalisation which requires that all steel embedded in the affected concrete elements are electrically continuous. Testing for electrical continuity should be in accordance with AS2832.5
- Extract a minimum number of cores to assess the in-situ characteristic strength of concrete. These cores can also be used to calibrate rebound hammer tests. The calibration results can later be used in case further and wider in-situ characteristic strength assessment is required which can be carried out by NDT.

1.3 Purpose of this report

The purpose of this report is to present a suitable rehabilitation option (s), based on the earlier review of the existing documentation and also based on the tests undertaken in September 2019 (part of this step 2 work).

1.4 Scope and limitations

This report: has been prepared by GHD for RMS and *may only be used and relied on by RMS for the purpose agreed between GHD and the RMS as set out in section 1.3 of this report.*

GHD otherwise disclaims responsibility to any person other than RMS arising in connection with this report. GHD also excludes implied warranties and conditions, to the extent legally permissible.

The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report.

The opinions, conclusions and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report. GHD has no responsibility or obligation to update this report to account for events or changes occurring subsequent to the date that the report was prepared.

The opinions, conclusions and any recommendations in this report are based on assumptions made by GHD described in this report (refer section(s) 1.5. of this report). GHD disclaims liability arising from any of the assumptions being incorrect.

1.5 Assumptions and Qualifications

Following assumptions and qualifications apply to this report.

- The results of the test locations are representative of the bridge elements. We consider that this is a reasonable assumption.

2. Scope of Work

2.1 Delamination Survey

Delamination survey was undertaken to estimate the deteriorated and defective areas and to supplement the visual inspection result undertaken during the initial site visit reported in Step 1 Deliverable.

The delamination survey was undertaken from the scaffold platforms and other safe locations.

The scope of work for the delamination survey is summarised in the table below:

Table 1 Scope of Delamination Survey

Element Type	Specified (% of total surface area)
Pier Column	100
Headstock	100
Diaphragm Wall	100
Abutment Wall and Wing walls	100
Girders	100
Deck Soffit	100

Figure 2 - Windsor Bridge viewing platform elements

2.2 Diagnostic Tests

Based on the desktop reviews and the initial site inspection and RMS's review recommendations, further testing of the proposed viewing platform elements required to understand the current condition of the bridge elements included the following tests:

Concrete element tests

- Covermeter survey
- Reinforcement continuity
- Concrete resistivity
- Carbonation test
- Surface hardness
- Concrete breakouts
- Concrete compressive strength testing

Metal tests

- Ultrasonic cast iron pier wall thickness

2.3 Diagnostic testing regime

The quantities of diagnostic testing for the bridge elements are summarised in Table 2 below.

Table 2 Scope of Diagnostic Testing

Diagnostic Testing	Set of Diagnostic Test/Test Area							
	A1 South Abutment headstock	A2 – A3 South abutment wall & wing wall	A4 – A5 South abutment cast iron pier	A6 Pier 1 headstock	A7 Pier 1 diaphragm wall	A8 – A9 Pier 1 columns	A10 – A11 Girders – Outer & Inner girders	A12 Deck soffit
Delamination Survey	All the viewing platform elements							
Reinforcement Covermeter Survey	1 survey area per test area (1 x 1 = 1 Survey)	1 survey area per test area (1 x 2 = 2 Surveys)	N.A.	1 survey area per test area (1 x 1 = 1 Survey)	1 survey area per test area (1 x 1 = 1 Survey)	N.A.	1 survey area per test area (1 x 2 = 2 Surveys)	1 survey area per test area (1 x 1 = 1 Survey)
Concrete breakouts	3 breakouts per test area (3 x 1 = 3 breakouts)	3 breakouts per test area (3 x 2 = 6 breakouts)	N.A.	3 breakouts per test area (3 x 1 = 3 breakouts)	3 breakouts per test area (3 x 1 = 3 breakouts)	N.A.	3 breakouts per test area (3 x 2 = 6 breakouts)	3 breakouts per test area (3 x 1 = 3 breakouts)
Reinforcement continuity Testing	2 set (2 x 1 = 2 sets)	2 sets (2 x 2 = 4 sets plus 1 set) & 1 set for abutment wall to headstock	N.A.	2 set (2 x 1 = 2 sets)	2 sets (2 x 1 = 2 sets) & 1 set for headstock to diaphragm wall	N.A.	2 sets (2 x 2 = 4 sets plus 1 set) & 1 set for abutment wall to headstock	2 sets (2 x 1 = 2 sets) & 1 set for deck to headstock
Resistivity Testing	1 test per test area (1 x 1 = 1 test)	1 test per test area (1 x 2 = 2 tests)	N.A.	1 test per test area (1 x 1 = 1 test)	1 test per test area (1 x 1 = 1 test)	N.A.	1 test per test area (1 x 2 = 2 tests)	1 test per test area (1 x 1 = 1 test)
Carbonation Testing	3 samples per test area (3 x 1 = 3 tests)	3 samples per test area (3 x 2 = 6 tests)	N.A.	3 samples per test area (3 x 1 = 3 tests)	3 samples per test area (3 x 1 = 3 tests)	N.A.	3 samples per test area (3 x 2 = 6 tests)	3 samples per test area (3 x 1 = 3 tests)
Concrete Coring for UCS tests	N.A.	N.A.	N.A.	3 cores Ø75 mm x 160 mm long	N.A.	N.A.	3 cores Ø75 mm x 160 mm long	N.A.
Uniformity of Concrete in-situ (Surface Hardness Assessment)	2 tests per test area (2 x 1 = 1 tests)	2 tests per test area (2 x 2 = 4 tests)	N.A.	3 tests for coring locations 2 tests per test area ((2 x 1)+3 = 5 tests)	2 tests per test area (2 x 1 = 1 tests)	N.A.	3 tests for coring locations 2 tests per test area ((2 x 2)+3 = 7 tests)	2 tests per test area (2 x 1 = 1 tests)
Ultrasonic wall thickness measurements	N.A.	N.A.	3 measurements per pier (3 x 2 = 6 measurements)	N.A.	N.A.	3 measurements per pier (3 x 2 = 6 measurements)	N.A.	N.A.

Notes:

1. The cast iron pier wall thickness was measured at bottom, middle and top sections.

2.4 Structural Information Gathering

GHD has received some documents including past investigation reports and drawings from RMS for the desktop study (Step 1).

Upon the completion of the desktop study, there were some gaps in the information and GHD needed to obtain/verify those missing information for completing the structural assessment. The missing information included and not limited to slab thickness, wing wall and abutment wall dimensions, reinforcement arrangements and joint details.

3. Methodology

3.1 Access

The delamination survey and diagnostic testing were undertaken from scaffolds setup by RMS at the testing locations of the bridge elements. The scaffold arrangement during the investigation is presented in Figure 3 below.

Figure 3 Scaffold setups for the investigation at the bridge

3.2 Girder Nomenclature

The numbering system adopted for the girders is presented below.

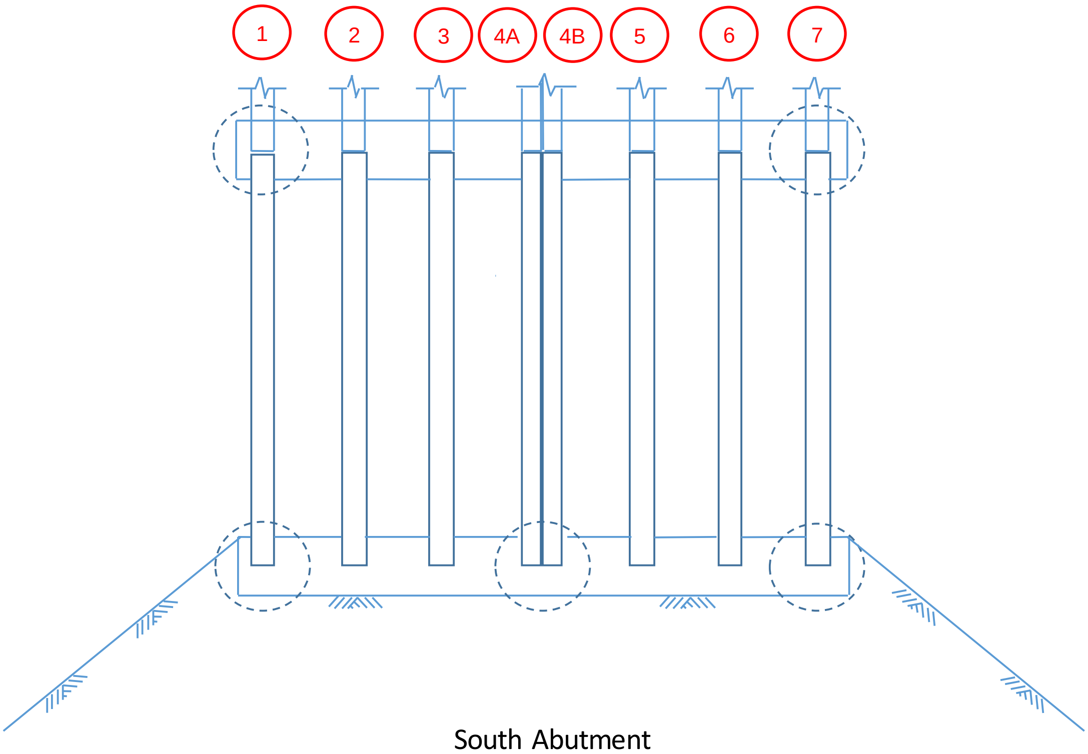


Figure 4: Nomenclature for Girders

3.3 Delamination Survey

The delamination survey was conducted on all of the accessible elements from the scaffold platforms and safe locations by lightly tapping the concrete surface areas with a hammer attached to an extension pole. The extent of delamination found in the concrete elements was estimated and recorded on the sketches.

3.4 Reinforcement covermeter survey

Covers to reinforcement were measured non-destructively using an electromagnetic covermeter on each Diagnostic Testing Area (DTA) locations. The cover readings were calibrated against physical measurements of the cover at breakout locations.

3.5 Reinforcement continuity

Reinforcement electrical continuity was assessed as per AS2832.5 by measuring electrical resistance and DC voltage between two sections of exposed reinforcement. The resistance and DC voltage were measured using a high impedance multimeter. Test points were located in each element and in the adjacent elements, to assess continuity within the elements as well as between elements.

3.6 Concrete electrical resistivity testing

The electrical resistivity of the concrete was measured on representative areas of concrete. Concrete electrical resistivity was measured using a soil resistance meter adopting the modified Wenner four probe method at 50 mm probe spacing. The probes were inserted in the drill holes.

3.7 Carbonation testing

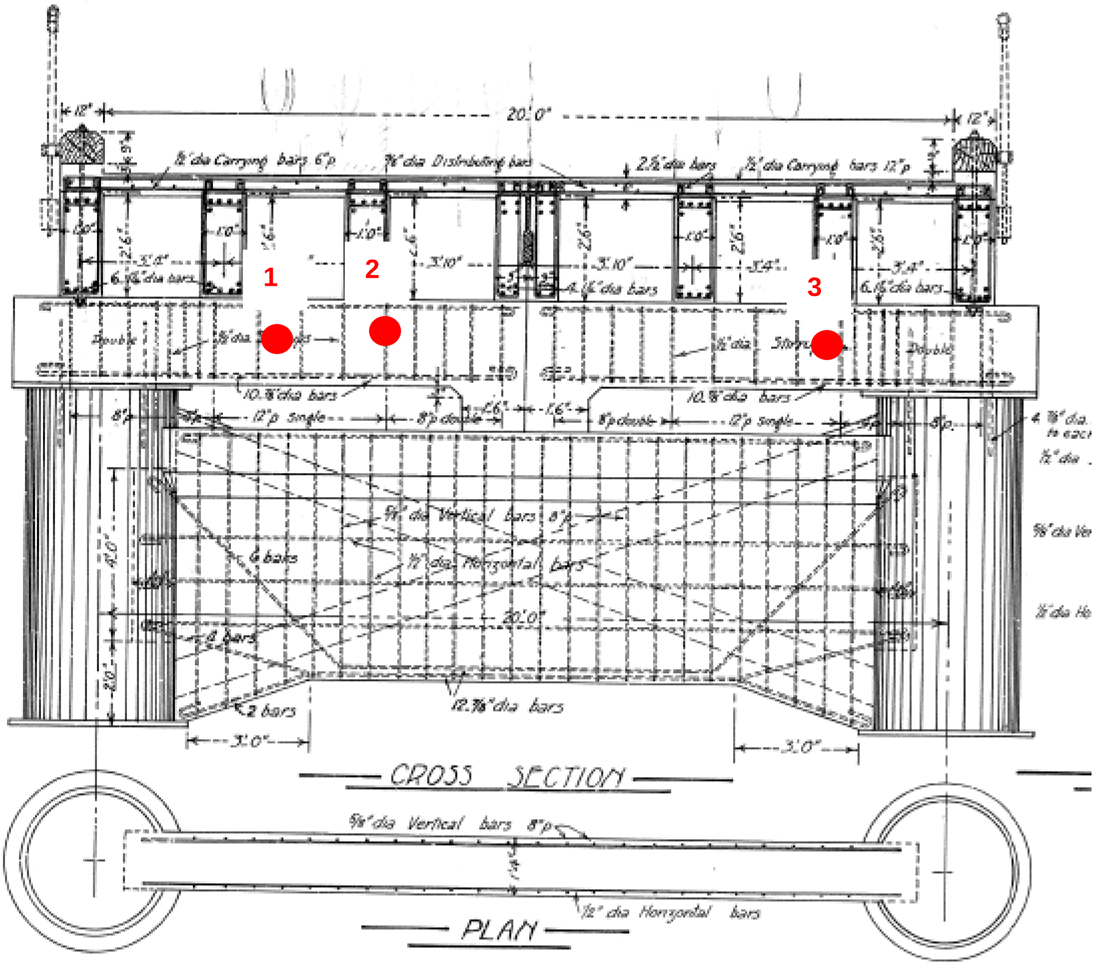
The depths of carbonation were measured using phenolphthalein as a pH indicator, sprayed onto a freshly exposed concrete face and/or the core samples. The phenolphthalein pH indicator test was in accordance with EN 14630 – 06.

3.8 Surface hardness (Schmidt Hammer test)

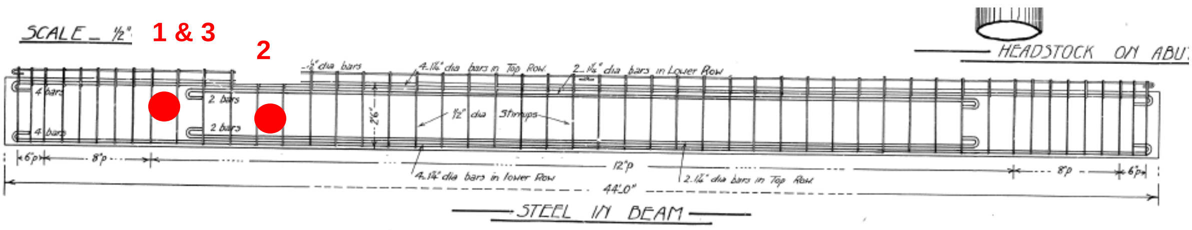
The uniformity of concrete in situ was assessed to detect suspected areas of poor quality or deteriorated concrete in each DTA in accordance with ASTM C 805 – 08 Standard test method for rebound number of hardened concrete.

3.9 Compressive strength testing

Concrete core samples were retrieved from girders and Pier 1 headstock. The cores were extracted using a core machine at different locations to minimise structural impact by selecting the least stressed areas.



2 x outside girders as the spacing between the inside girders are approximately 1 m.



The core sample were approximately Ø75 mm x 160mm long.

The cores were tested for compressive strength in accordance with AS1012.14.

3.10 Concrete breakouts

Breakouts in the bridge concrete elements were made to expose reinforcement. The breakouts were used to calibrate the covermeter readings, to assess corrosion condition of the reinforcement and to undertake reinforcement electrical continuity tests.

All breakouts were repaired using a crystalline-forming cementitious slurry, 'Xypex Concentrate' to the surface of the core hole or breakout followed by a non-shrink cementitious repair mortar. The restored surfaces was finished flush to the parent concrete.

3.11 Cast Iron Thickness Measurements

Cast iron pier wall thickness was measured using Cygnus 2 Hands Free Multiple Echo Ultrasonic Thickness Gauge. The gauge was used to take wall thickness readings from Pier 1 elements. The accuracy of the instrument is 0.1 mm (100 μm), depending on material and condition.

Figure 5: Cygnus 2 Hands Free Multiple Echo Ultrasonic Thickness Gauge

The nominated steel thickness measurement locations are shown as red dots in Figure 6 below.

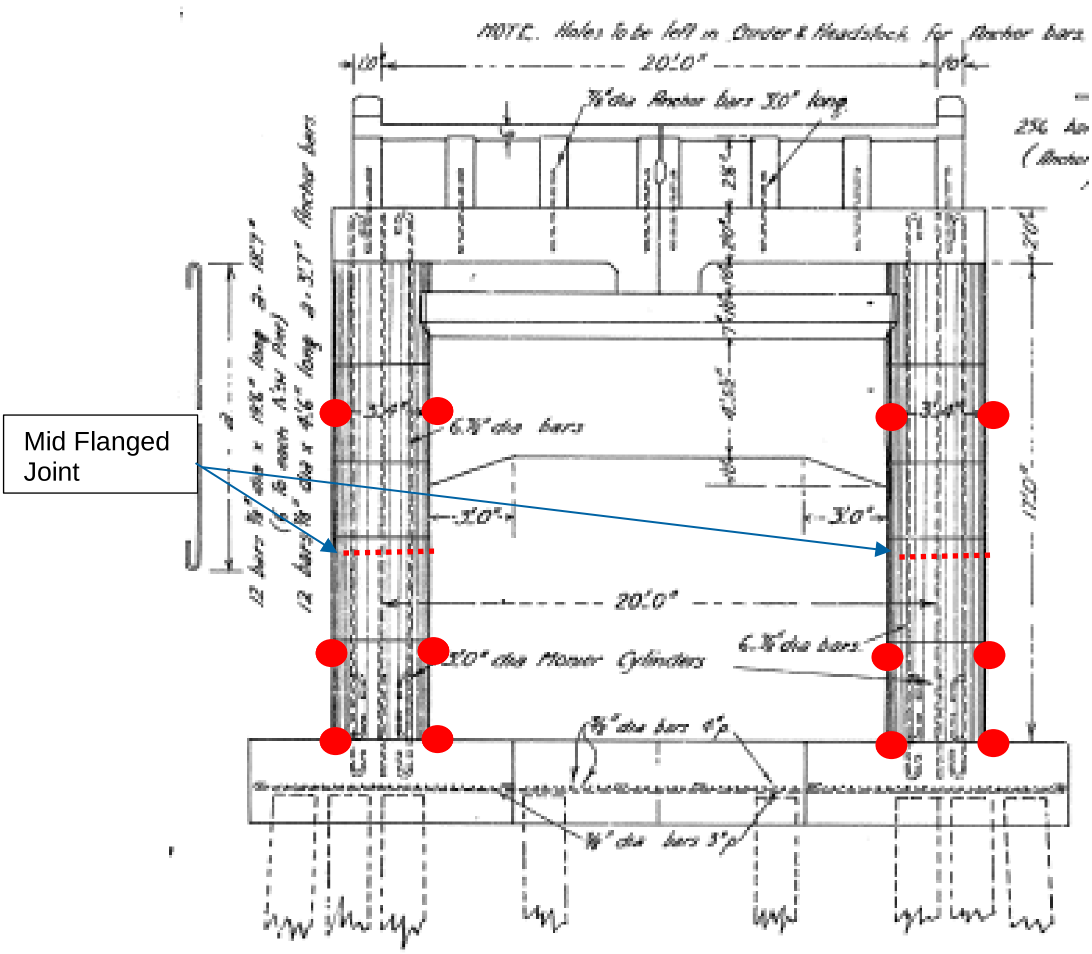


Figure 6: Pier 1 column wall thickness measurement locations

3.12 Structural Gap Information

The information required for the bridge structural assessment were identified at the completion of the desktop study (Step 1). The following information were obtained/verified during the site investigation.

