Addendum 2

December 10, 2020

Request for Proposal R20-16 Fifth Street Bridge Upgrades – Addendum No. 2

This addendum forms part of the RFP document and shall be read, interpreted, and coordinated with all other parts. The costs of all work contained herein shall be included in the submission. The following revisions supersede the information contained in the original documents to the extent referenced and shall become part thereof.

Item No. 1 – Questions and Answers

Questions 1 thru 7 Cathodic Protection Items 26.2 – 26.10

1. How much can be changed with regards to the specifications? For example, we can probably reduce the number of data loggers from 6 to 2 through the use of multiplexers on the data loggers, this would likely alter the monitor wiring as well. Please confirm.

   Reducing the data loggers from 6 to 2 is acceptable. The drawings provided are a guide and it is anticipated that the contractor will propose modifications to the proposed design and zoning.

2. There are specifications around the requirement for a Master Electrical Contractor, please confirm that this only applies for the AC power to the rectifier. The rectifier and everything on the DC side should not need this requirement.

   The Master Electrical Contractor requirements shall be applied to all AC power sources only.

3. Reference electrodes typically last closer to 30 years. Can the specifications have the duration reduced from 50 years to 30?

   This is acceptable.

4. The specifications require the deck to be soaked for 12 hours prior to completing the half-cell potential survey. Please confirm if the deck must remain soaked for the entire 12 hours prior to performing the test as this will likely be very difficult to achieve based on the time of year and evaporation? Can we just hose it down 12 hours before and leave it?

   The deck can be hosed down 12 hours before. Additional measures such as application of wet burlap on the surface can be used to keep the deck moist. It is anticipated that water would need to be applied to the deck several times during the 12-hour period.

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5. Item 3.9 in the CP specifications references Holiday detection. I've never heard of this before in concrete. Please provide clarification on what is required with this?

   A high voltage holiday detector shall be connected to the reinforcing steel and set to 1KV. The probe head shall be moved over the concrete surface. A 'jeep' will be heard when a short is detected between the probe head and metallic objects near the concrete surface such as reinforcing steel or tie wires. This will allow for detection of potential short circuits to the anode and will identify areas for remedial work prior to installation of the anodes.

6. The Contractor shall be responsible for maintenance and repair to system not covered under the warrantee which is required due to all factors besides vandalism and acts of god. This sounds like during the 5 year warranty period if there is any repairs required to be done to the system, the warranty starts over. Does this mean that the warranty starts over for items that have been repaired or for the whole system, which in our opinion is not a reasonable requirement?

   The warrantee period and the maintenance period should be considered separate. Any repairs or requirements of the warrantee will not cause a restart of the warrantee period for the project as a whole, only the element that has been repaired or replaced.

7. The specifications do not mention the use of modems. Does this mean the data loggers are to communicate externally?

   Modems shall be supplied for the monitoring system capable of transmitting data over cell network. The modems shall be activated and operational at the time of commissioning. The contract for the cell phone data plan shall run for a period of 6 months from substantial completion. It is however anticipated that the City will use manual downloading of the data at the data loggers. Access for manual downloading of the data logger shall be provided.

Additional Questions

8. SS 09 10 10 Section 1.1.1 talks about the installation of steel shrouds over the openings above the floor beam ends (some noted on drawings as well). We would request additional clarification on exactly what is required, Drawings, etc. To have a sealed shroud this will likely involve the cutting and removal of some structural items i.e. lacing on the chords, significant field measurements, extensive engineering, etc. In addition these are typically 316 stainless steel

   Removal of structural elements will not be required. It is not anticipated that significant field measurements and extensive engineering will be required. The Contractor shall supply shop drawings for review by Contract Administrator.

   A sample from the Burrard bridge has been provided here for information:

9. With regards to the railing (SS 09 00 00 Section 3.9.1.1) Can you clarify exactly what I required to be removed and galvanized? The requirements are clear for the outside hand railing but question is in regards to the inside railing, does any of the following components need to be removed, and galvanized or just abrasive blasted and painted in place?

The railing items identified in the photo shall be painted in place.

10. Can you confirm that temporary traffic lights for single lane alternating traffic will be acceptable in lieu of Traffic Control Persons? i.e. note on Lane Closure plan.

    The contactor shall provide a traffic management plan that meets the needs of the project traffic flow. Temporary lights in lieu of Traffic Control Persons would likely be acceptable but the operation and adequacy of the lights would be the responsibility of the contractor.

11. Section 9.3 states that a copy of the safety program manual be sent with submission whereas 12.3e states within 10 days from notice of award, does our Safety manual need to be sent with the submission? If so would a link be sufficient considering the size of this document?

The contractors project specific safety plan shall be required 10 days upon notice of award. It is not required with the RFP submission

12. Schedule B – Project Description (Pg.29) mentions maintaining access and the connections under the bridge on either side of the river. Please confirm that we are able to close the section of the road directly under the east side of the bridge. With scaffolding and containment required for the lead abatement we will be required to hang scaffold under the bridge which will encroach on the approximate 8' clearance under this section making it unsafe for any public traffic under the bridge.

    If required, the section of road under the east bridge abutment can be closed to facilitate the project components. It is expected that this section or roadway shall be closed only when necessary to complete the work at that location and for most of the project duration the road shall remain open. During closure, the contract will be expected to adjust traffic flows to ensure traffic can access the parks.

13. Supplementary specifications 1.4.4.2 (Environmental Protection) mentions that "it is suggested that containment be based on SSPC – Guide 6" please confirm that a class 1A containment is required for this project since it is a lead abatement project. Typically we would see something like the below specified in the tender/RFP documents:

    d) <u>Containment</u>

    Containment shall meet the requirements of SSPC-Guide 6, Class 1A (for abrasive blasting) and Class 1W (for water washing). Containment system component types shall comply with the following: Type A2 – Flexible Walls, Type B1 – Air Impenetrable and Type B3a – Water Impermeable, Type C1 – Rigid Support Structure, Type D1 – Fully Sealed Joints, Type E2 – Resealable Doors, Type F1 – Controlled Air Make-up, Type G1 – Forced Air Input, Type H2 – Visual Verification of Air Pressure, Type I1 – Minimum Specified Air Movement, and Type J1 – Dust Filtration.

    Containment is addressed in section 1.6 of ss 09 00 00. This references a Class 1A containment for abrasive blasting an 1W for water washing.

14. Bid Table 22 26.3 SS 1.10.3 Supply and Installation of Reference Electrodes. Can this be changed to a lump sum item as it won't necessarily be 21 units, as it will depend on the CP design and number of zones.

    The intent for the unit rate pricing is to provide pricing that is based on the number of actual Reference Electrodes installed. The proponents shall provide a cost per electrode and shall be paid for the actual number of electrodes installed.

15. Bid Table 22 26.8 SS 1.10.7 Supply and Installation of Rectifiers. Can this be changed to a lump sum item as it won't necessarily be 2 units, as it will depend on the CP design and ultimate number of zones.

The intent for the unit rate pricing is to provide pricing that is based on the number of actual Rectifiers installed. The proponents shall provide a cost per rectified and shall be paid for the actual number of rectifiers installed.

16. Bid Table 22 26.9 SS 1.10.8 Supply and Installation of Data logger and Data Management System. Can this be changed to a lump sum item as it won't necessarily be 6 units to monitor the structure, based on the CP Design.

    The intent for the unit rate pricing is to provide pricing that is based on the number of actual Reference Electrodes installed. The contractor shall provide a cost per datalogger and shall be paid for the actual number installed.

17. Request a more accurate location for the following:

    - Bridge Deck Junction Box
    - Sidewalk Junction Box
    - Under Bridge Junction Box

    These locations are up to the contractor to locate depending on their design.

18. Contract Drawings – 5th Street Bridge Upgrades o 22160-102 – Schematic plan of bridge deck has preliminary ICCP zoning identified. Please clarify that final design zoning is the responsibility of the CP Designer and can deviate from this identified zoning?

    The final design of zoning is the responsibility of the CP Designer. It is acceptable to deviate from the proposed zoning provided that evidence to support alternate zoning is provided. It is anticipated that the supplemental condition assessment will guide the actual zones for the system.

    - Detail A – If an alternate to the underside ICCP is conducted, for example the galvanic system, do all the galvanic anodes need to be monitored? Please clarify the monitoring requirements for a galvanic system in this area.

      If galvanic anodes are chosen, then a representative number shall be monitored per zone. Each zone will comprise of all of the anodes along the interstitial space between the floor beams as depicted as zones 11 to 21 on drawing 22160-102.

    - Detail A – If a galvanic system is to be substituted for the ICCP on the underside, would it also have to have a design life of 50 years. Most galvanic CP systems have a design life of 20-25 years.

      This is understood. The contractor shall submit pricing for both the galvanic and iccp ribbon anode system below the bridge as per the Schedule of Quantities and Prices items 26.5 and 26.6. One option of these two will be selected for protection of the top of the floor beams.

    - Detail A – If a galvanic system is substituted on the underside, a design that follows NACE SP0216 would be acceptable?

The galvanic system can be designed in accordance with SP0216 and ISO 12696. The more stringent clauses of the two standards shall apply.

- Detail A – Is the bottom mat of steel reinforcement electrical continuous with the steel beam?

  This is very likely. However, this needs to be confirmed by the Contractor during the work.

- Detail A – Clarify how many reference electrodes are required for the CP on the underside of the deck?

  One per representative zone. At this time we anticipate 10 zones.

19. Cathodic Protection of Steel in Concrete Section SS 26 42 00 o 1.5.3.5 – Troubleshooting steps is a broad statement without clear bounds of scope. Can the CP engineer just reference pertinent NACE and monitoring manufacturing documentation regarding troubleshooting?

    Yes this is appropriate. The contractor should expect that the troubleshooting will include basic steps to be undertaken in the event of no readings, system non-operation and if readings are well outside of the expected ranges.

- 1.5.12 – It is required that an electrical engineer stamps the power distribution to the anodes with connections details. These are low voltage items that are to specified to be designed by a NACE certified CP Specialist. Not all NACE CP Specialist are also Electrical Engineers. Is it ok if the Electrical Engineer only stamps the AC power components and the rectifier manufacture provides the wiring drawings for the rectifier? All other CP components will by stamped by a P.Eng who is a NACE CP Specialist, but that P.Eng is not necessarily for Electrical Engineering?

  An electrical engineer is to stamp the drawings for all ac power. Low voltage items can be designed by the NACE CP Specialist / P.Eng. and do not require a stamp from an electrical engineer.

- 1.6.1 – Where have the condition assessment reports which include chloride profiling, reinforcement potential surveys, depth of cover measurements been made available?

  They are appended to this addendum.

- 1.7.1 – It is unclear how to design a system with the possibility of an early failure of isolated parts of the anode system. Can this be changed to just design in accordance with NACE SP0290 and/or ISO 12696.

  The design of the impressed current anodes shall be designed to meet the requirements of ISO 12696.

- 1.7.10 – Can the EOR clarify what they mean by redundancy of monitoring components? Also, what does the EOR mean by “capacity to readily isolate section of the ICCP system.” Does this just indicate the ability to turnoff or isolate a zone?

  Redundancy for monitoring components addresses such items as multiple connections to the reinforcing steel to allow for structure potential measurement even if one connection is lost. The “capacity to readily isolate sections of the ICCP system” refers to zoning of the system and being able to shut off distinct zones and monitor the potential and current of distinct zones.

- 1.9.1 – For the computerized data-based management system, is it the intention of the owner to have this throughout the life of the structure? Typically for an ICCP monitoring system there is two levels of data management. The first is the data collected by the logger, which is stored on the logger and can be automatically sent to the owner or engineer. This does not include any graphs or presentation of the data beyond a data table. This first level is included in the purchase and installation of the equipment. The second level is a management system that creates graphs and presents the data, this is a service that is charged on a monthly or yearly basis. Is it the intension of the owner to enter into a yearly contract with a provider for this second level data-based management system for the service life of the ICCP system?

  It is the intention of the city to receive the first level of data: "The first is the data collected by the logger, which is stored on the logger and can be automatically sent to the owner or engineer." The contractor shall provide the details of the proposed monitoring for the 5-year maintenance agreement as per the optional item O.03 of the schedule of quantities and prices table.

- 1.9.6 – Data management systems can be designed to send alerts based on user-definable limits. However, this is a pre-processing step, in that it is set up upon commissioning and can be adjusted but is always forward facing, in that the alerts are only applied to incoming data. Is that the intention of this item? Or is the intention as post-processing step where the user is to define the limit and then the management system goes back and identifies the data points over this limit?

  The intention of this item is to create alerts in the event of over or under protection of individual zones.

  The intention of this item is to create alerts in the event of over or under protection of individual zones or abnormal data. The alerts shall be applied to incoming information. It is noted that regular depolarization trials are to be programmed into the system and that alerts are to be provided for zones that show over or under protection after the depolarization trials.

- 2.4.5 – Reference electrodes typically don't have service lives of 50 years; can this be adjusted to 30 years?

  Yes.

- 2.7.7 – The reference electrodes come from the manufacture with #14 AWG single-core wire, which does not meet this specification. In addition, the monitoring equipment is typically wired using #16 AWG single-core wire. Can the specification be altered to accommodate these situations? All CP wiring is #12 AWG or better, which will meet this requirement.

  Yes, this is acceptable.