1) Bridge top slab thickness, reinforcement size, spacing and cover.
2) Check whether reinforcement in slab at the joint between Span 1 and Span 2 is continuous or discontinuous
3) Wing wall and abutment wall thickness and reinforcement size and spacing
4) Retained soil height at the abutment and wing wall
5) Steel wall thickness of piers
6) Headstock dimension
7) Check whether headstock has a joint at mid span as per the drawing.

4. Results

4.1 Delamination Survey

A visual inspection and delamination survey of the concrete elements of Windsor Bridge (span 1) was undertaken to estimate the deteriorated and defective areas.

Visual inspection was undertaken on all accessible areas of the first bridge span including piers, diaphragm walls, abutment wall, headstocks, girders and deck soffit. The full results of the inspection can be found in Appendix A.

Typical observations and findings are summarised in Sections 4.1.1 to 4.1.4. The estimated areas of physical damage is provided in Appendix B

Based on the results, it appears that the Span 1 concrete elements are in a fair condition. Some areas displayed localised advanced concrete deterioration.

It may be noted that:

- The cracks were recorded visually and there was no attempt to measure the depth of the cracks. (The cracks will be appropriately repaired, hence the existing cracks should not be an issue).
- No attempt was made to measure section loss in the isolated exposed reinforcement. (Any section loss will be augmented and hence section loss at present should not be an issue).

4.1.1 Pier 1

Headstock

Headstock 1 appears to be in a fair to poor condition (Figure 7), with:

- Large areas of spalling, including an area approx. 600 x 600 mm on the South face, West side
- Two areas of spalling (150 x 150 mm on East side and 400 x 400 mm on West side approximately) on the North face.

Figure 7: Area of spalling on Headstock 1 south face, west side

Diaphragm

The Diaphragm appears to be in fair to good condition, with the only visual defect observed is an area of delamination approximately 300 x 300 mm on the North face, West side.

Cast iron piers and cross braces

The piers appear to be in a fair condition.

Numerous incidences of localised coating failure and corrosion were observed on the cast iron piers and braces. Refer Figures 8-9.

Figure 8: Span 1 cast iron piers and cross beams

Figure 9: Localised corrosion on pier

4.1.2 South Abutment

Headstock

The abutment headstock is generally in a fair to good condition, with:

- An area of spalling approximately 300 x 400 mm on the western side of the abutment headstock was present.

Abutment Wall

The abutment wall is generally in a good condition, with:

- Extensive graffiti was noted
- No delamination was observed on the abutment wall.

Figure 10: Extensive graffiti noted on south abutment wall

Wing Walls

The wing walls appear to be generally in a fair to poor condition, with:

- Area of delamination on the eastern wing wall, approximately 500 x 500 mm
- Large vertical crack on the eastern wall, approximately 1500 mm high and 5 mm width. Refer Figures 11-12.
- Area of delamination on the western wing wall, approximately 1200 mm x 200 mm
- Large horizontal crack on the western wing wall, approximately 1200 mm high and 3 mm width
- The concrete itself was noted to be quite soft (i.e. fairly low-strength).
- Based on the visual observation, it appears that the larger cracks on the wing wall are not live cracks. Until conversion of Span 1 to a viewing platform it is recommended that crack movement be monitored for safety purposes. Prior to conversion of viewing platform normal crack repair will be required as mass concrete and new retaining wall is proposed in front of the existing wing wall as per Jacobs design drawings DG-1050 and DG-1051.

Figure 11: East wing wall vertical cracking, upper

Figure 12: East wind wall vertical cracking, lower

Abutment Piers

The piers appear to be in a fair to good condition.

No notable visual defects or delamination was observed on the three piers integrated into the abutment wall.

Figure 13: Abutment piers, general condition

4.1.3 Girders

The girders appear to be in a poor condition, with:

- Numerous past repair concrete patches, many of which are now detached
- Outer girders 1 and 7 are noted to be in significantly worse condition than interior girders 2 to 6 (refer Figure 14 and Figure 15 below); interior girders are in generally fair condition however outer girders are in quite poor condition
- Areas of localised spalling, predominantly on outermost girders (Girder 1 and Girder 7), although an area of spalling is also present on Girder 3. The defect areas generally average approximately 200 mm diameter. A larger area of spalling (approx. 600 mm x 300 mm) is present on Girder 7.
- Wet surface with moss growth apparent on Girder 1 (westernmost girder)
- Cracking of approximately 1 mm width was noted on Girders 3 and 6

Figure 14: Typical interior girder condition (G2-G6)

Figure 15: Spalling on Girder 1

4.1.4 Deck Soffit

The deck soffit appears to be in a fair condition with:

- Two areas of localised spalling between Girders 5 and 6, each approximately 200 mm in diameter
- Cracking noted between each girder above piers, coincident with joints between Span 1 and Span 2 girders. Note that the deck soffit appears to have been continuously cast.

Figure 16: Crack in deck soffit between Span 1 and Span 2

4.1.5 Defect Summary

A summary of the visual and delamination survey record is listed below:

- Total spalled/delaminated areas: < 5 m^2
- Total crack length: < 5 lin m.

4.2 Diagnostic Testing

The diagnostic test results are presented in Appendix C to Appendix K. The following Sections 4.2.1 to 4.2.8 present the diagnostic test results summary.

4.2.1 Reinforcement Covermeter Survey

The full concrete reinforcement covermeter survey results are presented in Appendix C. A summary of the test result is presented in Table 3 below.

Table 3 - Covermeter Survey Result Summary

Element	Calibrated Reo Depth (mm) ± 0.5 mm					
	Horizontal			Vertical		
	Max.	Min.	Mean	Max.	Min.	Mean
Headstock Pier 1	41	32	37	25	15	22
Diaphragm Pier 1	47	32	38	19	17	18
Girder 1	36	29	32	26	16	20
Girder 7	46	46	46	21	18	20
Deck Soffit	35	22	31	55	32	44
Abutment Headstock	40	35	38	25	11	18

During the investigation, the covermeter used could not detect any reinforcement in the in Abutment wall and wingwalls. It maybe that reinforcement (if any) are deeper and not detectable by ordinary covermeter used in this investigation.

It may be noted that a Hilti PS 35 Ferro detector was used to estimate cover to reinforcement. The accuracy of measurements depends on the cover value:

Table 4 – Hilti PS 35 Ferro Detector Accuracies

Cover Value	Equipment Measurement Accuracy
5 – 60 mm	±3 mm
60 – 80 mm	±5 mm
80 – 100 mm	±7 mm
100 – 120 mm	±11 mm

The limit of the covermeter used is approximately 120 mm cover to reinforcement.

4.2.2 Electrical Continuity

One of the existing reinforcement drawing (Drawing 0182 492BC0104 Sheet 2 of 19 by the Department of Public Works) suggests that the reinforcement in concrete elements may be electrically continuous within the elements, but not across the elements. However, Drawings also suggest that reinforcement from the left part of the Headstock did not continue to the right part, i.e, they were constructed in two halves.

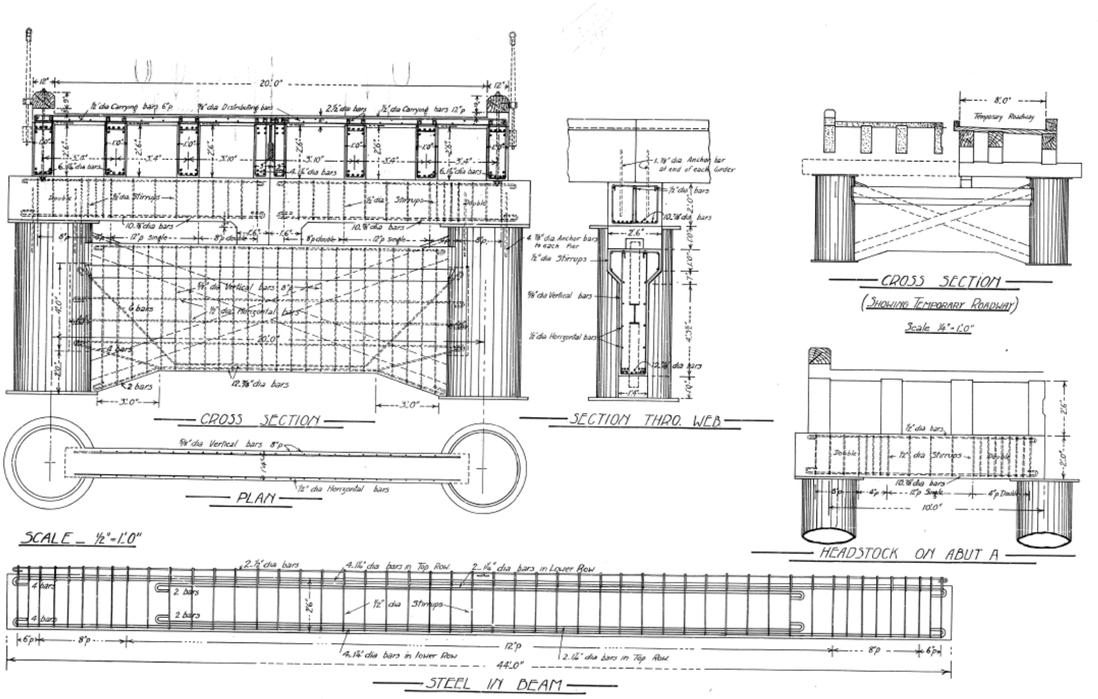


Figure 17 Department of Public Works NSW (Drawing 0182 492BC0104 Sheet 2 of 19)

For the Girders, the electrical continuity test results suggest that the reinforcement is generally electrically continuous within and between the concrete girders.

The headstocks (both abutment and Pier 1) appeared to had constructed in two halves. The existing drawing suggests that the reinforcement in headstocks are connected within the each half of the elements, but not across the halves. GHD's electrical continuity test results also support this drawing information.

The electrical continuity results for diaphragm wall suggest that the reinforcements are likely to be electrically continuous within the elements.

The electrical continuity results for deck soffit suggest that the reinforcements are most likely electrically continuous but there is an element of doubt on the continuity status.

The test results for Span 1 elements suggest that application of an electrochemical repair (such as Realkalisation) will be feasible.

The detailed results of the electrical continuity testing are provided in Appendix E.

Further commentaries on the “doubtful” electrical continuity test results and applicability of any Electrochemical treatment are provided below:

One of the criteria for electrical continuity presence is <5 mV potential difference between two pieces of reo (refer RMS B361D). The deck soffit potential difference values were -0.6mV, 1.3mV, and -0.9mV which are less than 5 mV. However, the resistance values were high at around 50 ohm, 150 ohm, and 50 ohm which are much higher than the 2-3 ohm value expected for continuity presence. We noted that none of the measured values were negative (when leads were reversed) which suggests that electrical continuity most likely is present. It is likely that the touch resistance between the two pieces of reinforcement used for measurement was high but they are not in reality electrically discontinuous. GHD has observed such anomalies in numerous structures and has noticed that when CP current is applied, the electrical continuity is restored.

If an electrochemical treatment is used, the element of doubt could be addressed by:

(a) In the trial stage, electrical continuity will be measured by applying DC current. Potential shift also will be specified to be measured. We consider that the reinforcement most likely will show electrical continuity once DC current is applied, And,

(b) A “Provisional” item in the Bill of Quantities shall be used, for achieving electrical continuity. If the trial results in (a) above suggest electrical discontinuity, the provisional item to achieve electrical continuity shall be utilised.

The headstocks (both abutment and Pier 1) appeared to have constructed in two halves. The existing drawing suggests that the reinforcement in headstocks are connected within the each half of the elements, but not across the halves. GHD’s electrical continuity test results also support this drawing information. If electrochemical repair is used, such discontinuity issue can be addressed by having separate reinforcement connections in the two halves

4.2.3 Electrical Resistivity of Concrete

The resistivity readings suggest generally low corrosion rate supported by concrete except for the Pier 1 diaphragm wall and wing wall. The readings from Pier 1 diaphragm wall and wing wall both indicated moderate corrosion rates would be supported by the concrete.

The resistivity results also suggest that concrete is generally highly resistive. However, application of an electrochemical repair (such as Realkalisation) will not be prohibitive.

The detailed results of the electrical resistivity tests are presented in Appendix F.

4.2.4 Carbonation Testing

Carbonation testing was undertaken on the diaphragm walls, south abutment, Pier 1 concrete elements, girders, deck soffit and wing wall. The carbonation test results are summarised in Table 5 below.

Table 5 – Carbonation Test Result Summary

Element	Measured carbonation depth range (mm)	Minimum concrete cover (mm)	Mean concrete cover (mm)
Abutment Wall	70 - 80	Note 1	Note 1
Abutment Headstock	20 - 30	11	17
East Wing wall	70	Note 1	Note 1
West Wing wall	>80	Note 1	Note 1
Headstock Pier 1	30 - 50	15	22
Diaphragm Pier 1	15 - 40	17	18
Girders	20 - 60	16	20
Deck Soffit	25 - 40	22	31

Note 1: Reinforcement could not be detected on the abutment wall and wing walls, by the covermeter used in this investigation.

The carbonation test results suggest that concrete carbonation has exceeded the minimum cover reinforcement for all elements and also the mean cover reinforcement for most of the elements.

It is concluded that carbonation induced corrosion is one of the main deterioration mechanism for those elements. An electrochemical repair option (such as Realkalisation) therefore will be an appropriate repair option technically.

The detailed carbonation test results are presented in Appendix G.

It may be noted that it is likely that the carbonation de-passivation may be 10 -1 5 mm ahead of the measured carbonation depth. However, the same recommended remedial options (Reakalisation recommended in this report for carbonated concrete) are applicable for increased depth of carbonation.

4.2.5 Concrete Compressive Test

Uniaxial compressive strength (UCS) testing was undertaken on the six concrete core samples taken. This testing was carried out in a laboratory offsite. Three cores were taken from Pier 1 headstock, and a further three cores from girders within Span 1. The compressive strength results of the cores are presented in Table 6 below.

The full UCS laboratory assessment results are presented in Appendix I.

Table 6: Core Samples Compressive Strength Results Summary

Area	Core No.	Detailed Test Area	Corrected Compressive Strength (MPa) **
Headstock Pier 1	H 1	6.1/C1	23.0
Headstock Pier 1	H 2	6.1/C2	32.5
Headstock Pier 1	H 3	6.3/C1	54.0
Girder 1	G 1	10.1/C1	31.0
Girder 1	G 2	10.1/C2	46.0
Girder 7	G 3	10.2/C1	50.0

** Core compressive strength corrected in accordance with AS3600 recommendations

The mean calculated compressive strength for the Headstock Pier 1 and the girders are approximately 36 MPa and 42 MPa respectively. GHD has used compressive strength values lower than the core strength values in it's structural assessment.

4.2.6 Surface Hardness Assessment

Schmidt hammer was used to assess the surface hardness of concrete elements. The surface hardness numbers are presented in Appendix H.

The surface hardness measurements were untaken on the diaphragm walls, south abutment, Pier 1 concrete elements, girders, deck soffit and wing walls, and the corresponding mean values are presented in Table 7 below.

A correlation was attempted to be established between the core compressive strength and the Schmidt hammer test results. Refer Appendix J for the correlation.

Based on the correlation established, compressive strength is estimated at locations where coring was not undertaken. Refer AAppendix H.

Table 7 – Calibrated Surface Hardness Test Result Summary

Element	Mean surface hardness values
Abutment Wall	27
Abutment Headstock	31
East Wing wall	24
West Wing wall	24
Headstock Pier 1	33
Diaphragm Pier 1	37
Girders	35
Deck Soffit	38

4.2.7 Concrete Breakouts

Concrete breakouts were made at the DTA locations and at several other locations to expose reinforcement and inspect the condition of reinforcement.

Most of the exposed reinforcement was found to have sustained minor surface corrosion.

The full concrete breakout results are presented in Appendix D.

Figure 18: General condition of reinforcement in breakouts

4.2.8 Pier Wall Thickness Survey

GHD has used Cygnus 2 Hands Free Multiple Echo Ultrasonic Thickness Gauge to measure the cast iron pier wall thickness.

The pier wall thickness survey results are included in Appendix K.

The ultrasonic gauge readings in the upper sections appeared to be affected by the acoustic property of the cast iron where reliable readings could not be obtained (i.e. Appendix K Location ID 1, 4, 7 & 10). This may be due to sound scattering where the sound energy were scattered from individual grains formed during the casting in the pier material.

The mean thicknesses measurements from the Pier 1 column are presented in table below.

Location	Mean Thickness (mm)
Top	N.A.
Middle	27
Bottom	24

The cast iron piers were originally constructed in 1875. In 1895, an additional 8 feet pier section was added to the existing piers, to raise the bridge deck out of a potential flood zone.

Coring and assessment of the piers was undertaken by CTI in 2005 and 2011. A number of core holes were drilled to directly measure wall thickness.

CTI's Pier 1 column thickness measurements from the report is reproduced below.

Date Cored	Column	Location	Casting Length (mm)	Residual Cast Iron (mm)
10/03/2005	East (Downstream)	Above water, above flange of upper section (column extension, close to the GHD's top measurement location)	35	35
10/03/2005	East (Downstream)	300 mm below flange (Close to the GHD's middle measurement location)	26	26
10/03/2005	East (Downstream)	1600 mm below flange (Close to the GHD's bottom measurement location)	25	25

Above Tidal Zone

Design drawings for the Windsor Bridge indicated a 1.25 inch pier wall design thickness, or approximately 31 mm. The piers were noted to be concrete-filled.