- 2.9.3 – The pre-approved rectifier, listed at 2.10.19, does not meet the criteria of remote control of power supply direct current output. This is also something the monitoring equipment cannot do. Can this requirement be removed?

  "Shall be integrated" to be changed to "may be integrated"

- 2.10.10.C – The reference electrodes and steel connection for their readings go directly to the monitoring equipment, not to the rectifier. Can this specification be adjusted?

  Yes. It is understood that the reference electrodes and steel connections go directly to the monitoring equipment.

- 3.3.2 – Can the GPR be used for measuring cover-depth as well? The GPR can be calibrated on the deck to provide accurate cover-depth measurements.

  No, GPR is not considered accurate enough.

- 3.4.1 – Can the EOR clarify the procedure for pre-soaking the deck for 12 hours? Is this just wetting it once 12 hours before testing, or is this constant water on the deck for 12 hours prior to testing. Are there concerns regarding safety and traffic? Can traffic be operating on the deck during these 12 hours?

  See answer to question 4.

- 3.4.2 – "The Contract Administrator shall identify areas on the underside of the bridge adjacent to the I-Beams to be protected with the ICCP system." What are the criteria for which areas are to be protected and not protected?

  This will be based on a visual assessment to identify areas of corrosion at the upper flange / concrete interface in combination with chloride sampling and potentially half cell testing.

- 3.5.1 – What is the pass or fail criteria for the tensile testing?

  The tensile bond shall be greater than 1.25 MPa

- 3.6.3 – What does "make good all test locations" mean?

  If the Owner requires chipping to remove cover concrete at an isolated location to review the condition of reinforcing steel then the contractor shall reinstate the concrete at no cost to the owner. Additionally if the Owner drills into the concrete to obtain powder samples for chloride analysis then the contractor shall repair the test locations.

- 3.9.1 – Can the CP engineer propose an alternate method? VCS has had success with a magnet that is run over the surface of the deck. Is that acceptable?

  Further information is needed to determine if this is an acceptable method.

- 2.5.9 – Does not indicate DC voltage from each zone should be measured but then 3.10.2 indicates it should be. Please clarify.

  DC voltage from each zone shall be measured.

- 3.18 – There is no price item for Operation and Maintenance in the bid table and this is indicated as to be contracted separately per 3.18.7. Are the contractors to submit pricing for this?

  The 5-year maintenance agreement is listed as an optional item in the Schedule of Quantities

- 3.18.3.A – Please define what "satisfactory performance" is?

  Satisfactory performance indicates that only minor adjustments at the rectifier are required and that no repairs are required.

- 3.18.3.C – Please define what "satisfactory performance" is?

  Satisfactory performance indicates that only minor adjustments at the rectifier are required and that no repairs are required.

- 3.18.3.E – Please define what "satisfactory performance" is?

  Satisfactory performance indicates that only minor adjustments at the rectifier are required and that no repairs are required.

- 3.18.5 – "The Contractor shall be responsible for maintenance and repair to system not covered under the warrantee." This statement is not clear, the contractor is responsible for everything not covered by a warrantee, but that would be a warrantee.

  The warrantee period and the maintenance period should be considered separate. The costs proposed for maintenance of the system should not include warrantee items and associated costs which would be expected to be covered under the initial project contract. Items that are deemed to be outside of warrantee shall be covered under the maintenance contract.

- 3.19.3 – Is the monitoring equipment to have a modem, or just on-site download of the data from the datalogger storage?

  See answer to question 7.

- ▪ If there is no modem, how is the data management system to interface with the data logger. Data management systems are typically stand-alone system, primarily web-based.

  See answer to question 7.

- 3.20.11 – All pre-bagged material will have a resistivity measured and provided by the manufacture, is that sufficient, instead of having to test each batch of pre-bagged on site?

FM 5-578 requires the samples to be moisture cured prior to testing, so the bag material will already be in place and cured by time the results are gathered.

Resistivity measurements by the manufacturer are acceptable provided that the testing has been completed within the last year. It is not necessary to test each bag on site. A minimum of one test for resistivity will be required to be completed on one of the bags used on site.

- 3.23.1 – Please define substantial performance of the work

Substantial Performance of the work is defined by the MMCD General Conditions.

20. Are There any height restrictions for the bridge containment.

Height and traffic restrictions are detailed in Supplemental Specification 01 55 00

21. How much room is available on either side of the bridge for material storage or laydown

The contractor may use the bridge approach areas on either side of the bridge for equipment as long as there is adequate space for traffic lane configurations and pedestrian movements

22. What are the City noise bylaws and are they able to be modified for this project?

The City of Courtenay noise bylaw can be found online:

https://www.courtenay.ca/EN/main/city-hall/bylaw-enforcement/noise.html

The City will consider allowances for work outside of the bylaw if appropriate noise abatement or other measures are put in place and that the work is deemed to be an advantage to the schedule or costs of the project. For the purposes of the RFP the proponents should highlight these advantages and the potential abatement measures.

Item No. 2 – Clarifications/RFP Updates

Section SS 01 52 01 Temporary Structures

Add clause 1.3.2

.2 The contractor shall supply a mobile office for the exclusive use of City of Courtenay Staff and the Contract Administrator. The trailer shall be a Britco 10x32 mobile office with two separate offices and a main foyer. Alternates subject to approval of Contract Administrator. The mobile office shall be provided with two desks and two chairs, 240 V power for heaters, 120 V power for air conditioning and electrical plugs, and a reliable internet connection. The site trailer shall be provided for the duration of the project.

Section SS 09 00 00 Recoating of Structural Steel

Add clause 3.6.4.2

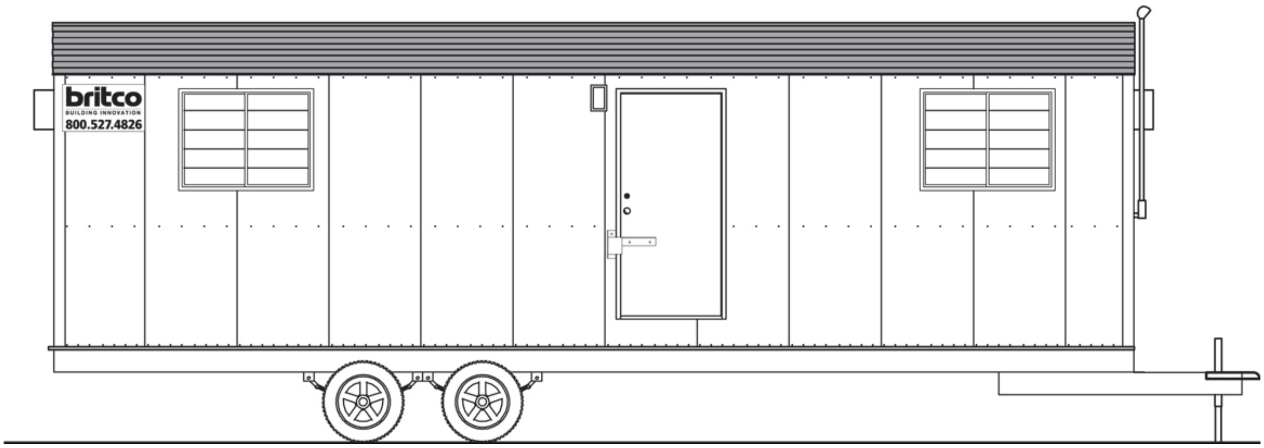
.2 After blast cleaning, sharp edges shall be rounded to a minimum 2mm radius.

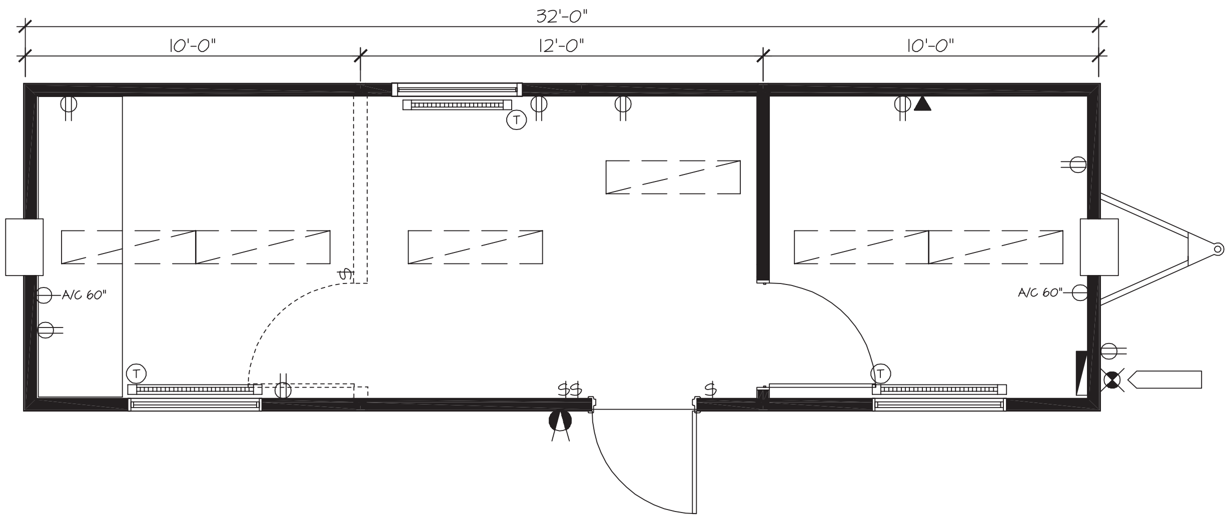
Acknowledgement of this Addendum as part of your submission is required.

End of Addendum No. 2

Bernd Guderjahn, SCMP
Manager of Purchasing
City of Courtenay

10 x 32
Mobile Office





General Specifications

Exterior Siding: Metal siding & metal fascia

Interior Walls: Wood paneling (New York Birch) or Vinyl-clad paneling (Norwich Pearl)

Windows: 4'-0" x 3'-0" XO horizontal slider with insect screen & security bars

Exterior Doors: Solid-core door with passage set, deadbolt, check chain and steel lockbox

Interior Doors: Hollow-core door, prefinished wood with passage set (if applicable)

Floor: Vinyl-composite tile (commercial grade)

Ceiling: Vinyl-clad gyproc (white)

Roof: EPDM (rubber) roofing

Heating/Cooling: Electric baseboard heaters (240V) and air conditioning (120V)

Lighting: Fluorescent lights, 2-bulb, surface-mounted; Exterior lights

Electrical: 120/240 volt single phase, mast & weatherhead, electrical panel, duplex wall receptacles and tel/data conduits (jacks & wiring not included)

FIFTH STREET BRIDGE

BRIDGE DECK & STEEL COATING EVALUATION REPORT

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Project no: 161-05488-00

Issue date: June 2016

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REVISION HISTORY

CLIENT

REV. NO	ISSUE DATE	DESCRIPTION OF REVISION

TABLE OF CONTENTS

APPENDICES

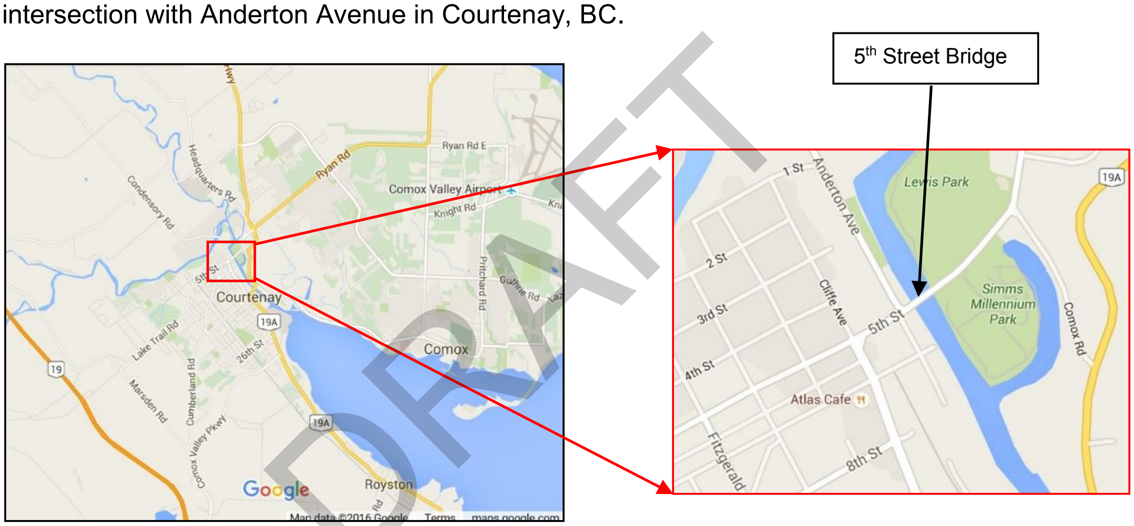
TABLES

1 INTRODUCTION

Hatch Mott MacDonald Ltd. (HMM) and Urban Systems Ltd. (USL) have been engaged by the City of Courtenay to complete an initial scoping study to determine what, if any, upgrades are required for the 5th Street Bridge to maintain the structure in operating condition for an additional twenty plus years. As part of this work, HMM retained WSP Canada Inc. (WSP) to conduct an evaluation of the concrete deck and the structural steel coating. The field investigation was conducted on May 12, 2016.

2 LOCATION

The 5th Street Bridge crosses the Courtenay River and is located on 5th Street just east of the intersection with Anderton Avenue in Courtenay, BC.