The top core location was noted on the Pier 1 upper castings by GHD inspectors, which were re-measured. These core locations indicated a wall thickness of approximately 35 mm of the upper pier half, corresponding to measurements obtained by CTI in 2005. On this basis, the upper pier segment appears not to display any significant section loss at the measured location.

Figure 19: Pier 1 East Column core hole in the upper section (Left) and the lower section (Right) from the past investigation

The CTI's measurement on Pier 1 column was 26 mm. The GHD's ultrasonic gauge mean reading was 27 mm, which is close to the CTI's measured value.

The lower casting above water level was found in 2005 to be generally 23 mm thick on average. The original wall thickness of the casings including residual cast iron and graphitisation was approximately 38 – 40 mm, indicating significant section loss of at least 15 mm on average due to graphitisation.

Within & Below Tidal Zone

The CTI’s measurement on Pier 1 column 1600 mm below the flange was 25 mm. The GHD’s ultrasonic gauge mean reading is 24 mm, which is close to the CTI’s measure value.

Surface protrusions similar to nodules were noted by CTI in 2005 below the tidal zone. The maximum thickness of these protrusions and the corrosion product was approximately 55 mm, and 20 – 30 mm on average.

Remaining wall thickness of cast iron as measured by CTI in 2011 was found to vary significantly, from a maximum 27 mm to a minimum of 2 mm. Graphitisation was found to be frequently in excess of 20 mm, such that residual cast iron thickness would frequently less than 10 mm.

Cleaning and application of suitable protective coatings above the tidal zone can minimise the graphitisation of the cast iron piers by limiting the availability of oxygen for the chemical process at the metallic interfaces.

For the within and below tidal zone, a sacrificial anode cathodic protection (SACP) may be suitable for protecting the cast iron pier sections.

4.3 Structural Information

The information required for the bridge structural assessment were obtained during the site investigation and presented below.

4.3.1 Bridge slab thickness, reinforcement size, spacing and cover

The bridge slab thickness is approximately 245 mm thick from the measurements on site. Based on the concrete breakout and covermeter survey results, the reinforcement appears to be ≈ Ø12 mm bars laid in a mesh pattern with spacing of 150 mm to 200 mm centre to centre. The measure concrete cover to the reinforcement (on the deck soffit) was approximately 35 mm.

4.3.2 Check whether reinforcement in slab at the joint between Span 1 and Span 2 is continuous or discontinuous

A covermeter survey undertaken on the deck soffit at the Span 1 and Span 2 joint area suggests that the reinforcement in the slab at the joint are continuous.

From durability consideration, the above suggests that when the slab is cut at the Span 1/ Span 2 interface, the exposed slab reinforcement should be treated adequately against future corrosion.

In the visual inspection, there are transverse cracks where a joint should have been made in the deck between Span 1 and Span 2 to prevent the cracking in the deck (See Section 4.1.4).

4.3.3 Wing wall and abutment wall thickness and reinforcement size and spacing

The wall thickness is estimated to be approximately 300 mm.

The covermeter used could not detect any reinforcement in the wing walls and abutment wall.

4.3.4 Retained soil height at the abutment and wing wall

The height of abutment wall and wing walls from the existing ground level were measured up to 2150 mm and 3500 mm respectively.

4.3.5 Cast iron wall thickness of piers

The cast iron wall thickness of Pier 1 columns were measured at various locations (See Appendix K). The cast iron wall thicknesses ranged between 24 mm to 35 mm thick for the Pier 1 columns.

4.3.6 Headstock dimensions

The Pier 1 and abutment headstocks were measured and the dimensions were 1) Length = 7400 mm, 2) Width = 765 mm and 3) Height = 600 mm. The measurements appear to be consistent with the existing drawing measurements.

4.3.7 Check whether headstock has a joint at mid span as per the drawing

One of the existing drawing showed a joint in Pier 1 headstock. During the site investigation, this joint was observed in the headstock. The reinforcement electrical continuity test results supported this observation.

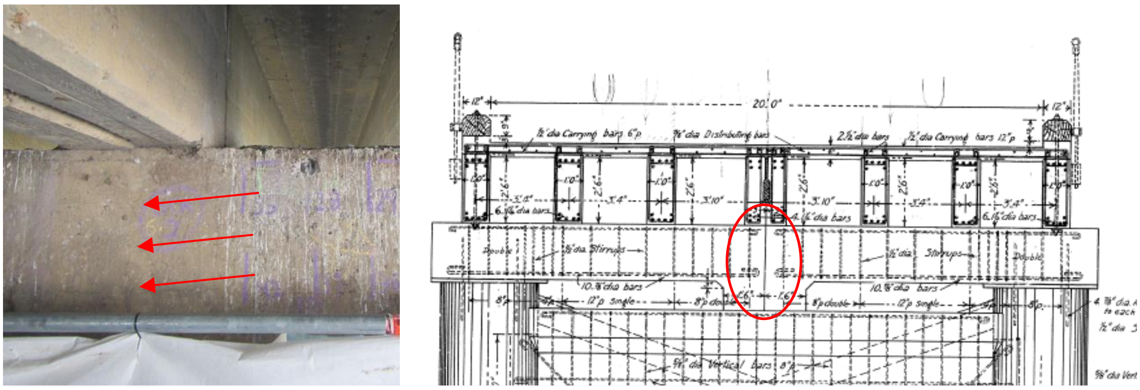


Figure 20: The joint in Pier 1 headstock directly under Girders 4A & 4B (Left) and in the drawing (Right)

5. Discussion

5.1 Visual Inspection and Delamination Survey

The visual inspection and delamination survey was conducted on all accessible areas of Span 1 including piers, diaphragm walls, abutment wall, headstocks, girders and deck soffit.

The concrete elements exhibited a fair to poor condition with some past concrete repairs and numerous areas of concrete deterioration present, especially in outer girders 1 and 7. The interior girders (2 to 6) and deck soffit were generally in a fair condition.

It is noted on the Jacobs' Windsor Bridge viewing platform detailed concept design drawings that new abutment wing walls will be constructed. No action is noted for the large cracks found the wing walls in this report.

A summary of the visual and delamination survey is listed below:

- Total spalled/delaminated areas: < 5 m^2
- Total crack length: < 5 m

5.2 Diagnostic Testing

The diagnostic test results confirm that the bridge concrete elements proposed for the viewing platform are suffering from carbonation induced corrosion. The results are consistent with the past investigation findings. Chloride induced reinforcement corrosion was not reported to be present in earlier reports.

The electrical resistivity of the bridge concrete elements are generally high and low corrosion rate will be supported by concrete except for the Pier 1 diaphragm wall and wing walls where moderate corrosion rates are expected to be supported by concrete. The electrical resistivity results also suggest that an electrochemical repair technique such as realkalisation may be applied as the concrete is not extremely dry.

The electrical continuity test results indicate that the reinforcements are generally electrically continuous within elements. There is an element of doubt on reinforcement electrical continuity status for the deck soffit. Also, the reinforcement appear to be generally electrically discontinuous across the concrete elements except for the girders. The above electrical continuity results suggest that an electrochemical repair technique such as realkalisation is feasible.

The compressive strength and Schmidt hammer test results indicates that the compressive strengths are generally in the order of 30 – 40 MPa for the concrete elements except for the abutment and wing walls where lower compressive strengths of 25 – 30 MPa were estimated based on the test results.

As noted in the 2011 underwater Diving Inspection report, the condition of the cast iron piers below the tidal zone is potentially poor. That report does not have information on condition of the Piers in mud. Above water level, the pier condition is noted as better.

The earlier CTI inspection report suggested presence of graphitisation of the cast iron.

Therefore, the recommendation of "regularly monitor the graphitisation of pier columns" in the 2013 RMS report ("Performance load testing & investigation of deck & pier (2011 & 2012 report – Bridge over Hawkesbury River on Bridge St/Wilberforce Road (MR182) at Windsor (B415)", dated February 2013) is deemed applicable. However, protection measure should be used to minimise on going graphitisation.

5.3 Structural Adequacy

The existing bridge was designed for heavy traffic loading. Due to proposed conversion of existing Span 1 to a viewing platform, the live load on the structure will be significantly reduced. The maximum live load on the viewing platform will be around 5 kPa. Based on this live load and information on the work-as-executed drawings and additional site inspection and tests, it is possible that Span 1 can be converted as a viewing platform.

Further commentaries on structural assessment are provided below:

- GHD's preliminary assessment suggests that once the cracks in Girders are repaired using crack injection, the crowd loading (5 KPa) can be easily supported by the structures.
- GHD has noted during site inspection that, only limited number (1-2) reinforcement were exposed associated with cracks in girder concrete. Even if 2-3 bars are missing (which is unlikely), the structure will be adequate for 5 KPa load according to our structural analysis. GHD however will recommend augmenting any reinforcement that has lost >10% of it's section due to corrosion.

Regarding the cast iron pile, GHD's further commentaries include:

- The 15 mm section loss indicated in the report refers to the average of the section loss measured by CTI in their 2011 report. In Pier 1, the loss is less, nil according to CTI 2005 Report and it indicates 25mm remaining thickness
- GHD's preliminary assessment suggests that with the assumed section loss (assuming 15mm loss, that means 10mm remaining) due to graphitization (on Pier 1), the crowd loading (5 KPa) can be easily supported by the structures as per our structural analysis. As a conservative approach, we have assumed 15mm section loss circumferentially.

Refer Appendix L for structural analysis calculations.

6. Conclusions and Recommendations

The Windsor Bridge is found to be generally in a poor condition. It is approximately 144 years old and such poor condition is not unexpected. However, Span 1 can be converted into a viewing platform and a life in excess of 25 years is possible if the defects/deterioration are addressed.

6.1 Concrete Elements

The investigation results suggest a strong indication of active corrosion presence in all of the concrete elements due to the significant carbonation.

Considering the deterioration mechanisms and the extent of physical damage, the following two (2) repair options are considered for the bridge concrete elements.

Option 1 – Undertake conventional concrete patch repairs to the delaminated and spalled areas followed by application of coating to the elements.

Option 1 does not demand immediate significant costs and provides an estimated remaining service life of less than 5 years. However, it is a reactive asset management approach that does not address the cause of deterioration and simply delays the time to undertake the remedial work or condemn the structure.

Also, Option 1 will have the serviceability concerns. We understood that RMS has an intention to have minimal maintenance regime for the design life required. Option 1 will not provide the required minimal maintenance regime required.

This Option 1 therefore is not recommended.

Option 2 – Undertake patch repair of spalled and delaminated areas in the bridge concrete elements. Re-alkalise concrete elements and apply a coating that is resistive to mainly carbonation and having some chloride.

This remedial option (Option 2) is an effective remedial option to prevent the onset of reinforcement corrosion or stop the progress of reinforcement corrosion in elements already corroding. This option is long lasting (greater than 25 years) and is very effective. This option is recommended.

The investigation results suggest that realkalisation is feasible. Such technique requires only one time application and should provide life in excess of 25 years. The existing defects however will require to be rectified prior to realkalisation application

The Abutment wall and wing wall will not require electrochemical treatment due to cover value being too high at >120mm.

We understand that new retaining wall and new barriers and topping slab will be constructed. GHD's engagement is not to comment on the new structures to be designed and constructed. It is expected that the new elements will have adequate Durability provision for the required design life.

Regarding durability for the exposed reinforcement in slab at the span 1 / 2 interface, the exposed reinforcement are to be cut back 50 mm into the concrete and the void patched with minimum 40 MPa cementitious repair material. According to AS5100.5 Table 4.14.3.2, a minimum cover of 45 mm is required for 40 MPa concrete in B1 environment prevailing at the Bridge site. Therefore, the cover of 50 mm should be adequate for 100 years life.

6.2 Cast Iron (Steel) Elements

The earlier underwater Diving Inspection report suggested the condition of the cast iron piers below the tidal zone to be potentially poor. That report does not have information on condition of the Piers in mud. Above water level, the pier condition was noted as better.

The CTI report and GHD's steel thickness measurement suggest that the thickness has not significantly lost in the last ten years since the CTI measurements. Graphitisation however is an issue that require to be addressed.

We suggest that:

- The section above water level be coated with a suitable epoxy coating
- A sacrificial anode catholic protection (SACP) be applied for the section below water level (and in mud) which will also provide some protection to the inter-tidal zone.

6.3 Conclusion

The existing bridge was designed for heavy traffic loading. Due to the proposed conversion of existing Span 1 to a viewing platform, the live load on the structure will be significantly reduced. The maximum live load on the viewing platform will be around 5 kPa. Based on this live load and information on the work-as-executed drawings and additional site inspection and tests, it is possible that Span 1 can be converted as a viewing platform.

Therefore based on the studies, it **can be concluded** that:

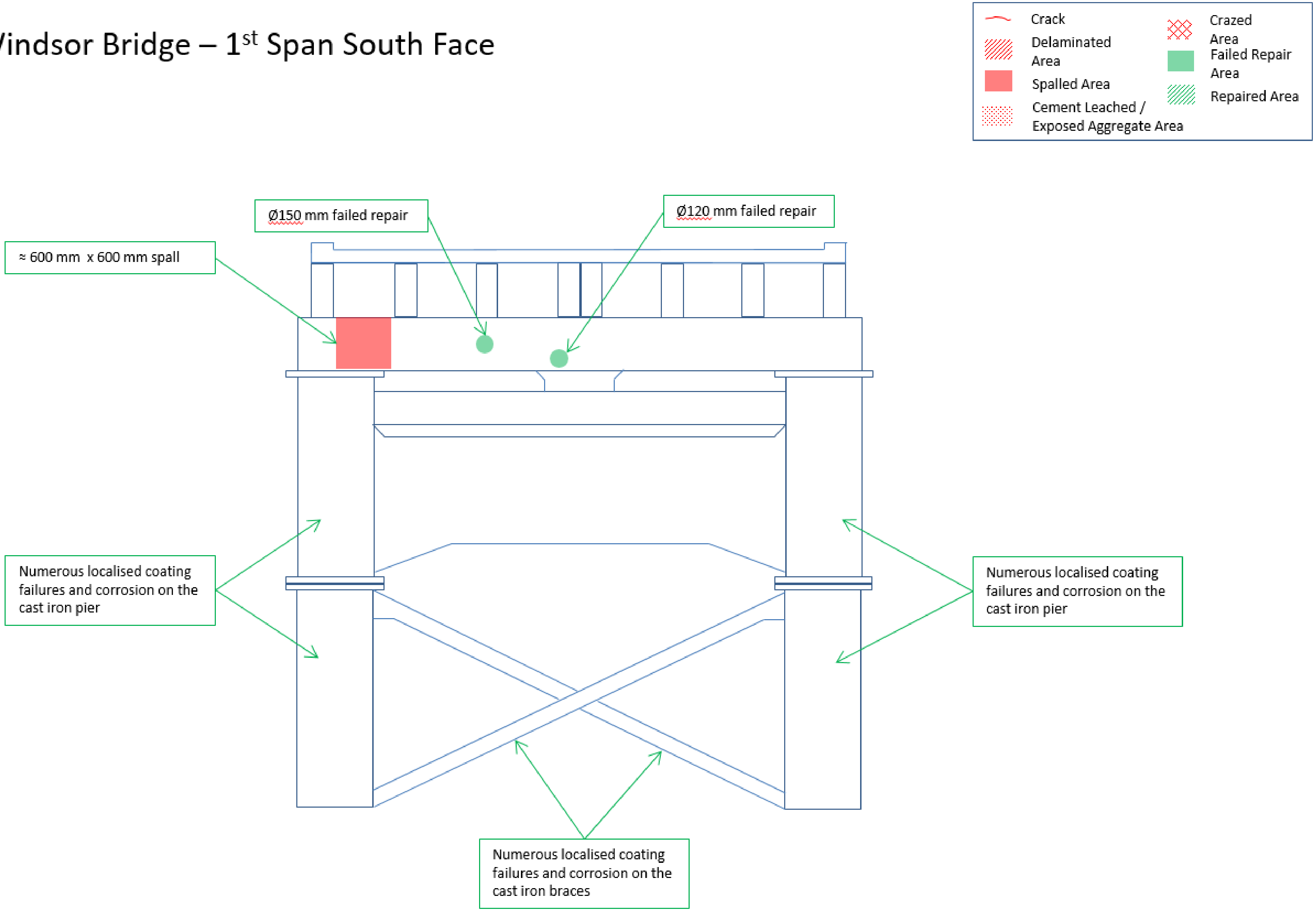
- The existing Span 1 can be converted to a viewing platform
- For long life (>25 years), realkalisation would be a suitable rehabilitation solution and this is recommended. Realkalisation shall be followed by application of a coating that is resistive to mainly carbonation and having some chloride resistance.
- A suitable rehabilitation option would include:
  - Install a sacrificial anode cathodic protection (SACP) system for the steel (cast iron) piles below water/mud section.
  - For steel (cast iron) piles above water/mud, apply a suitable coating.
  - For the concrete elements, repair all cracked/spalled/delaminated concrete. Cracks >0.3 mm shall be repaired using crack injection.
  - For concrete elements, apply realkailsation to the reinforced concrete elements. This realkalisation may not be required for the abutment and wing walls.
  - For concrete elements, apply a coating that is resistive to mainly carbonation and having some chloride resistance.
  - When the slab at the Span 1/ Span 2 interface is cut, make adequate durability provision for the reinforcement in the cut section.

We understand that new retaining wall and new barriers and topping slab will be constructed. GHD's engagement is not to comment on the new structures to be designed and constructed. It is expected that the new elements will have adequate Durability provision for the required design life.