3 SCOPE OF WORK

As part of this assignment the following work was completed:

3.1 BRIDGE DECK CONDITION ASSESSMENT

- Visual condition survey of the structural elements of the bridge and bridge deck. Noteworthy observations were photographed and logged.
- Delamination survey generally in accordance with ASTM D4580 *Standard Practice for Measuring Delaminations in Concrete Bridge Decks by Sounding* at the top surface of the bridge deck to identify delaminated areas.
- Measurement of corrosion potential values generally in accordance with ASTM C876 *Standard Test Method for Corrosion Potentials of Uncoated Reinforcing Steel in Concrete*. Measurements were made on a grid pattern of 0.6 m (transverse) x 1.0 m (longitudinal) over the top surface of the bridge deck.

- Removal of concrete core samples at six locations for measurement of the water-soluble chloride ion content in accordance with CSA A23.2-4B, and alkalinity of the concrete pore solution. The coring was performed to the approximate level of the bottom mat of reinforcement.
- Measurement of the cover to steel reinforcement at various locations.
- Preparation of recommendations for rehabilitation options and an opinion on repair estimates.

3.2 STRUCTURAL STEEL COATING ASSESSMENT

- Visual review of the in-place coating system to determine the extent of the advancement of the deterioration of the coating system by utilizing background report prepared by McElhanney in 2008 as the baseline.
- Assessment of the degree of rusting in accordance with ASTM D610 “Standard Practice for Evaluating Degree of Rusting on Painted Steel Surfaces” was completed at select locations.
- Determination of the most cost-effective method for extension of the service life of the bridge for an additional twenty years while satisfactorily protecting the structural steel from corrosion-related deterioration.
- Development of a proposed rehabilitation program that accounts for presence of lead in existing primer and provides a framework for environmental and worker protection.
- Provision of an opinion on the repair estimate for the re-coating work.

4 BACKGROUND

A background report prepared in 2008 by McElhanney Consulting Services Ltd. (McElhanney) was provided for review. This report indicates that a 60 mm concrete overlay was placed on the deck in 1984. The steel truss members were coated in 1983. The structural elements of the bridge underwent a seismic upgrade in 1996. Aside from expansion joint replacement in 2012, it is unknown whether any additional has been completed since 2008.

The McElhanney report indicates that at the time of the field review in August 2008:

- Structural steel was in good to excellent condition;
- Steel coating was worn out and in need of full-scale replacement;
- Minor concrete spalling had occurred at localized areas of the sidewalk;
- Sidewalk railings were in good condition;
- Concrete bridge deck was in good condition with isolated areas of delamination;
- Bridge deck expansion joints were in a state of disrepair;
- Bridge substructure was in good condition; and
- Rip-rap was in good condition.

5 BRIDGE DESCRIPTION

Built in 1960, this single-span structure is straight and level, and carries one lane of traffic in each direction. The exposed concrete deck is supported by several deep transverse steel wide-flange floor beams and shallow longitudinal steel stringers that are in turn supported by a steel through-truss with a Pratt Truss configuration. The entire superstructure is supported by two concrete abutments. The concrete abutments at each end of the bridge consist of a bearing seat, ballast wall, and wing walls.

There are concrete curbs and sidewalks along both sides of the bridge; the combined width of the curb and sidewalk along each side is approximately 2.5 m (8 ft); the curbs are 0.27 m high.

There are steel rails along both sides of the bridge which are attached to steel posts which are anchored to the side of the outermost stringers. The sidewalk along each side is delineated from traffic by c-channel and a hollow, tubular rail.

The bridge has an overall length of 76.2 m (250 ft) as measured between centreline of the bearings and an overall roadway width of 7.3 m (24 ft) giving a total deck area of approximately 556 m^2 (6,000 ft^2). The chainage referenced in this report increases from west to east. A drawing showing the general arrangement of the bridge is presented in Appendix A; Photos 1 through 4 in Appendix B show standard deck and elevation views.

6 VISUAL INSPECTION

The following comments are based on a visual inspection. Photographs showing standard site and component views, typical and isolated defect and deterioration severity levels, and those of other noteworthy importance are logged in Appendix B.

6.1 DECK SLAB

6.1.1 DECK SURFACE

Visually, the concrete deck surface is in fair to good condition with minor delaminations and cracking. The delaminations are generally spread throughout the deck with larger delaminations concentrated in the eastbound lane adjacent to both abutments (Photo 5 and Photo 6).

There are some partial-width transverse cracks present at some locations in the eastbound lane. These cracks have been injected with an epoxy, which appears to be doing an acceptable job of sealing the cracks at some locations than has at others (Photo 7 to Photo 10). The width of cracks observed vary between 0.1 mm and 0.2 mm. There are also minor pop outs at two localized areas in the eastbound lane (Photo 11).

There is a moderate amount of paste loss along the wheel paths in both directions, which has exposed the tops of coarse aggregate (Photo 12); however, the rideability of the deck is generally good. The remaining areas of the deck has also experienced paste loss though to a lesser extent (Photo 13).

6.1.2 CURBS, SIDEWALK AND RAILINGS

The concrete curbs are in generally good condition with no noticeable deterioration. The sidewalks are in fair condition with partial to full width transverse cracks at multiple locations (Photo 14 to Photo 16). The width of the cracks vary between 0.1 mm to 0.3 mm. The sidewalk has been patched with a cementitious grout in localized areas; these patches appear to be in good condition.

The coated steel rails and posts are in fair condition with several areas of coating failure. There is minor damage to the north rail at a localised area at approximate mid span of the bridge (Photo 17). The deck drains are generally free and clear of debris (Photo 18).

6.1.3 JOINTS

There is an expansion joint at the west abutment and a fixed joint at the east abutment. These joints were replaced in 2012. Both joints are in good condition with no significant deterioration observed (Photo 19 and Photo 20). The joint information is presented in Table 1.

Table 1: Summary of Joint Information

CHAINAGE (M)	JOINT TYPE	JOINT WIDTH (MM)	CONDITION OF ARMOUR	CONDITION OF SEALS
0.0	Expansion	60	Good	Good
77.2	Fixed	50	Good	Good

6.1.4 ASPHALT APPROACHES

Both asphalt approaches to the bridge are generally straight with a slight slope increasing from east to west at the west end of the bridge, and decreasing from east to west at the end (Photo 21 and Photo 22). There is minor ravelling at both approaches, and minor rutting in the wheel paths at the west approach.

6.2 DECK UNDERSIDE

6.2.1 STEEL MEMBERS

The exposed concrete deck is supported by several 845 mm deep transverse steel I floor beams and 450 mm deep longitudinal wide-flange steel stringers which are in turn supported by a steel through-truss with a Pratt Truss configuration (Photo 23 and Photo 24). The lateral support to the transverse floor beams is provided by steel cross bracing. Both the steel beams and stringers appear to be in generally fair to good condition in most areas. There is coating failure and corrosion in some areas, particularly at the top flange of the beams and stringers (Photo 25)

The steel truss members that are above the deck surface also appear to be in good condition. However, the coating has failed in multiple areas.