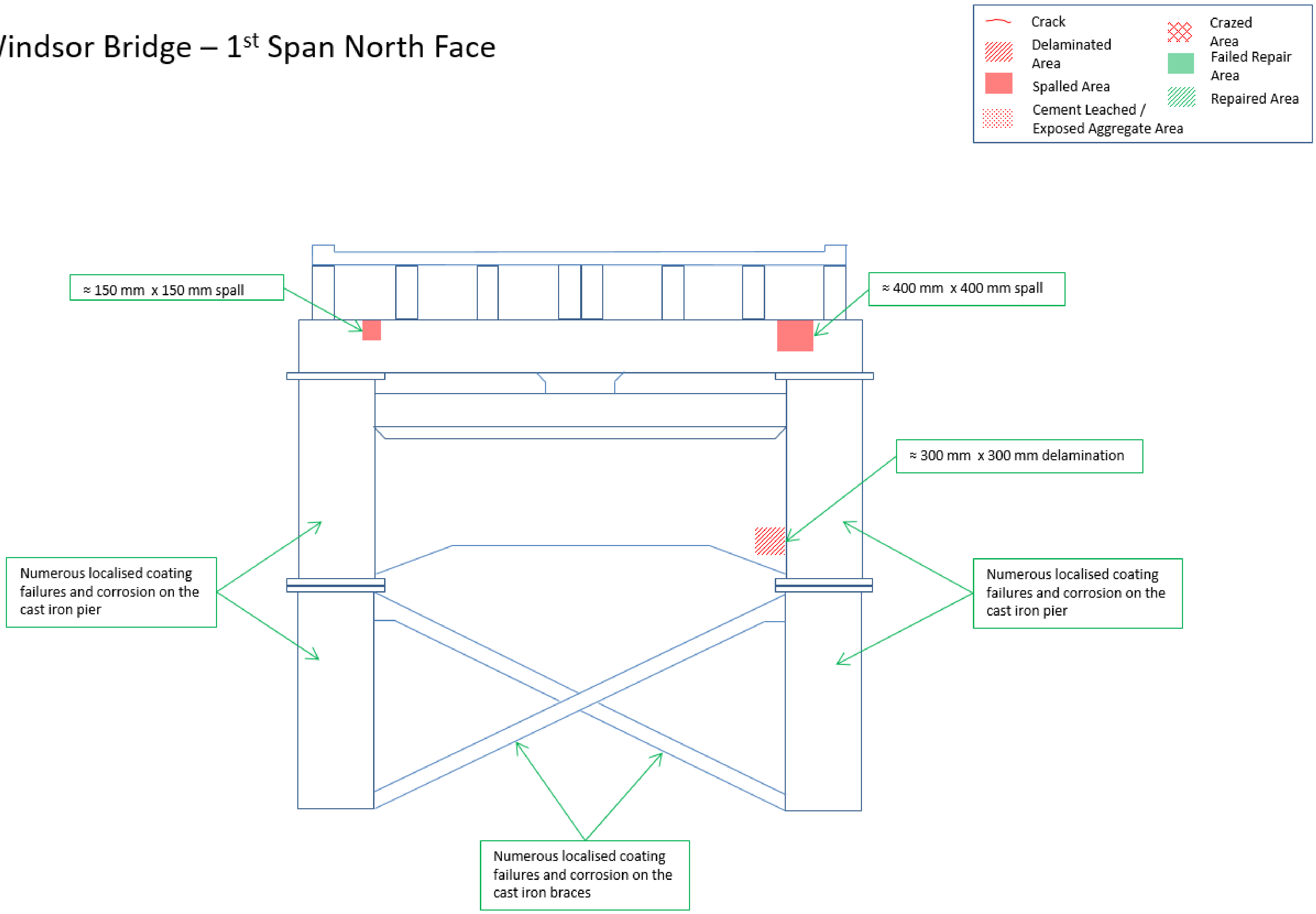
We assume that routine inspection as per RMS Standard procedure will be undertaken even after undertaking all repairs.

Appendix A – Visual and Delamination Survey Results

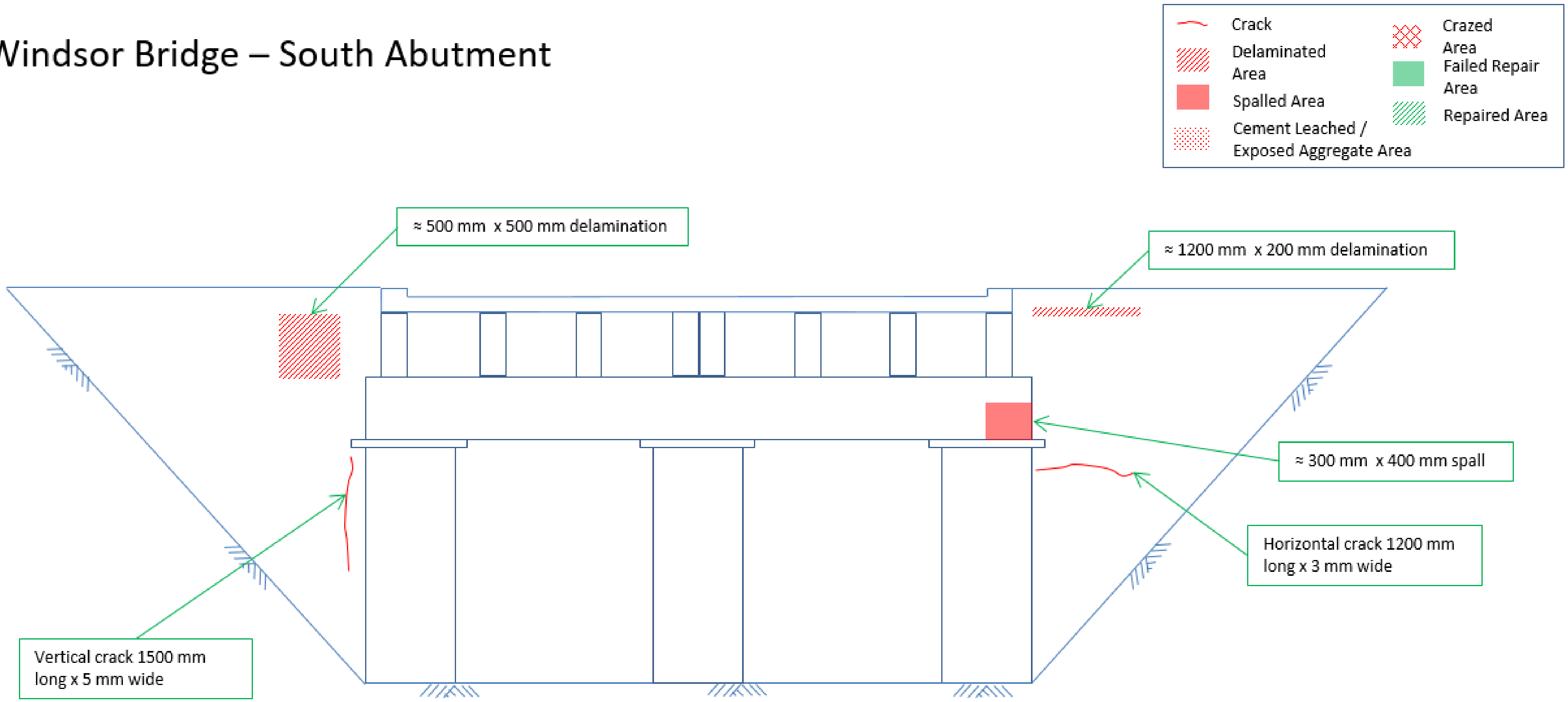
Windsor Bridge – 1st Span South Face



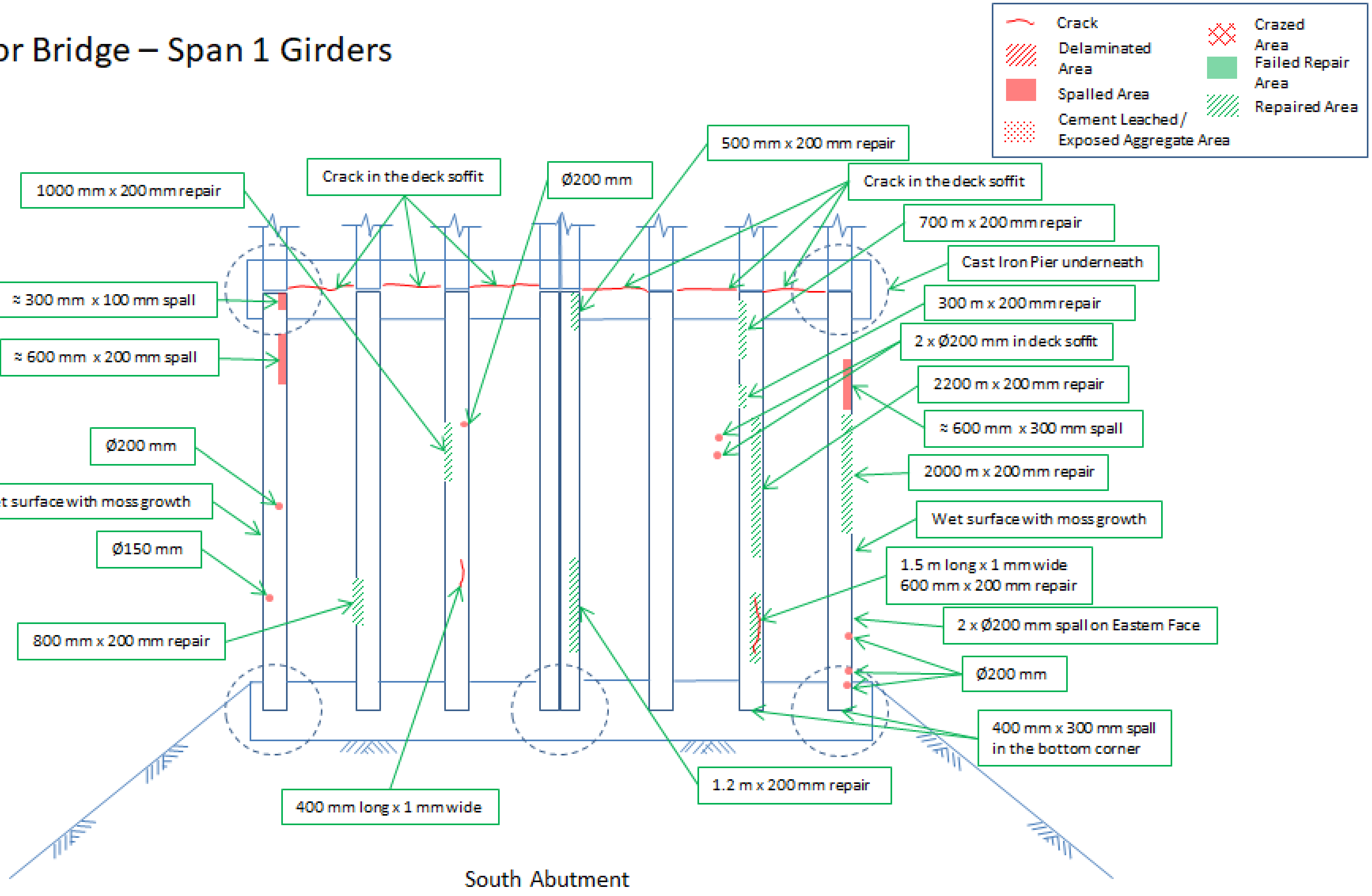
Windsor Bridge – 1st Span North Face



Windsor Bridge – South Abutment



Windsor Bridge – Span 1 Girders



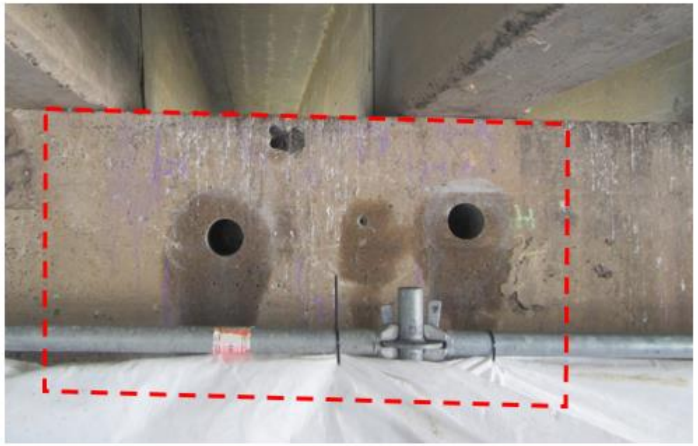
Appendix B – Estimated areas of physical concrete damage and corrosion activity

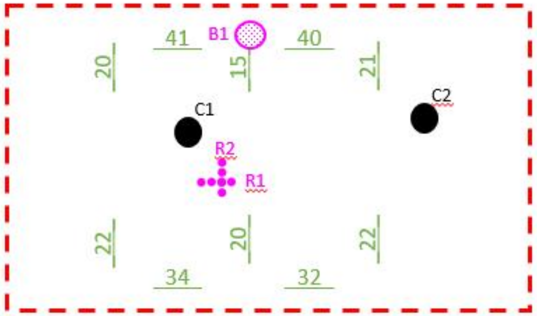
Element	Dominant Corrosion Mechanism	Physical Concrete Damage			Corrosion Activity		
		Area (m^2)	≈ % total surface area	Comments	Area (m^2)	≈ % total surface area	Comments
Abutment Wall	No corrosion (unreinforced)	< 0.1	< 1%	Large angular aggregate and relatively softer concrete	< 0.2	< 1%	No reinforcement detected by the covermeter used in this investigation.
Wing Walls	No corrosion (unreinforced)	< 0.5	< 1%	Large angular aggregate and softer concrete. 2 x cracks, 1.5 m and 1.2 m length	< 1	< 1%	No reinforcement detected by the covermeter used in this investigation.
Abutment Headstock	Carbonation Induced Corrosion	< 0.2	< 1%	Localised spall only	Entire	100%	Surface corrosion on the reinforcement observed in the breakouts.
Girders	Carbonation Induced Corrosion	< 2.5	< 1%	2 x cracks, 1.5 m and 0.4 m length	Entire	100%	Surface corrosion on the reinforcement observed in the breakouts. Corroded reinforcement observed where spalled
Deck Soffit	Carbonation Induced Corrosion	< 0.5	< 1%	6 x cracks, each ≈1 m in length in Span 2 deck soffit.	Entire	100%	Surface corrosion on the reinforcement observed in the breakouts.
Pier 1 Headstock	Carbonation Induced Corrosion	< 0.6	< 1%	Localised spalls only	Entire	100%	Surface corrosion on the reinforcement observed in the breakouts.
Pier 1 Diaphragm	Carbonation Induced Corrosion	< 0.1	< 1%	Localised spall only	Entire	100%	Surface corrosion on the reinforcement observed in the breakouts.

***Total damaged area requiring repair is estimated to be < 5 m^2 and crack repair < 5 m**

Appendix C – Covermeter Survey Results

Pier 1 Headstock (South Face Eastern Side)





Typical reo spacing:
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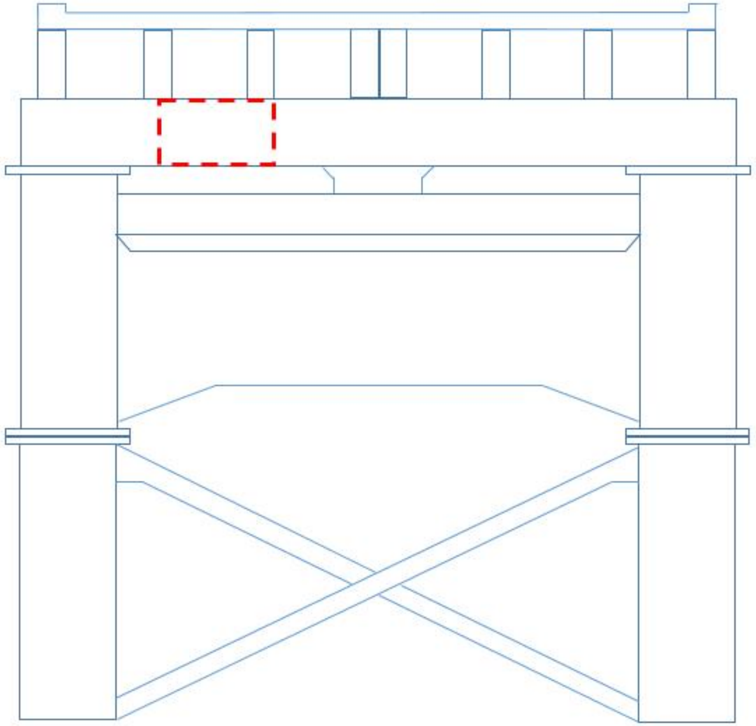
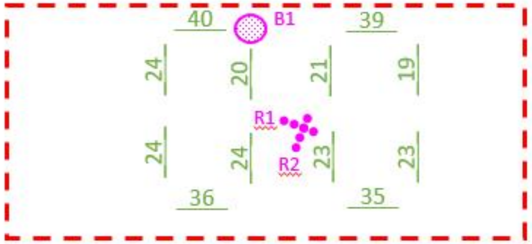
Horizontal = 550 mm



Key

- C1 Core Sample
- B1 Concrete Breakout
- Test Area
- R1 Concrete Resistivity
- 80 Calibrated Reinforcement Cover (mm)

Pier 1 Headstock (South Face Middle Section)



Typical reo spacing:
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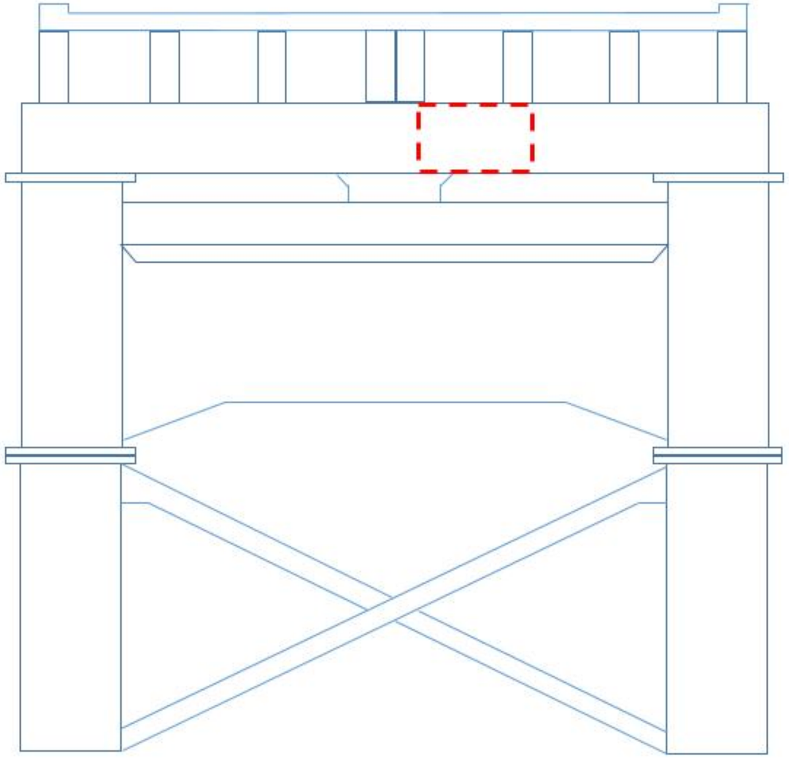
Horizontal = 550 mm