6.2.2 DECK SOFFIT

Visually, the deck soffit between the river shores appears to be in good condition; however, a close examination was not possible (Photo 23 and Photo 24). It is noted that pattern cracking with efflorescence was observed from the river banks at a localized area at the western end of the bridge (Photo 24).

The deck soffit between the abutments and the river banks is generally in fair condition with areas of cracking with efflorescence and localized spalling. There are areas of minor cracking with efflorescence near the east abutment, while at the areas adjacent to the west abutment there is a moderate amount of pattern cracking with efflorescence (Photo 26 to Photo 28). It is noted that a close review of the soffit immediately adjacent to the abutments was not possible as access was restricted by a chain-link fence.

There are some areas where minor spalling of the concrete was observed, which generally has occurred adjacent to the top flange of the floor beams or stringers near the west abutment (Photo 29).

The curb and sidewalk soffit along both sides of the bridge is also in generally fair condition with areas of cracking, and localized spalling. There are a few areas where spalling has occurred which has exposed the corroding reinforcing steel; the concrete cover to the bottom mat of reinforcing steel is shallow in these areas (Photo 30 to Photo 32).

There is leakage staining visible in some areas of the deck, particularly in the outer portions of the sidewalk and curb soffit adjacent to the vertical tension members (Photo 33 to Photo 35).

6.2.3 ABUTMENTS

Both abutments appear to be in good condition; however, a close review was not possible due to the presence of chain-link fence. There is leakage visible at the bearing areas at both abutments; the water in these areas is leaking through the curb openings for the truss members (Photo 36 to Photo 38).

6.2.4 BEARINGS

The bearings at both abutments appear to be in generally good condition; however, as with the abutments, a close review was not possible due to the presence of chain-link fence (Photo 39 and Photo 40).

7 DELAMINATION SURVEY

The delamination survey was performed in accordance with ASTM D4580 using the Chain Drag Method as outlined in Procedure B. The survey was performed over the entire deck surface, and the results are listed in Table 2 and graphically presented in Drawing 1 in Appendix C.

The delamination survey indicates that less 1% of the deck surface is currently delaminated with small delaminations present at several locations in the westbound lane. A few larger delaminations are present adjacent to the abutments in the eastbound lane.

Table 2: Summary of Delamination Data

	DECK AREA	DELAMINATIONS (EXCLUDING PATCHES)		DELAMINATED PATCHES		SOUND PATCHES		TOTAL DELAMINATIONS (INCLUDING DELAMINATED PATCHES)		TOTAL PATCHES & DELAMINATIONS	
Lane	(m²)	Count	Area (m²)	Count	Area (m²)	Count	Area (m²)	Area (m²)	%	Area (m²)	%
Westbound Lane	274.3	9	1.5	0	0.0	0	0.0	1.5	0.5	1.5	0.5
Eastbound Lane	281.9	10	2.9	0	0.0	0	0.0	2.9	1.0	2.9	1.0
Total	**556.3**	**19**	**4.4**	**0**	**0.0**	**0**	**0.0**	**4.4**	**0.8**	**4.4**	**0.8**

8 HALF-CELL POTENTIAL SURVEY

The half-cell potential survey was conducted generally in accordance with ASTM C 876 using a standard copper/copper-sulphate reference electrode (CSE) over the entire deck surface. Electrical connection to the steel was achieved by coring to the depth of the top mat of steel at two locations, one in each lane; these locations were used for the connection points for the remainder of the survey. A 0.61 m (transverse) by 1.0 m (longitudinal) grid was used for data collection, translating into 1001 individual potential measurements with an average potential reading of -53 mV_{CSE}. The results of the survey are summarized in Table 3, and are graphically presented in Drawing 2 in Appendix D.

Approximately 96% of the deck area surveyed is at a low probability for corrosion of the reinforcing steel. However, elevated corrosion potentials are present in some areas of the deck.

Table 3: Summary of Half-cell Potential Survey

PROBABILITY OF CORROSION	POTENTIAL RANGE MV_{CSE}	PERCENTAGE OF DECK AREA
<10%	More +ve than -199	**96**
Uncertain	-199 to -350	**4**
>90%	More -ve than -350	**0**

9 EXTRACTED CORES

In order to determine chloride profiles down to the bottom mat of reinforcing, it was necessary to extract the samples by coring to a depth of approximately170 mm. A total of six cores were extracted at select locations from sound areas of the deck for chloride ion sampling, three from each lane; these cores correspond to cores 1 through 6. Two additional cores, Cores 7 and 8, were removed to expose the top mat of reinforcing for connection points for the half-cell survey. The photographs of the extracted cores and the condition of the exposed reinforcing at the core locations is shown in Photo 41 to Photo 44. The core sample locations are graphically presented in Drawing 1 in Appendix C. The observations regarding each core are summarized in Table 4.

Table 4: Core Log

CORE NO.	LANE	CHAINAGE (M)	OFFSET FROM NEAREST CURB (M)	REBAR DEPTH (MM)	DIRECTION OF REBAR	SIZE OF REBAR (MM)	REBAR CONDITION	OVERLAY DEPTH (MM)
1	EB	2.6	0.6	(180)	L	10M	Fair	75
2	EB	31.4	2.5	176*	L		NV	74
3	EB	56.6	1.1	(164)	L	10M	Fair	65
4	WB	15.1	0.8	(162)	L	10M	Good	70
5	WB	41.1	2.3	(175)	L	10M	Fair	75
6	WB	75.6	2.9	167* (202)	L T	10M (5 mm wire mesh)	NV Good	80
7	EB	22.6	0.5	(95) (113)	T L	15M 10M	Good Good	70
8	WB	43.6	0.4	(96) (112)	T L	15M 10M	Good Good	75

Notes:
*Denotes rebar depth obtained with the use of ground penetrating radar (GPR).
Values shown in parenthesis are actual measured cover.
Codes: EB – Eastbound; WB – Westbound; T – Transverse; L – Longitudinal; NV – Not Visible

10 TOP COVER MEASUREMENTS

Clear concrete cover thickness to the top mat of rebar was measured throughout the bridge deck; the results of this survey are presented in Table 5. The concrete cover to the rebar ranges from 45 mm to 120 mm with an average value of approximately 83 mm.