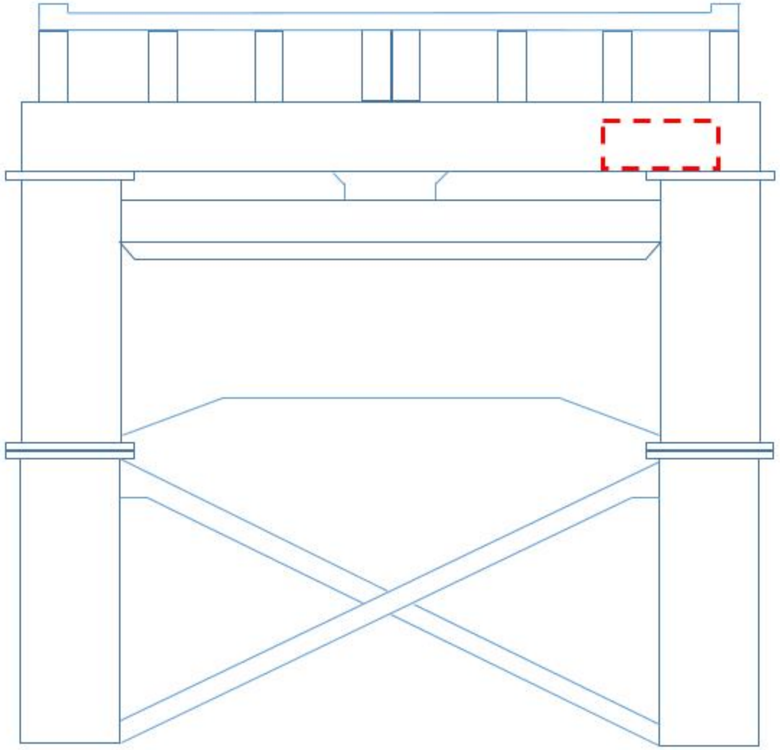


Key

- C1 Core Sample
- B1 Concrete Breakout
- Test Area
- R1 Concrete Resistivity
- 80 Calibrated Reinforcement Cover (mm)

Pier 1 Headstock (South Face Western Side)

Typical reo spacing:
Vertical = 100 - 300 mm



Key

C1 Core Sample

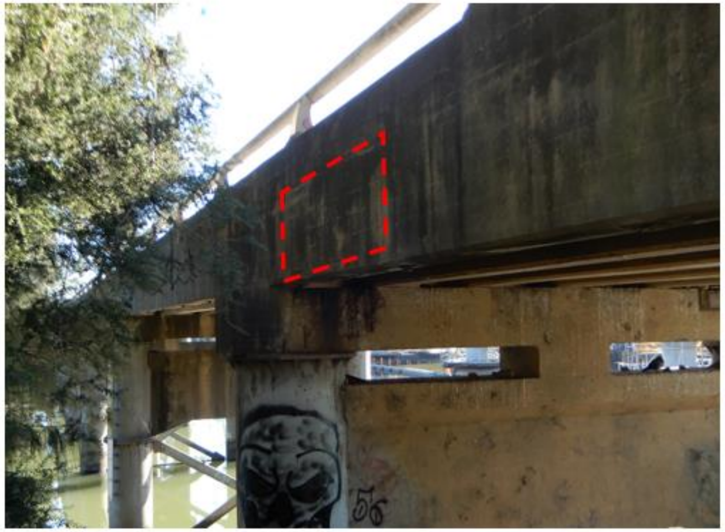
B1 Concrete Breakout

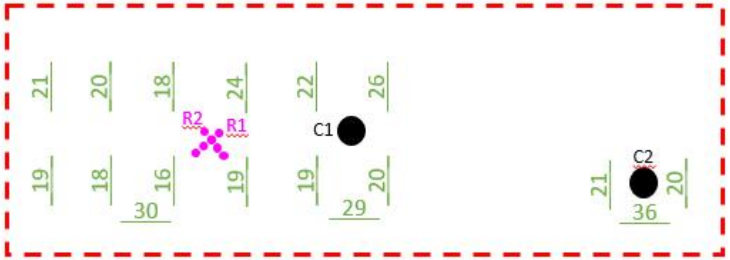
Test Area

R1 Concrete Resistivity

80 Calibrated Reinforcement Cover (mm)

Girder 1 (North End – West Face)

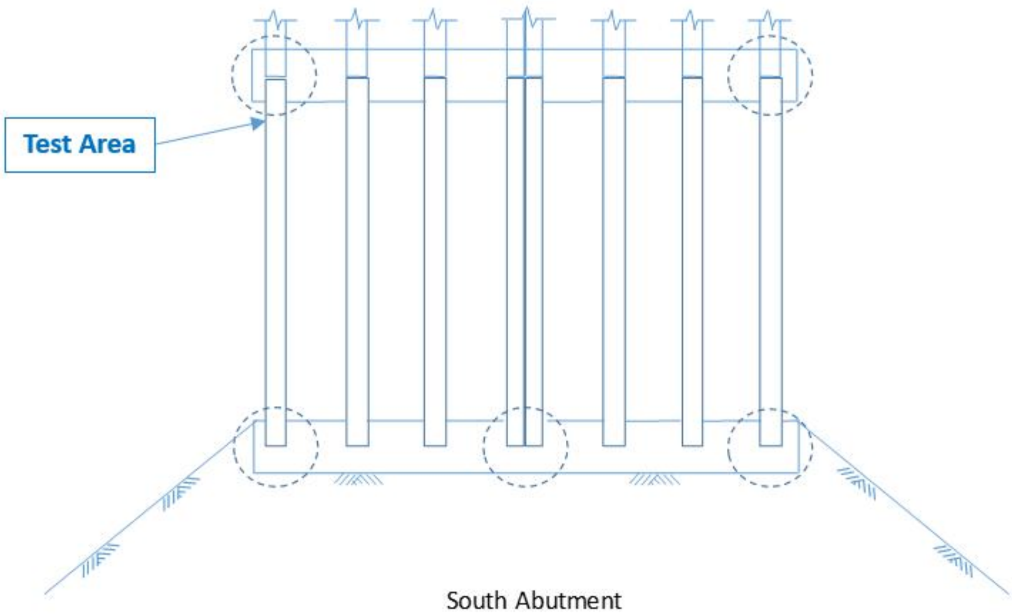




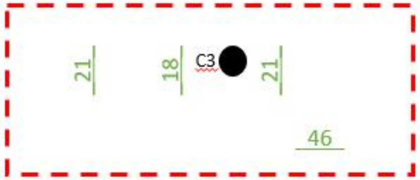
Typical reo spacing:
Vertical = 200 - 300 mm
Horizontal = 650 mm

Key

- C1 Core Sample
- B1 Concrete Breakout
- Test Area
- R1 Concrete Resistivity
- 80 Calibrated Reinforcement Cover (mm)



Girder 7 (North End – East Face)



Typical reo spacing:
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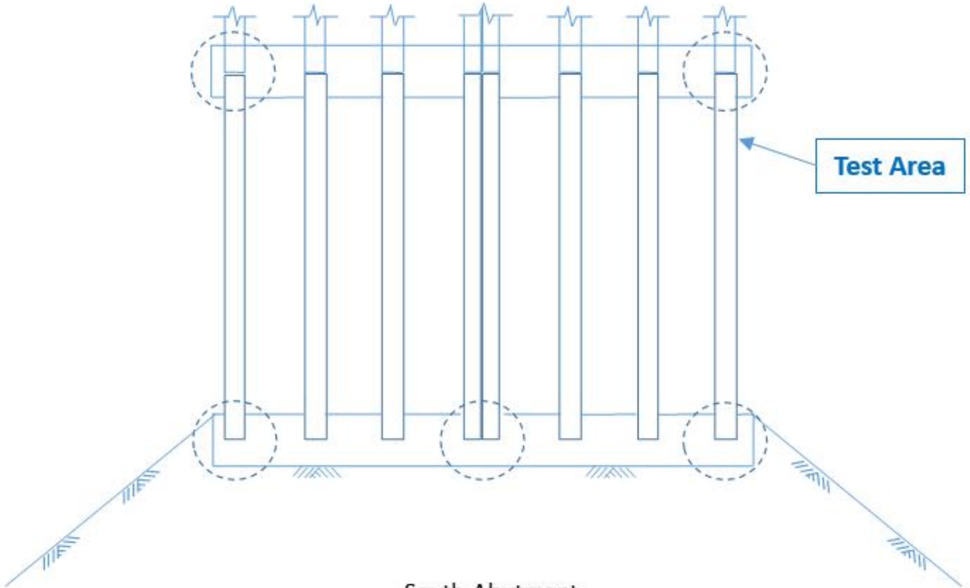
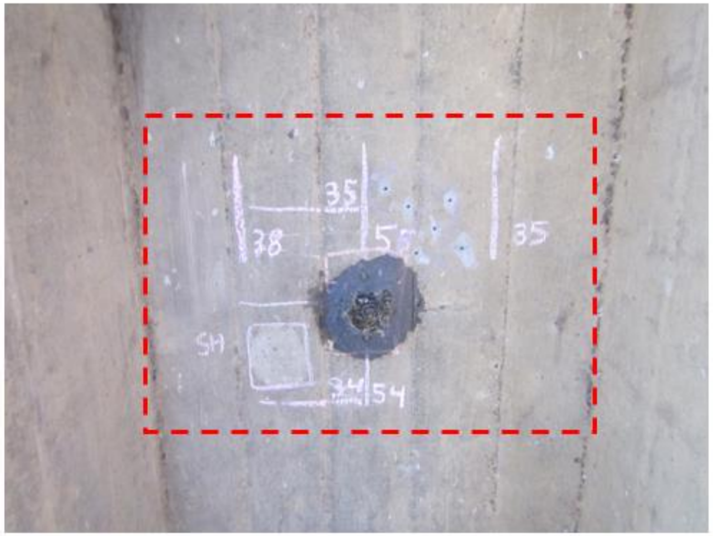
Horizontal = 550 mm

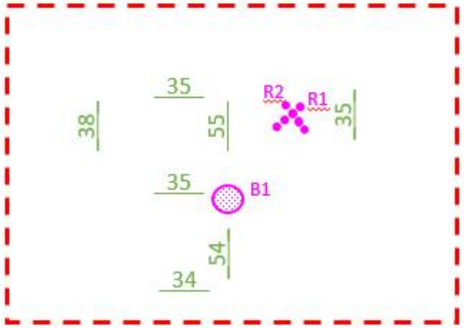


Key

C1 Core Sample	R1 Concrete Resistivity
B1 Concrete Breakout	80 Calibrated Reinforcement Cover (mm)
Test Area	

Deck Soffit between Girders 6 - 7 (North End)

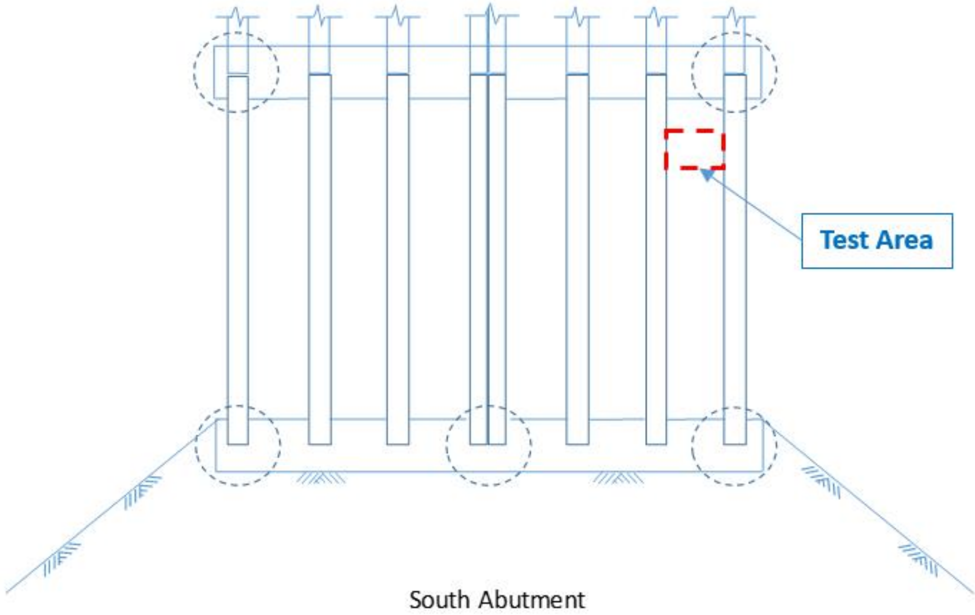




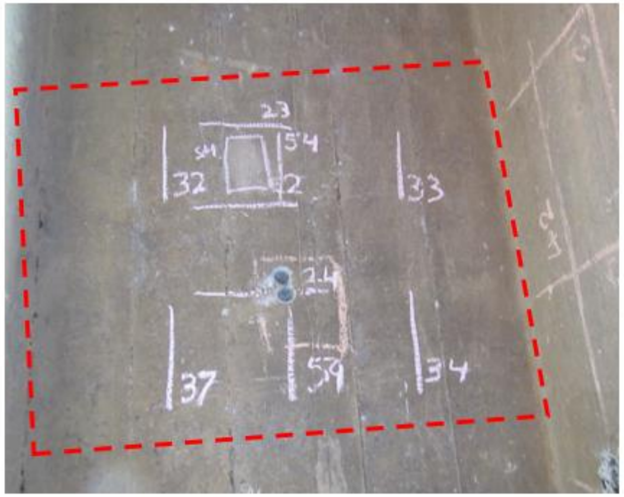
Typical reo spacing:
Vertical = 200 mm
Horizontal = 150 mm

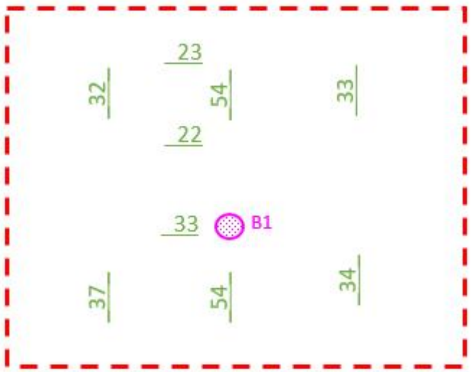
Key

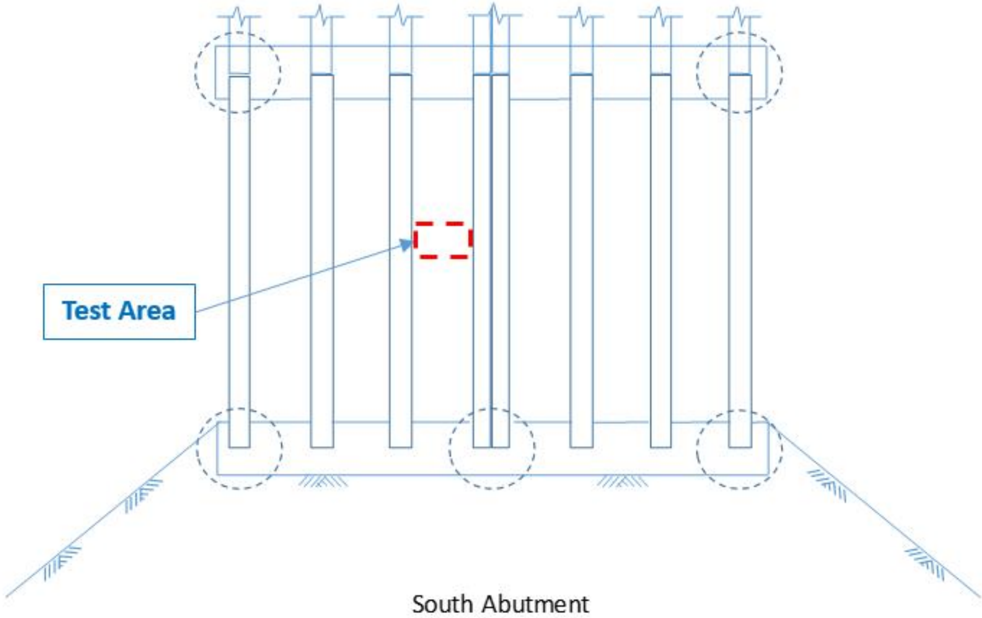
- C1 Core Sample
- B1 Concrete Breakout
- Test Area
- R1 Concrete Resistivity
- 80 Calibrated Reinforcement Cover (mm)



Deck Soffit between Girders 3 – 4A (Middle Section)





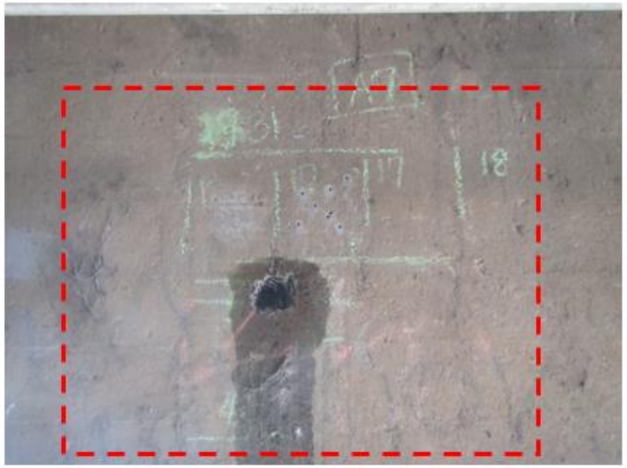


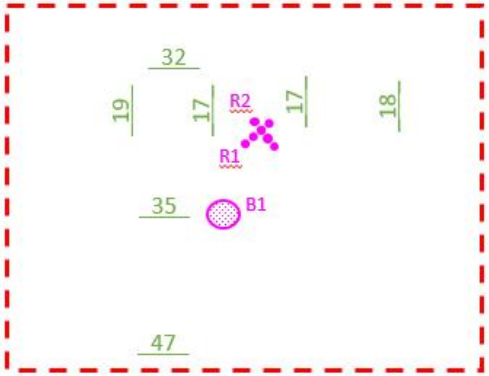
Typical reo spacing:
Vertical = 150 mm
Horizontal = 150 mm

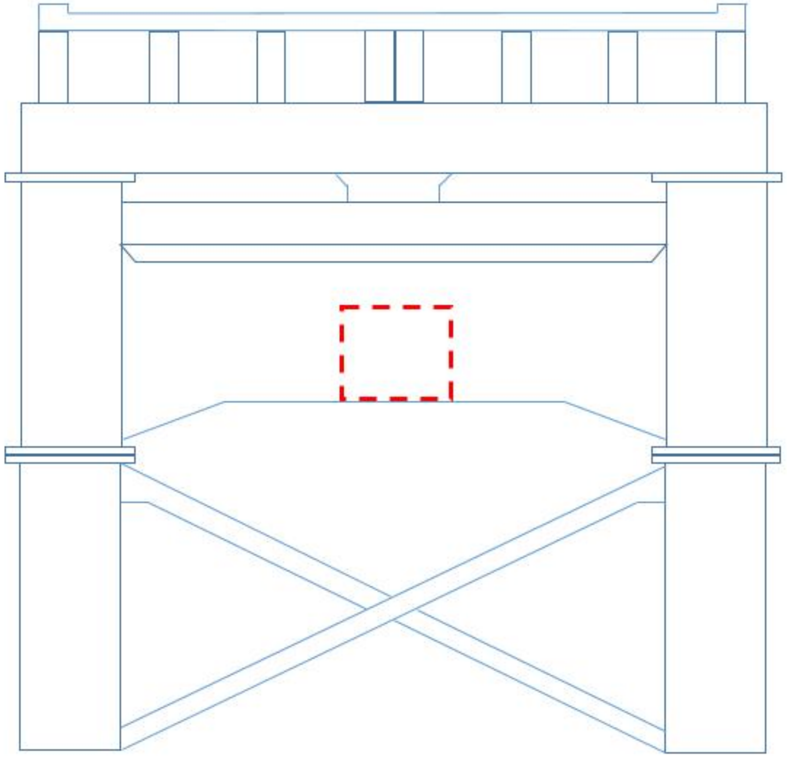
Key

- ● C1 Core Sample
- B1 Concrete Breakout
- Test Area
- R1 Concrete Resistivity
- 80 Calibrated Reinforcement Cover (mm)

Diaphragm Wall (South Face Middle Section)







Typical reo spacing:
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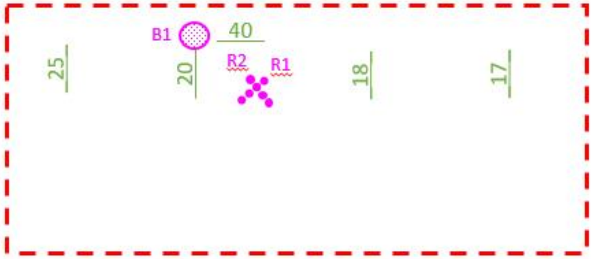
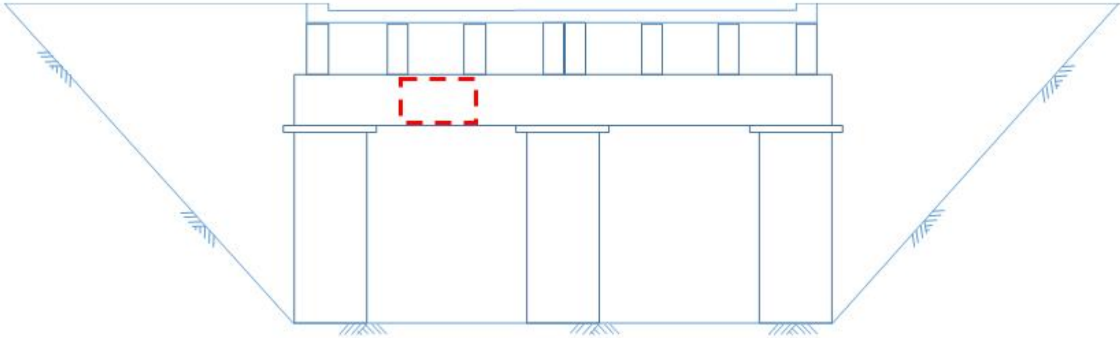
Horizontal = 300 mm

Key

- C1 Core Sample
- B1 Concrete Breakout
- Test Area
- R1 Concrete Resistivity
- 80 Calibrated Reinforcement Cover (mm)

Abutment Headstock (North Face Eastern Side)



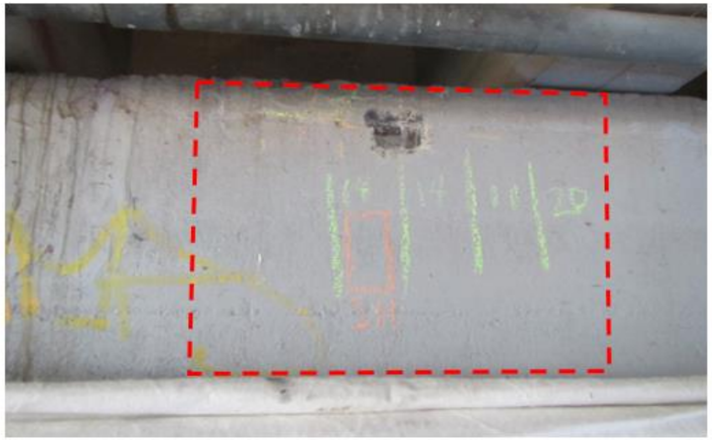


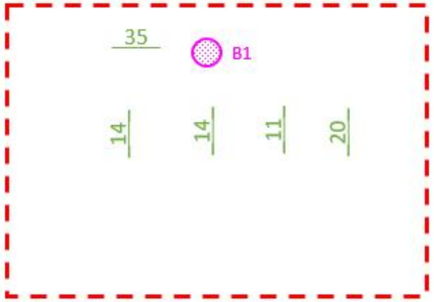
Typical reo spacing:
Vertical = 270 - 360 mm

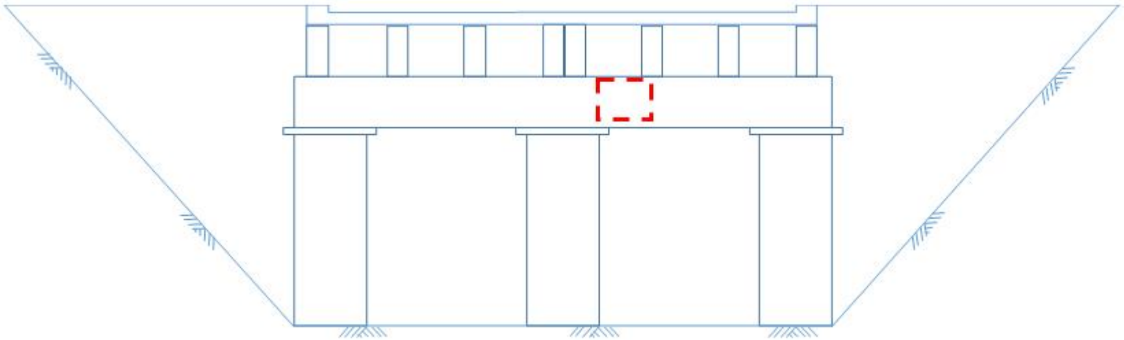
Key

● C1	Core Sample	R1	Concrete Resistivity
B1	Concrete Breakout	80	Calibrated Reinforcement Cover (mm)
	Test Area		

Abutment Headstock (North Face Middle Section)





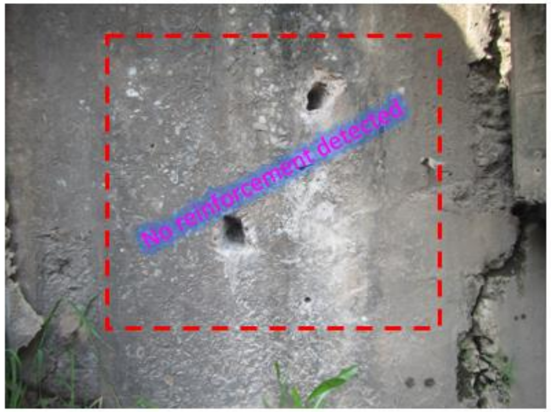


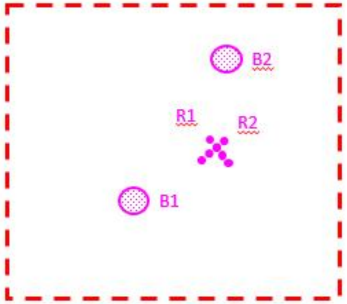
Typical reo spacing:
Vertical = 110 - 120 mm

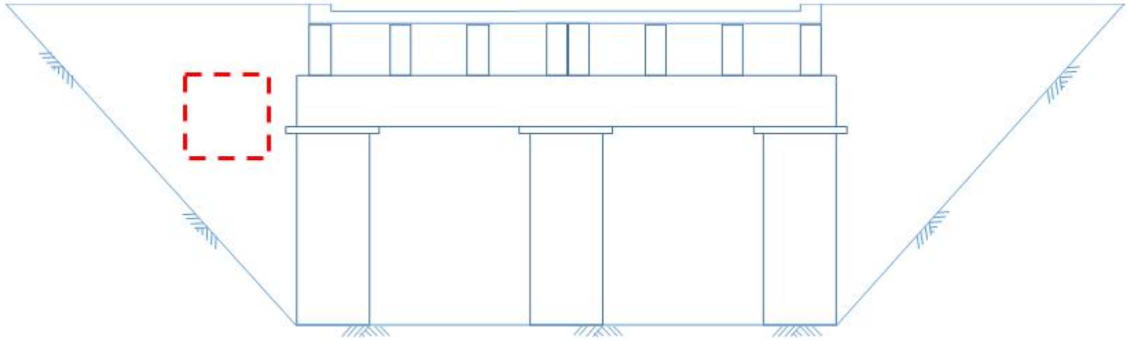
Key

- C1 Core Sample
- B1 Concrete Breakout
- Test Area
- R1 Concrete Resistivity
- 80 Calibrated Reinforcement Cover (mm)

Abutment Wing Wall (East Side)







Key

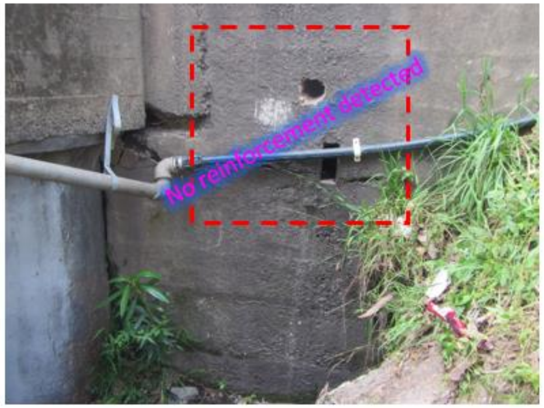
- C1 Core Sample
- B1 Concrete Breakout
- Test Area
- R1 Concrete Resistivity
- 80 Calibrated Reinforcement Cover (mm)

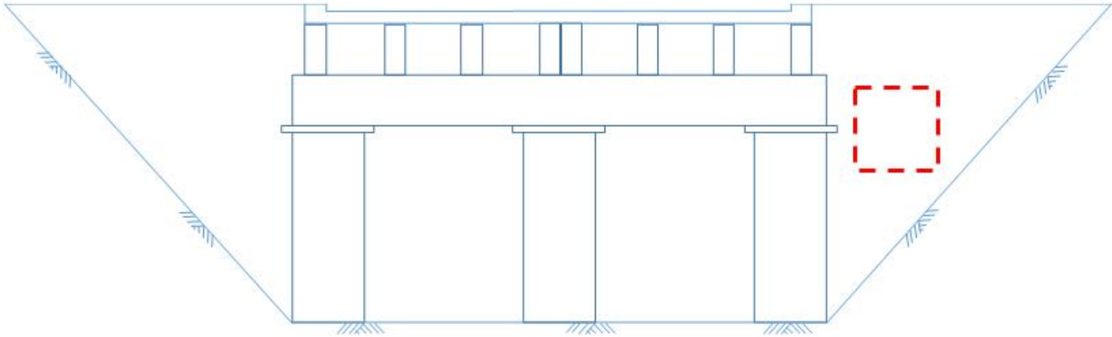
Breakout Photos

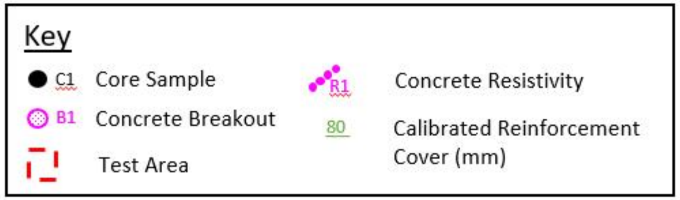
Photo 1 East side Wingwall Breakout B1

Photo 2 East side Wingwall Breakout B2

Abutment Wing Wall (West Side)



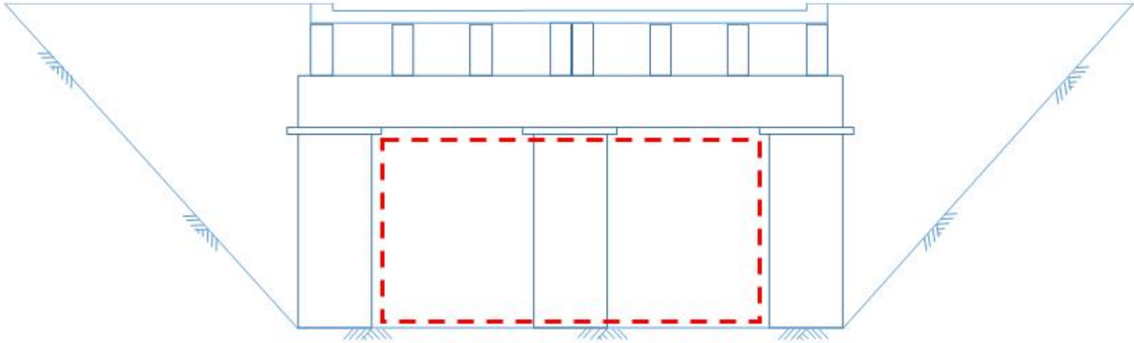




Breakout Photos

Photo 3 West side Wingwall Breakout B1

Abutment Wall



Key

C1	Core Sample	R1	Concrete Resistivity
B1	Concrete Breakout	80	Calibrated Reinforcement Cover (mm)
	Test Area		

Breakout Photos

Photo 4 Abutment Wall Breakout B1

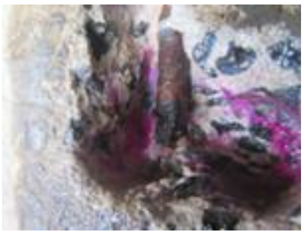
Photo 5 Abutment Wall Breakout B2

Photo 6 Abutment Wall Breakout B3

Covermeter Results Summary (Minimum/Maximum/Mean per Location)

Element	Location	Calibrated Reo Depth (mm)					
		Horizontal			Vertical		
		Max.	Min.	Mean	Max.	Min.	Mean
Pier 1 Headstock	South Face Eastern Side	41	32	37	22	15	20
Pier 1 Headstock	South Face Middle Section	40	35	38	24	19	22
Pier 1 Headstock	South Face Western Side	36	34	35	25	20	23
Girder 1	North End Western Face	36	29	32	26	16	20
Girder 7	North End East Face	46	46	46	21	18	20
Deck Soffit	Area between Girders 6- 7 North End	35	34	35	55	35	46
Deck Soffit	Area between Girders 3 - 4A Middle Section	33	22	26	54	32	41
Diaphragm Wall	South Face Middle Section	47	32	38	19	17	18
Abutment Headstock	North Face Eastern Side	40	40	40	25	17	20
Abutment Headstock	North Face Middle Section	35	35	35	20	11	15

Appendix D – Concrete Breakout Survey Results

Element	Breakout Details & Comment	Photo
Girder 3 North End East Face	- A vertical reinforcement (Ø12 mm) at ≈ 20 mm. Minor surface corrosion on the reinforcements.	
Girder 4A North End West Face	- A horizontal reinforcement (Ø40 mm) found at ≈ 40 mm and a vertical reinforcement (Ø12 mm) at ≈ 20 mm. Minor surface corrosion on the reinforcements.	
Girder 4B North End East Face	- A horizontal reinforcement (Ø40 mm) found at ≈ 32 mm and a vertical reinforcement (Ø12 mm) at ≈ 21 mm. Minor surface corrosion on the reinforcements.	
Girder 5 North End East Face	- A horizontal reinforcement (Ø40 mm) found at ≈ 35 mm and a vertical reinforcement (Ø12 mm) at ≈ 24 mm. Minor surface corrosion on the reinforcements.	
Girder 6 North End East Face	- A vertical reinforcement (Ø12 mm) at ≈ 27 mm. Minor surface corrosion on the reinforcements.	
Girder 7 North End West Face	- A horizontal reinforcement (Ø40 mm) found at ≈ 30 mm. Minor surface corrosion on the reinforcements.	

Element	Breakout Details & Comment	Photo
Abutment Headstock North Face Eastern Side	- A horizontal reinforcement (Ø12 mm) found at ≈ 40 mm and 2 x vertical reinforcement (Ø12 mm) at ≈ 20 mm. Minor surface corrosion on the reinforcements.	
Abutment Headstock North Face Middle Section	- A horizontal reinforcement (Ø12 mm) found at ≈ 35 mm and 2 x vertical reinforcement (Ø12 mm) at ≈ 14 mm. Minor surface corrosion on the reinforcements.	
Abutment Headstock North Face Western Side	- A horizontal reinforcement (Ø12 mm) found at ≈ 30 mm and a vertical reinforcement (Ø12 mm) at ≈ 15 mm. Minor surface corrosion on the reinforcements.	
Pier 1 Headstock South Face Eastern Side	- A horizontal reinforcement (Ø12 mm) found at ≈ 40 mm and 2 x vertical reinforcement (Ø12 mm) at ≈ 15 mm. Minor surface corrosion on the reinforcements.	
Pier 1 Headstock South Face Middle Section	- A vertical girder stud reinforcement (Ø36 mm) found at ≈ 165 mm directly under Girder 3 at the concrete core sample location C2.	
Pier 1 Headstock South Face Western Side	- A horizontal reinforcement (Ø12 mm) found at ≈ 40 mm and 2 x vertical reinforcement (Ø12 mm) at ≈ 20 mm. Minor surface corrosion on the reinforcements.	
Pier 1 Headstock North End Western Side	- A vertical reinforcement (Ø12 mm) at ≈ 20 mm. Minor surface corrosion on the reinforcements.	

Element	Breakout Details & Comment	Photo
Diaphragm Wall South Face Middle Section	- A horizontal reinforcement (Ø12 mm) found at ≈ 35 mm and a vertical reinforcement (Ø12 mm) at ≈ 18 mm. Minor surface corrosion on the reinforcements.	
Diaphragm Wall South Face Western Side	- A vertical reinforcement (Ø12 mm) at ≈ 15 mm. Minor surface corrosion on the reinforcements.	
Diaphragm Wall South Face Eastern Side	- A vertical reinforcement (Ø12 mm) at ≈ 15 mm. Minor surface corrosion on the reinforcements.	
Deck Soffit Girders 6 - 7 North End	- A horizontal reinforcement (Ø12 mm) found at ≈ 35 mm. Minor surface corrosion on the reinforcements.	
Deck Soffit Girders 3 - 4A Middle Section	- 2 x horizontal reinforcement (Ø12 mm) found at ≈ 33 mm. Minor surface corrosion on the reinforcements.	
Deck Soffit Girders 1 - 2 North End	- A horizontal reinforcement (Ø12 mm) found at ≈ 35 mm. Minor surface corrosion on the reinforcements.	