Table 5: Top Cover Measurements (mm)

CHAINAGE (M)	EB LANE (CURB EDGE)	EB LANE (CENTRE)	BRIDGE CENTERLINE	WB LANE (CENTRE)	WB LANE (CURB EDGE)
2	110	113	65	113	99
10	87	99	58	97	99
20	57	53	61	54	45
30	99	94	67	84	96
40	98	109	64	96	85
50	105	105	72	101	98
60	93	94	65	92	87
70	53	48	62	53	55
76	97	98	62	92	102

11 LABORATORY TESTING

As discussed previously, samples for determination of water-soluble chloride ion content were obtained by coring. The cored specimens were sliced into 20 mm increments and crushed into a powder, which was then analyzed for water-soluble chloride ion content in accordance with CSA A23.2-4B. Sampling locations are graphically presented in Drawing 1 in Appendix C and the test results are summarized in Table 6.

The top mat of rebar is at an estimated average depth of 83 mm from the deck surface, and the bottom mat is at approximately 170 mm. The chloride ion concentrations at the depth of the top mat of rebar at five of the six locations and three of the six locations at the bottom mat are at, or greater than, the generally accepted corrosion initiation threshold. This threshold value may vary between 0.03% to 0.05% by mass of concrete, depending on several conditions such as moisture, temperature, and pH of surrounding concrete.

At some locations there is an increase in chloride ion concentrations from one depth to the next. This indicates that not all of the chloride-contaminated concrete was removed during the previous resurfacing work. A review of the extracted cores confirms the presence of the deck overlay; the average depth of overlay is approximately 73 mm.

The alkalinity of the concrete pore solution in new concrete (pH > 12) passivates the steel at the steel/concrete interface and reduces corrosion rates to negligible levels. Carbonation of the concrete will result in a lower pH and causes breakdown of the passivation layer. At pH values of approximately 10 or lower, the steel is no longer passivated and active corrosion may be initiated.

Laboratory results indicate that the pH at the level of the top mat at the majority of the locations ranges from 12.0 to 12.4, which indicates that the alkalinity of the concrete is sufficiently high to conclude that carbonation-induced corrosion mechanism is not occurring at the present time.

Table 6: Water-soluble Chloride Content

CORE NUMBER	REBAR COVER (mm)	SAMPLE DEPTH (mm)	WATER-SOLUBLE CHLORIDE IONS CSA A23.2-4B (% MASS OF CONCRETE)	pH
1	110*/T (180/L)	5-20	**0.157**	12.1
		20-40	**0.147**	12.2
		40-60	**0.102**	12.3
		60-80	**0.071**	12.1
		80-100	**0.075**	12.2
		100-120	**0.069**	12.3
		120-140	**0.064**	12.3
		140-160	**0.043**	12.3
		160-180	0.027	12.2
2	94*/T 176*/L	5-20	**0.117**	12.1
		20-40	**0.127**	12.3
		40-60	**0.077**	12.2
		60-80	**0.082**	12.2
		80-100	**0.079**	12.2
		100-120	**0.095**	12.3
		120-140	**0.077**	12.3
		140-160	**0.085**	12.2
		160-179	**0.051**	12.1

Table 6: Water-soluble Chloride Content Continued

CORE NUMBER	REBAR COVER (mm)	SAMPLE DEPTH (mm)	WATER-SOLUBLE CHLORIDE IONS CSA A23.2-4B (% MASS OF CONCRETE)	pH
3	99*/T (164/L)	5-20	**0.116**	12.2
		20-40	**0.098**	12.1
		40-60	**0.101**	12.2
		60-80	**0.123**	12.2
		80-100	**0.089**	12.0
		100-120	**0.114**	12.3
		120-140	**0.096**	12.2
		140-160	**0.077**	12.2
		160-179	**0.074**	12.2
4	99*/T (162/L)	5-20	**0.062**	12.3
		20-40	**0.062**	12.3
		40-60	**0.053**	12.3
		60-80	**0.059**	12.2
		80-100	**0.082**	12.2
		100-120	**0.083**	12.4
		120-140	**0.060**	12.3
		140-160	0.029	12.2

Table 6: Water-soluble Chloride Content Continued

CORE NUMBER	REBAR COVER (mm)	SAMPLE DEPTH (mm)	WATER-SOLUBLE CHLORIDE IONS CSA A23.2-4B (% MASS OF CONCRETE)	pH
5	96*/T (175/L)	5-20	**0.059**	12.1
		20-40	**0.062**	12.2
		40-60	0.025	12.1
		60-80	0.028	12.3
		80-100	0.027	12.2
		100-120	0.023	12.3
		120-140	0.014	12.3
		140-160	0.008	12.2
		160-176	0.008	12.3
6	65*/T 167*/L	5-20	**0.078**	12.1
		20-40	**0.093**	12.1
		40-60	**0.097**	12.3
		60-80	**0.095**	12.3
		80-100	**0.137**	12.2
		100-120	**0.141**	12.2
		120-140	**0.108**	12.2
		140-160	**0.081**	12.2
		160-179	**0.056**	12.3
		180-209	**0.0321**	12.1

Notes:
*Denotes rebar depth obtained with the use of ground penetrating radar (GPR).
The bottom mat of rebar is at an average measured depth of 170 mm and top mat at 83 mm.
Values shown in parenthesis are actual measured cover.
Chloride ion concentrations greater than the corrosion initiation threshold are shown in bold.
T – Transverse; L – Longitudinal;

12 STEEL COATINGS

12.1 OBSERVATIONS AND FIELD TESTING

WSP's bridge coating review was limited to a non-destructive visual examination from the deck level and walkways adjacent to the substructure. Our findings are summarized as follows:

- At least seven coatings were visually discernable on the structure in varied zones including:
  - Lead oxide primer;
  - Silver midcoat (Carbomastic 15 per Trans Canada Coatings Consultants (TCCC));
  - Excessively thick and distressed green top coat (Polyclad 936 vinyl per TCCC);
  - White epoxy (north end portal repair);
  - Blue-grey touch-up (primarily graffiti repair);
  - Weathered bronze coloured top coat (below deck); and
  - Hot dip galvanized plan bracing (below deck).
- It appeared that the silver mid coat (perceived to be Carbomastic 15) on the truss superstructure and railings remained largely intact and continuous with the exception of the upper surfaces of the top chord (Photo 45 to Photo 47).
- The extent of coating failure and the extent of active rusting of the substrate was estimated per ASTM D610. Visual estimates are described in Table 7.