Appendix E – Reinforcement Continuity Test Results

Electrical continuity test within elements

Connection		Resistance (Ω)				Steel Continuity
From	To	Lead Resistance	Forward	Reverse	DC (mV) *	
Girders						
Girder 1 North End Horizontal Reo	Girder 1 South End Horizontal Reo	0.2	2.5	2.5	0.1	Yes
Girder 1 North End B2 Horizontal Reo	Girder 1 North End B1 Vertical Reo	0.2	16.2	16.1	0.4	Yes
Girder 2 North End Horizontal Reo	Girder 2 South End Horizontal Reo	0.2	1.2	1.1	0	Yes
Girder 2 North End B1 Vertical Reo	Girder 2 North End B1 Horizontal Reo	0.2	6	7.3	0.1	Yes
Girder 3 North End Vertical Reo	Girder 3 South End Horizontal Reo	0.2	4.5	4.9	0	Yes
Girder 3 North End B1 Horizontal Reo	Girder 3 North End B2 horizontal Reo	0.2	8.5	8.6	0	Yes
Girder 4A North End Horizontal Reo	Girder 4A South End Horizontal Reo	0.2	0.3	0.4	0	Yes
Girder 4B North End Horizontal Reo	Girder 4B South End Vertical Reo	0.2	1.7	1.8	0	Yes
Girder 4B North End B2 Vertical Reo	Girder 4B North End B2 Horizontal Reo	0.2	1.9	1.8	0	Yes
Girder 4B North End B1 Vertical Reo	Girder 4B North End B2 Vertical Reo	0.2	0.7	1.1	0.1	Yes
Girder 4A North End B2 Vertical Reo	Girder 4A North End B2 Horizontal Reo	0.2	4.4	4	0	Yes
Girder 5 North End Horizontal Reo	Girder 5 South End Horizontal Reo	0.2	1.3	1.4	0	Yes
Girder 5 North End B1 Vertical Reo	Girder 5 North End B2 Horizontal Reo	0.2	0.6	0.5	0	Yes
Girder 6 North End Vertical Reo	Girder 6 South End Horizontal Reo	0.2	1.3	1.5	-0.1	Yes
Girder 6 North End B1 Vertical Reo	Girder 6 North End B2 Vertical Reo	0.2	1.7	1.8	0	Yes
Girder 7 North End Horizontal Reo	Girder 7 South End Horizontal Reo	0.2	0.4	0.5	0	Yes

Connection		Resistance (Ω)				Steel Continuity
From	To	Lead Resistance	Forward	Reverse	DC (mV) *	
Abutment Headstock						
Abutment Headstock East Side Horizontal Reo	Abutment Headstock West Side Vertical Reo	0.2	169K	-168K	-168	No ++
Abutment Headstock East Side Horizontal Reo	Abutment Headstock West Side Horizontal Reo	0.2	-167K	168K	167	No ++
Abutment Headstock West Side Vertical Reo	Abutment Headstock West Side Horizontal Reo	0.2	41.9	31	-1.7	Maybe
Pier 1 Headstock						
Pier 1 Headstock East Side Vertical Reo	Pier 1 Headstock West Side Horizontal Reo	0.2	109	110	109	No ++
Pier 1 Headstock East Side Vertical Reo	Pier 1 Headstock West Side Vertical Reo	0.2	55.7	700	-44	No ++
Pier 1 Headstock East Side Horizontal Reo	Pier 1 Headstock West Side Horizontal Reo	0.2	-10K	109K	107	No ++
Pier 1 Headstock East Side Horizontal Reo	Pier 1 Headstock East Side Vertical Reo	0.2	27	22	-1	Yes
Pier 1 Headstock B1 Vertical Reo	Pier 1 Headstock B1 Horizontal Reo	0.2	7.6	7.3	0.4	Yes
Pier 1 Diaphragm Wall						
Pier 1 Diaphragm Wall Centre Horizontal Reo	Pier 1 Diaphragm Wall Centre Vertical Reo	0.2	8.7	7	0.3	Yes
Pier 1 Diaphragm Wall Centre Horizontal Reo	Pier 1 Diaphragm Wall West Side Vertical Reo	0.2	6.7	6.4	0	Yes
Deck Soffit						
Deck Soffit G1-G2 Transverse Reo	Deck Soffit G3-G4A Transverse Reo	0.2	56	53	-0.6	Yes
Deck Soffit G6-G7 Transverse Reo	Deck Soffit G3-G4A Transverse Reo	0.2	152	149	1.3	Maybe

* <1mV is an indication of electrical continuity presence, as per AS2832.5 Section 5.2

++ The headstocks were seem to have constructed in two halves, with reinforcement in one half did not continue to the other half. Therefore, this electrical discontinuity tests may be considered as between two elements.

Electrical continuity test between elements

Connection		Resistance (Ω)				Steel Continuity
From	To	Lead Resistance	Forward	Reverse	DC (mV)	
Girder 2 North End Horizontal Reo	Girder 1 South End Horizontal Reo	0.2	3.1	4	0.5	Yes
Girder 2 North End Horizontal Reo	Girder 3 South End Horizontal Reo	0.2	5	5.1	0	Yes
Girder 4 North End Horizontal Reo	Girder 3 South End Horizontal Reo	0.2	5	3.9	-0.3	Yes
Girder 4A North End Horizontal Reo	Girder 4B South End Vertical Reo	0.2	3.8	4	0.2	Yes
Girder 4A North End Horizontal Reo	Girder 5A South End Horizontal Reo	0.2	5	5.7	1.3	May be
Girder 6 North End Vertical Reo	Girder 5A South End Horizontal Reo	0.2	5.3	7.7	1.4	May be
Girder 7 North End Horizontal Reo	Girder 5A South End Horizontal Reo	0.2	2.3	2.7	0.3	Yes
Girder 7 North End Horizontal Reo	Girder 6 South End Horizontal Reo	0.2	7	4.1	-1.2	Maybe
Abutment Headstock Vertical Reo	Pier 1 Headstock Vertical Reo	0.2	OL	OL	-	No
Abutment Headstock Vertical Reo	Girder 7 South End Horizontal Reo	0.2	96.4	29.9	-29	No
Deck Soffit G3-G4A Transverse Reo	Girder 4A North End Horizontal Reo	0.2	55	49	-0.9	Yes
Pier 1 Headstock East Side Vertical Reo	Pier 1 Diaphragm Wall East Side Vertical	0.2	-147K	141K	142	No
Pier 1 Diaphragm Wall Centre Horizontal Reo	Pier 1 Headstock West Side Vertical Reo	0.2	165	85	-35	No
Span 2 Girder 2 South Side Horizontal	Span 1 Girder 2 North Side Horizontal Reo	0.2	6	9.2	1.8	Maybe
Girder 6 North End B1 Vertical Reo	Girder 7 North End B1 Horizontal Reo	0.2	7.7	7.6	18	No
Girder 5 North End B2 Horizontal Reo	Girder 6 North End B1 Vertical Reo	0.2	6.6	4	-1	Yes
Girder 4A North End B1 Vertical Reo	Girder 4B North End B2 Vertical Reo	0.2	4.4	4.3	0	Yes
Girder 4A North End B2 Vertical Reo	Girder 4B North End B2 Vertical Reo	0.2	8.5	8	0.3	Yes
Girder 1 North End B2 Horizontal Reo	Girder 2 North End B1 Vertical Reo	0.2	4.7	6	0.8	Yes
Girder 3 North End B2 Vertical Reo	Girder 2 North End B1 Vertical Reo	0.2	7.1	7.8	0.2	Yes
Pier 1 Headstock B1 Vertical Reo	Girder 2 North End B1 Vertical Reo	0.2	108K	-110k	108	No

Appendix F – Resistivity Test Results

Element	Location	Reading (Ω)	Ω.cm	Corrosion Rate *
Pier 1 Headstock	West Side R1	4800	150720	Low
	West Side R2	7800	244920	Low
	Centre R1	4300	135020	Low
	Centre R2	3700	116180	Low
Girder 1	North End West Face R1	6100	191540	Low
	North End West Face R2	7600	238640	Low
Girder 5	North End East Face R1	11500	361100	Low
	North End East Face R2	9200	288880	Low
Deck Soffit	Girders 6 - 7 on North End	11500	361100	Low
		17500	549500	Low
Pier 1 Diaphragm Wall	South face Centre R1	1650	51810	Moderate
	South face Centre R2	2300	72220	Moderate
Abutment Headstock	Mid section R1	4950	155430	Low
	Mid section R2	4350	136590	Low
Wing Wall	East Side R1	1800	56520	Moderate
	East Side R2	1600	50240	Moderate
Abutment Wall	East Side	9400	295160	Low
	West Side	9700	304580	Low

* Likely corrosion rate sustainable by concrete

Appendix G – Carbonation Test Results

Location	Test Area	Type	Carbonation depth (mm)
Abutment Wall	Wall East Face	Breakout	70-80
Abutment Wall	Wall West Face	Breakout	70
Abutment Wall	Wall West Face	Breakout	70
Abutment Headstock	A1.1 Abutment East	Breakout	20
Abutment Headstock	A1.2 Abutment Middle	Breakout	30-35
Abutment Headstock	A1.3 Abutment End	Breakout	30
East Wingwall	Wingwall B1	Breakout	70
East Wingwall	Wingwall B2	Breakout	70
West Wingwall	Wingwall B1	Breakout	>80
Headstock 1	DTA 6.1	Core 1	50
Headstock 1	DTA 6.1	Core 2	50
Headstock 1	DTA 6.3	Core 3	30
Diaphragm	A7.1 West Side	Breakout	25
Diaphragm	A7.2 Mid Section	Breakout	40
Diaphragm	A7.3 East End	Breakout	15
Girders	DTA 10.1	Core 1	60
Girders	DTA 10.1	Core 2	40
Girders	DTA 10.2	Core 3	30
Girders	G3 North End East Face	Breakout	25
Girders	G4a North End East Face	Breakout	20
Girders	G4b North End East Face	Breakout	35
Girders	G5 North End East Face	Breakout	40
Girders	G6 North End East Face	Breakout	40
Girders	G7 North End West Face	Breakout	20-25
Deck soffit	A12 G1-G2	Breakout	25
Deck soffit	A12 G3-G4a	Breakout	30
Deck soffit	A12 G6-G7	Breakout	40

Appendix H – Surface Hardness Test Results

Colour Key:	MIN	MAX

Element	REF.	1	2	3	4	5	6	Angle / Remarks	Max.	Min.	Mean	Estimated Cylinder Compressive Strength (MPa)
Pier 1 Headstock	West Side - Location 1 (At Core 2 Location)	30	30	37	30	27	25	0°	37	24	29	29
		7	8	9	10	11	12					
		30	24	31	31	30	25					
	REF.	1	2	3	4	5	6	Angle / Remarks	Max.	Min.	Mean	Estimated Cylinder Compressive Strength (MPa)
	West Side - Location 2 (At Core 1 Location)	30	30	30	30	33	32	0°	34	28	31	30
		7	8	9	10	11	12					
		34	30	28	28	32	28					
	REF.	1	2	3	4	5	6	Angle/Remarks	Max.	Min.	Mean	Estimated Cylinder Compressive Strength (MPa)
	Centre - Location 1	28	35	40	30	34	36	0°	40	27	33	31
		7	8	9	10	11	12					
		32	36	38	27	31	37					
	REF.	1	2	3	4	5	6	Angle/Remarks	Max.	Min.	Mean	Estimated Cylinder Compressive Strength (MPa)
	East Side - Location 1	32	34	30	21	26	34	0°	41	21	32	30
		7	8	9	10	11	12					
		30	34	32	36	32	41					
	REF.	1	2	3	4	5	6	Angle/Remarks	Max.	Min.	Mean	Estimated Cylinder Compressive Strength (MPa)
	East Side - Location 2 (At Core 1 Location)	42	44	42	42	38	39	0°	45	36	40	33
		7	8	9	10	11	12					
		43	38	36	45	40	37					
Girder 1	REF.	1	2	3	4	5	6	Angle/Remarks	Max.	Min.	Mean	Estimated Cylinder Compressive Strength (MPa)
	North End West Face (At Core 1 Location)	34	35	30	30	35	30	0°	36	24	32	30
		7	8	9	10	11	12					
		24	30	36	35	34	30					
	REF.	1	2	3	4	5	6	Angle/Remarks	Max.	Min.	Mean	Estimated Cylinder Compressive Strength (MPa)
	North End West Face (At Core 2 Location)	28	26	28	30	31	36	0°	40	26	35	31
		7	8	9	10	11	12					
		35	30	39	38	39	40					
Girder 7	REF.	1	2	3	4	5	6	Angle/Remarks	Max.	Min.	Mean	Estimated Cylinder Compressive Strength (MPa)
	North End West Face	34	32	34	34	36	41	0°	41	32	38	33
		7	8	9	10	11	12					
		40	38	40	36	38	40					

Member	REF.	1	2	3	4	5	6	Angle/Remarks	Max.	Min.	Mean	Estimated Cylinder Compressive Strength (MPa)
Girder 6	North End East Face	36	42	38	36	36	39	0°	42	30	36	32
		7	8	9	10	11	12					
		30	38	36	35	38	32					
Girder 5	REF.	1	2	3	4	5	6	Angle/Remarks	Max.	Min.	Mean	Estimated Cylinder Compressive Strength (MPa)
	North End East Face	32	36	34	30	34	32	0°	36	28	32	30
		7	8	9	10	11	12					
		32	34	31	30	33	28					
Girder 4B	REF.	1	2	3	4	5	6	Angle/Remarks	Max.	Min.	Mean	Estimated Cylinder Compressive Strength (MPa)
	North End East Face	39	36	40	38	37	44	0°	44	34	38	33
		7	8	9	10	11	12					
		39	39	35	36	34	38					
Girder 4A	REF.	1	2	3	4	5	6	Angle/Remarks	Max.	Min.	Mean	Estimated Cylinder Compressive Strength (MPa)
	North End West Face	32	32	35	33	30	33	0°	35	29	32	30
		7	8	9	10	11	12					
		32	32	30	29	35	34					
Girder 3	REF.	1	2	3	4	5	6	Angle/Remarks	Max.	Min.	Mean	Estimated Cylinder Compressive Strength (MPa)
	North End East Face	38	38	38	38	30	37	0°	44	30	39	33
		7	8	9	10	11	12					
		41	40	40	39	44	39					
Girder 7	REF.	1	2	3	4	5	6	Angle/Remarks	Max.	Min.	Mean	Estimated Cylinder Compressive Strength (MPa)
	North End East Face (At Core 3 Location)	33	28	31	29	35	30	0°	38	25	31	30
		7	8	9	10	11	12					
		28	25	28	32	38	36					
Deck Soffit	REF.	1	2	3	4	5	6	Angle/Remarks	Max.	Min.	Mean	Estimated Cylinder Compressive Strength (MPa)
	Girders 6-7 North End	32	34	39	35	36	36	90°	40	32	36	32
		7	8	9	10	11	12					
		38	32	40	33	39	34					
	REF.	1	2	3	4	5	6	Angle/Remarks	Max.	Min.	Mean	Estimated Cylinder Compressive Strength (MPa)
	Girders 3-4A North End	47	48	44	41	45	43	90°	48	34	41	34
		7	8	9	10	11	12					
		41	42	36	35	42	34					

Strength estimated is based on the correlation established, refer Appendix J.

	REF.	1	2	3	4	5	6	Angle/Remarks	Max.	Min.	Mean	Estimated Cylinder Compressive Strength (MPa)
Pier 1 Diaphragm Wall	Mid Section	36	42	36	22	40	41	0°	42	22	34	31
		7	8	9	10	11	12					
		36	32	40	37	33	26					
	REF.	1	2	3	4	5	6	Angle/Remarks	Max.	Min.	Mean	Estimated Cylinder Compressive Strength (MPa)
	East Side	40	40	46	39	35	39	0°	49	35	40	34
		7	8	9	10	11	12					
		37	49	40	39	44	40					
Abutment Headstock	REF.	1	2	3	4	5	6	Angle/Remarks	Max.	Min.	Mean	Estimated Cylinder Compressive Strength (MPa)
	East Side	28	28	32	34	28	27	0°	34	26	29	29
		7	8	9	10	11	12					
		28	32	30	32	26	29					
	REF.	1	2	3	4	5	6	Angle/Remarks	Max.	Min.	Mean	Estimated Cylinder Compressive Strength (MPa)
	Mid Section	32	28	32	32	29	30	0°	36	28	32	30
		7	8	9	10	11	12					
		30	36	34	32	31	33					
Wing Wall	REF.	1	2	3	4	5	6	Angle/Remarks	Max.	Min.	Mean	Estimated Cylinder Compressive Strength (MPa)
	East Side	28	26	22	25	24	23	0°	28	18	24	27
		7	8	9	10	11	12					
		26	26	22	22	26	18					
Wing Wall	REF.	1	2	3	4	5	6	Angle/Remarks	Max.	Min.	Mean	Estimated Cylinder Compressive Strength (MPa)
	West Side	24	22	25	30	24	20	0°	30	20	24	27
		7	8	9	10	11	12					
		26	23	28	22	22	24					
Abutment wall	REF.	1	2	3	4	5	6	Angle/Remarks	Max.	Min.	Mean	Estimated Cylinder Compressive Strength (MPa)
	East Side	25	24	28	24	30	31	0°	31	23	27	28
		7	8	9	10	11	12					
		26	28	24	23	31	29					
Abutment wall	REF.	1	2	3	4	5	6	Angle/Remarks	Max.	Min.	Mean	Estimated Cylinder Compressive Strength (MPa)
	West Side	27	26	26	20	28	23	0°	30	20	26	28
		7	8	9	10	11	12					
		25	26	26	30	29	26					

Appendix I – Concrete Compressive Strength Test Results

Sydney Laboratory
Unit 5 / 43 Herbert St
Artarmon NSW 2064
email: artarmon@ghd.com.au
web: ghd.com.au/ghdgeotechnics
Tel: (02) 9462 4860
Fax: (02) 9462 4710

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GHD GEOTECHNICS

Compressive Strength of Concrete Specimens - Report

Report No: SYD
Issue No: 1

Client:	Roads & Maritime Services
Project:	Windsor Bridge Viewing Platform Rehabilitation Design
Location:	Windsor, NSW
Job No.:	12510815

Authorised signatory: D. Brooke
Date of Issue: 27/03/2018

Sample Details

Test Method: AS1012.9 and AS1012.14
Storage History: Tested as received

GHD Sample ID:	SYD19-0424-01	SYD19-0424-02	SYD19-0424-03		
Client Sample ID:	6.1/C1	6.1/C2	6.3/C1		
Borehole No.:	-	-	-		
Depth (m):	-	-	-		
Date Sampled:	not known	not known	not known		
Date Tested:	2/10/2019	2/10/2019	2/10/2019		
Sample Description:	Concrete Core	Concrete Core	Concrete Core		

Test Results

Sample Height (mm):	159.4	159.8	161.2		
Sample Diameter (mm):	80.8	80.8	80.8		
Sample Height/Diameter Ratio:	2.0	2.0	2.0		
Sample Bulk Density (kg/m3):	2319	2383	2414		
Moisture Content (%):	-	-	-		
Force Applied (kN):	103.6	145.1	241.0		
Uniaxial compressive strength (MPa):	20.2	28.3	47.0		
Corrected UCS (MPa):	**20.2**	**28.3**	**47.0**		
Correction Factor as per AS1012.14	1	1	1		
Mode of Failure:	Tensile	Tensile	Tensile		
Rejection Criteria					
Preconditioning	none	none	none		
Reinforcement	none	none	none		

Rejection Criteria:

Note 1
Note 2
Note 3
Note 4
Note 5

Testing machine Wykeham Farrance - 2000 kN

Sydney Laboratory
Unit 5 / 43 Herbert St
Artarmon NSW 2064
email: artarmon@ghd.com.au
web: ghd.com.au/ghdgeotechnics
Tel: (02) 9462 4860
Fax: (02) 9462 4710

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GHD GEOTECHNICS

Compressive Strength of Concrete Specimens - Report

Report No: SYD02350.1
Issue No: 1

Client:	Roads & Maritime Services
Project:	Windsor Bridge Viewing Platform Rehabilitation Design.
Location:	Windsor, NSW
Job No.:	12510815

Authorised signatory: D. Brooke
Date of Issue: 3/10/2019

Sample Details

Test Method: AS1012.9 and AS1012.14
Storage History: Tested as received

GHD Sample ID:	SYD19-0424-04	SYD19-0424-05	SYD19-0424-06		
Client Sample ID:	10.1/C1	10.1/C2	10.2/C1		
Borehole No.:	-	-	-		
Depth (m):	-	-	-		
Date Sampled:	25/09/2019	26/09/2019	27/09/2019		
Date Tested:	2/10/2019	2/10/2019	2/10/2019		
Sample Description:	Concrete Core	Concrete Core	Concrete Core		

Test Results

Sample Height (mm):	141.5	161.6	159.5		
Sample Diameter (mm):	80.7	80.8	80.9		
Sample Height/Diameter Ratio:	1.75	2.00	1.97		
Sample Bulk Density (kg/m3):	2352	2320	2418		
Moisture Content (%):	-	-	-		
Force Applied (kN):	142.2	205.7	223.2		
Uniaxial compressive strength (MPa):	27.8	40.1	43.4		
Corrected UCS (MPa):	**27.2**	**40.1**	**43.4**		
Correction Factor as per AS1012.14	0.98	1	1		
Mode of Failure:	Tensile	Tensile	Tensile		
Rejection Criteria					
Preconditioning	none	none	none		
Reinforcement	none	none	none		

Rejection Criteria:

Note 1
Note 2
Note 3
Note 4
Note 5

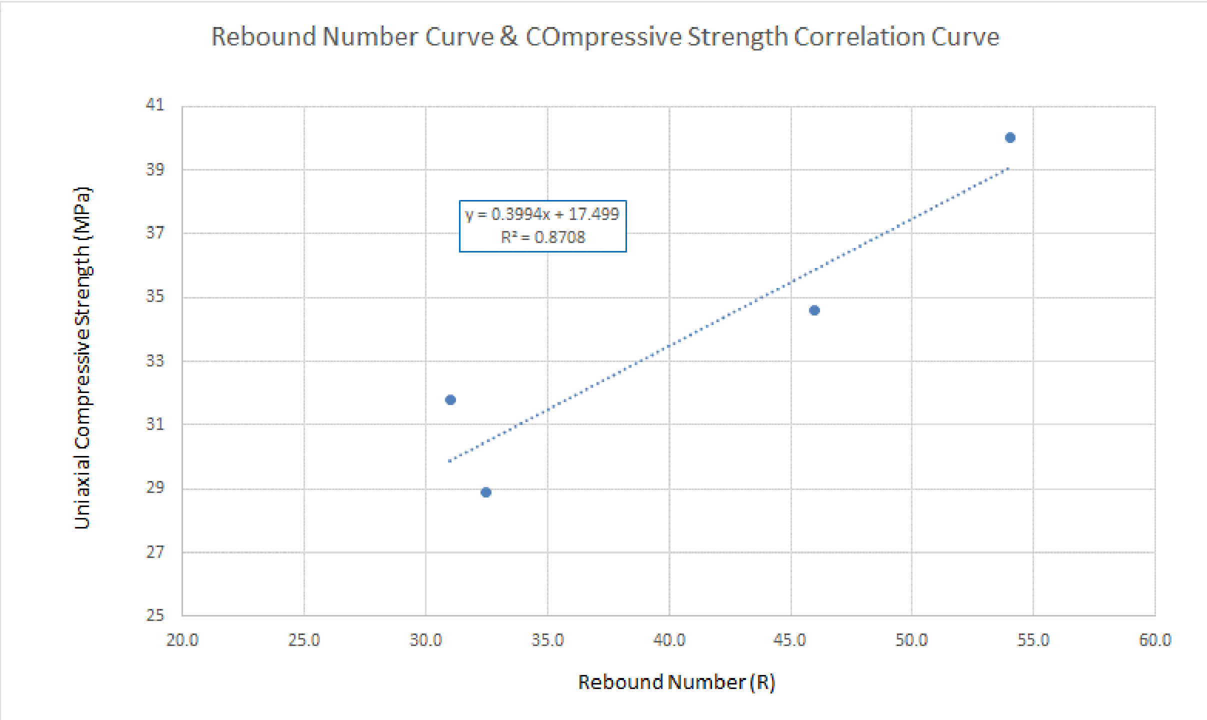
Testing machine Wykeham Farrance - 2000 kN

Appendix J – Rebound Number (R) and Compressive Strength Correlation Curve

Rebound Number and uniaxial compressive strength test results are presented below:

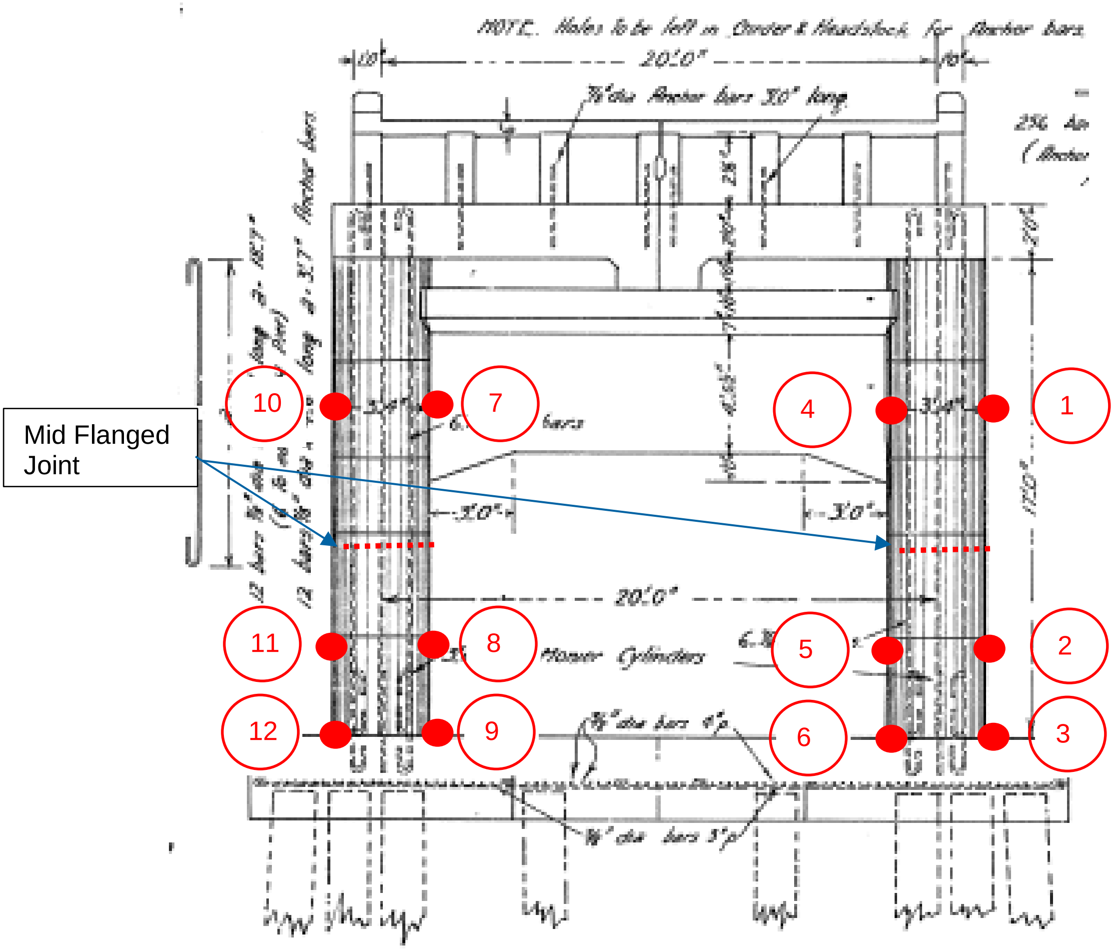
Location	Mean Rebound Number	Uniaxial Compressive strength Test (MPa)
Pier 1 Headstock Core 2	29	32.5
Pier 1 Headstock Core 1	31	23.0
Pier 1 Headstock Core 3	40	54.0
Girder 1 North End West Face Core 1	32	31.0
Girder 1 North End West Face Core 2	35	46.0
Girder 7 North End East Face Core 3	31	50.0

Note: The red coloured outliners omitted in the calculation



Note: Although we were looking for r^2 value >0.9 for correlation, a value of 0.87 is still considered to be an indication of a sensible correlation presence.

Appendix K – Pier Wall Thickness Survey Results



Pier	Location ID	Side	Location	Thickness (mm)			
				1	2	3	Mean
Pier 1 East	1	East	Top	-	-	-	-
	2	East	Middle	30	28	-	29
	3	East	Bottom	25.1	-	-	25
	4	West	Top	-	-	-	-
	5	West	Middle	30	32	-	31
	6	West	Bottom	24	25	24.5	25
Pier 1 West	7	East	Top	-	-	-	-
	8	East	Middle	25	26	24	25
	9	East	Bottom	22	22	23	22
	10	West	Top	-	-	-	-
	11	West	Middle	24	22	26	24
	12	West	Bottom	26	26	23	25

Appendix L – Structural Analysis Calculations

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CALCULATIONS

Client
Project Windsor Bridge Rehabilitation
Subject Veiwing platform

Job no. 21–12510815
Calcs by SH
Checked by

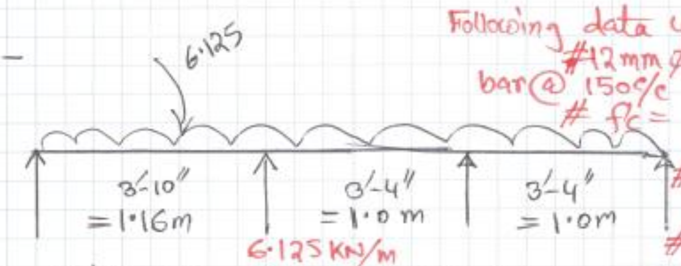
Sheet 1 of
Date 22/10/19
Date

The bridge was designed for heavy traffic loading.

Due to conversion of existing Road to a Viewing platform L.L on the structure will be significantly reduced.

Maximum L.L on the structure will be only around 5kpa due to conversion of viewing platform.

Deck slab :–



Following data used
12mm ø main bar @ 150c/c
f'c = 25 MPa
fy = 230 MPa
Cover = 35 mm

Deck slab thickness = 245mm (Site measurement)

D.L = 0·245×25 = 6·125 kpa
L.L = 5kpa
Factored load = 1·2DL+1·5LL
= 1·2×6·125+1·5×5
= 14·85 KN/m

$M^* = \frac{14{\cdot}85\times(1{\cdot}16)^2}{9}$
= 2·3 KN–m

Considering t = 245mm (cover = 35mm)
d = 245 – 35 – 6 = 204 mm

Bar size = 12mm ø

Running spreadsheet program using

12mm ø bar @ 150 c/c

use f'c = 25MPa
fy = 230MPa

GHD

CALCULATIONS

Client		Job no.	21-12510815	Sheet	2 of
Project	Windsor Bridge Rehabilitation	Calcs by	SH	Date	22/10/19
Subject	Vekoing platform	Checked by		Date	

$\phi M_u = 0.85 \times 33.72 = 28.6$ kN-m/m $> M^*$

$\therefore$ Slab O.K

<u>Girder</u>

Span of girder = 13m

Girder width = 1′ = 300 mm

Girder thickness = 2′-6″ + 245 mm

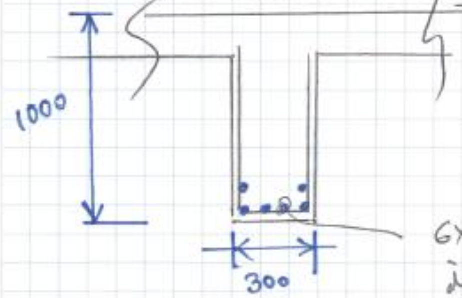
= 1000 mm

6.125
+ 1.2x7.5
= 15.125 kN/m

D.L = 0.3x1x25
= 7.5 kN/m

Load from deck = 6.125 kN/m

$$M^*_{+ve} = \frac{15.125 \times (13)^2}{8} = 319 \text{ kN-m}$$



t = 1000 mm

d = 1000 − 35 − 12 − 32 = 921 mm

Say tie size = 12mm

using f'_c = 25MPa

f_y = 230MPa

$A_{s\,req}$ = 2000 mm² ie 3 x 32mm ϕ bar

A_s prov = 6x 32 mm ϕ bar $\therefore$ O.K

GHD

CALCULATIONS

Client		Job no.	21-12510815	Sheet	3 of
Project	Windsor Bridge Rehabilitation	Calcs by	SH	Date	22/10/19
Subject	Viewing Platform	Checked by		Date	

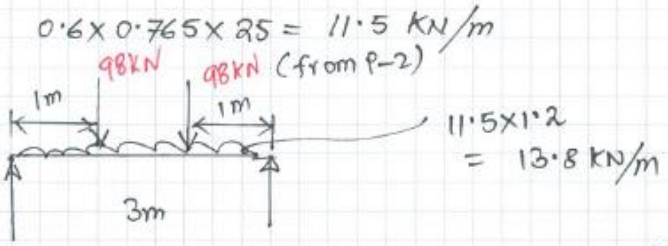
Headstock :-

span = 10' = 3m

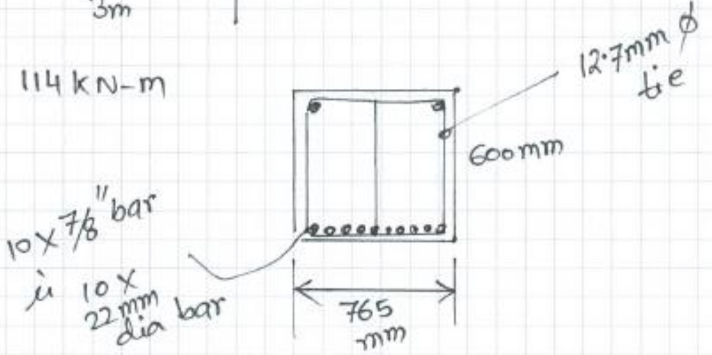
height = 2' = 600mm

width = 765 mm

D.L = 0·6 × 0·765 × 25 = 11·5 kN/m



$M^* = 114$ kN-m



t = 600 mm , Cover = 35mm

d = 600 − 35 − 12 − 11 = 542mm

Running spread sheet program

$As_{req} = 1350\ mm^2$ i.e. 4 × 22mm ø bar

As provided = 10 × 22 mm ø bar

∴ O·K

CALCULATIONS

Client	Job no. 21-12510815	Sheet 4 of
Project Windsor Bridge Rehabilitation	Calcs by SH	Date 22/10/19
Subject Viewing platform	Checked by	Date

Steel column :-

Buckling Analysis of ϕ1000 dia 10 mm Thick Column:

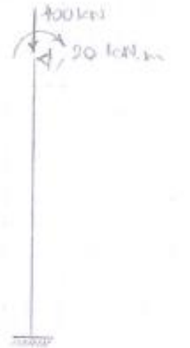
Load Case : $1.2\ G + 1.5\ Q$

Factored Live Load = 400 kN Compression

Assumed minimum eccentricity :

5% of the greatest dimension of the Section = $0.05 \times D = 50$ mm

Therefore, $M^* = 400 \times 0.05 = 20$ kN.m



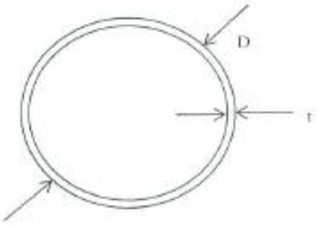
Running Buckling Analysis Mode

$N^* = 400$ kN

$M^* = 20$ kN-m

P-5

		Project No:
		Doc. No:
Steel Column Check	Revision:	Rev. Date:
Ref Discipline: Structural	B	Page No: X OF Y



Section Name	1000 x 10	
Diameter (D)	1000	mm
Axial Wall (t)	10	mm
Bending Wall (t)	10	mm
Steel Grade (fy)	250	MPa
Residual Stress	Hot Roll/Finish	
Fabrication Type	Hot Formed	
λsp	50	
λsy	120	
λs	100	
Compactness	NON-COMPACT	
λe	100	
λey	82	
Ixx	3.810E+09	mm4
Radius Gyration (r)	350.0178567	mm
de	905.5385138	mm
Area losses	0	mm2
Column Height	5000	mm
Support Condition	Pin-Fixed	
Support Type	Braced	
Ke	0.85	
Ag	31101.76727	mm2
An	31101.76727	mm2
Ae	28134.17216	mm2
Kf	0.904584357	
Le	4250	mm
Le/r	12.14223766	
λn	11.54843849	
αa	-2.042323	
αb	-0.5	
λ	12.56959999	
η	0	
ξ	26.1337479	
αc	1	
φNs	6330.188736	kN
φMs	1854.967725	kN.m
(N*/φNs) + (M*/φMsx)	0.073971128	[illegible]
φNc	6330.188736	
φMi	1737.753671	kN.m
N*	400	kN
M*	20	kN.m
N*<φNc	[illegible]	
M*<φMi	[illegible]	
V	2799.159054	kN

GHD

Level 15
133 Castlereagh Street
T:+61 2 9239 7100   F:61 2 9239 7199   E: sydmail@ghd.com

Document Status

Revision	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
A	O. Britt / S. Hasan / N. Endo	M. Ali		M. Ali		25.10.2019
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