Table 7: Extent of Area Corroded (ASTM D610 Score)(1)

COMPONENT	TCCC ESTIMATE 2008(2)	WSP ESTIMATE 2016(3)
Arch (Above Splash Zone)	6.80	5.50
Splash Zone (Deck - +3.5 m)	7.60	7.00
Fascia (Upstream and Downstream)	6.95	6.50
Protected Areas	7.30	7.00
Substructure (Including Plan Bracing)	6.30	5.50
Overall Average	7.00	6.30

Notes:
1. ASTM D610 Score pertains to the following area corroded:

D610 SCORE	% AREA CORRODED
10	< 0.01
9	< 0.03
8	< 0.1
7	< 0.3
6	< 1
5	< 3
4	< 10
3	< 16.7
2	< 33
1	< 50
0	> 50

2. ASTM D610 interpolated from TCCC Fifth Street Bridge Coating Evaluation 2008.
3. ASTM D610 score estimated visually.

- The extent of top coat failure and or substrate rusting has marginally increased since TCCC's comprehensive evaluation in 2008. We estimate that as much as approximately 1% of the total surface area overall is actively rusting; regions above the splash zone adjacent to the traffic path and below the deck were the most prominent examples (Photo 48 to Photo 51).
- Owing to widespread failure of the perceived vinyl topcoat, the superstructure surfaces appeared rough and were frequently contaminated with algae / biological deposits (**Photo 52 to Photo 54**).
- The most prominent instances of coating failure that exposed red lead oxide primer were observed on the railing systems adjacent to the roadway and adjacent to the sidewalks (**Photo 55 and Photo 56**, respectively).
- It appeared that considerable maintenance painting of the steel below the deck had been maintained for both corrosion control and graffiti repair (Photo 57 and Photo 58).
- Portions of the substructure fascia on both the west (upstream) and east (downstream) sides of the bridge were obscured by utilities (Photo 59 and Photo 60).
- Coating dry film thickness (DFT) was evaluated at accessible locations in the splash zone and below deck using an electronic gauge (Positector 6000) calibrated for ferrous substrates. Results are described in Table 8 below.

Table 8: Dry Film Thickness

LOCATION	GAUGE READING (MILS)	SPOT READING (MILS)
Edge Beam BF	19.4, 19.4, 19.9	19.6
Diagonal Brace, south end	17.5, 17.6, 17.6	17.6
Floor Beam, BF south	16.8, 17.1, 15.5	16.5
Edge Beam, BF south end	17.5, 18.7, 18.3	18.2
Top Chord at Sidewalk, south end	13.9, 14.0 14.1	14.0
Lacing, south end	23.2, 26.2, 24.5	24.6
Rub Rail, west	20.2, 16.8, 18.2	18.4
Vertical Web	29.3, 27.6, 27.8	28.2
Vertical Flange	20.5, 22.5, 21.4	21.5
Railing Post	20.4, 18.3, 18.5	19.1
Diagonal Web	23.2, 21.3, 22.1	22.2
Rub Rail, silver	13.4, 14.5 13.9	13.9
Vertical Flange	17.6, 16.4, 16.3	16.8
Rub Rail, primer only	3.6, 3.4, 3.1	3.37*
Rub Rail, primer only	2.9, 2.3, 2.8	2.67*
Rub Rail, primer only	2.0, 2.0, 1.9	1.97*

Table 8: Dry Film Thickness Continued

LOCATION	GAUGE READING (MILS)	SPOT READING (MILS)
Rub Rail, with silver	13.3, 14.4, 14.0	13.9
Hanger, adjacent to corrosion	5.7, 5.7, 6.5	5.97*
Hanger Flange	27.9 ,30.5, 29.4	29.3
Diagonal Brace, pitted	5.3, 4.3, 3.3	4.3*
Gusset Plate	22.4, 20.6, 21.6	21.5
Hanger	19.9, 20.3, 19.8	20.0
Railing Post, west	2.1, 4.6, 4.2	4.63*
Top Rail	11.8, 13.7, 17.2	14.2
Top Chord, web	15.7, 16.1, 16.4	16.1
Edge Girder, BF	19.1, 18.9, 18.6	18.9
Diagonal Brace	14.5, 14.7, 14.0	14.4
Railpost, east	3.9, 3.4, 3.7	3.67*
Hanger flange	15.6, 17.0, 17.1	16.6
Diagonal Brace, web	11.9, 11.3, 13.0	12.1
Rub Rail, alligator cracking	10.6, 11.8, 11.3	11.2

Table 8: Dry Film Thickness Continued

LOCATION	GAUGE READING (MILS)	SPOT READING (MILS)
Diagonal Brace, silver	11.7, 11.1, 11.8	11.5
Diagonal Brace	20.1, 19.9, 19.7	19.9
Hanger Web	14.3, 12.7, 14.3	13.8
Hanger, alligator cracking	25.1, 26.8, 22.1	24.7
Rail Post, primer only	3.7, 3.8, 4.3	3.93*
Lacing, truss element	14.0, 14.5, 15.1	14.5

Notes:
1. *Denotes incomplete coating system; spot measurements excluded from statistics.
2. Twenty-nine (29) observations; mean 18.0; standard deviation 4.68; maximum 29.3 mil; minimum 11.2 mils.

→ It was noted that the total coating system thickness varied widely (minimum 11.2 mils to a maximum of 29.3 mils). The average total system thickness was 18 mils. Instances of mechanical damage and/or wear that exposed the primer were primarily limited to the rub rail and hand railing adjacent to the vehicular lanes and sidewalks.

13 INTERPRETATION OF FINDINGS

Visually, the bridge deck is in fair to good condition with areas of minor delaminations and cracking. The delamination survey indicates that less than 1% of the deck surface has been delaminated. There is a minor amount of transverse cracking in the eastbound lane; the majority of these cracks in the deck have been sealed at the surface with an epoxy.

The half-cell survey indicates that the majority of the deck surface is at a low probability of the corrosion of the reinforcing steel. However, there are several closely-spaced equipotential lines in some areas of the deck which indicate active corrosion cells.

The chloride ion concentrations at the level of top mat of reinforcing steel at all but one sample location are at, or greater than, the corrosion initiation threshold of 0.03 to 0.05% by mass of concrete. Chlorides have reached the bottom mat of reinforcing steel at three of the six locations in sufficient concentration to initiate corrosion and are approaching the initiation threshold at other locations.

The pH values are sufficiently high to conclude that the carbonation-induced corrosion is not a concern at this time.

A review of the substructure and the deck soffit indicates that they are generally in fair condition in most areas. The deck soffit, curbs, and sidewalks are experiencing minor corrosion-related deterioration in localized areas; the concrete cover in these locations was observed to be low. The steel members have experienced coating failure in several areas of the deck, both at the substructure and superstructure level.

14 REHABILITATION/MAINTENANCE RECOMMENDATIONS

14.1 DECK CONDITION ASSESSMENT

The deck has generally experienced minor amounts of corrosion-related deterioration. The visual, delamination, and half-cell surveys indicate that there is some minor corrosion activity occurring in the reinforcing steel which is not widespread at this time. However, the results of the laboratory analyses indicate that elevated levels of chlorides, sufficient to initiate corrosion, exist at the level of the top mat of reinforcement generally throughout the deck. As such, it should be anticipated that widespread corrosion will occur in the near to mid future. It is also noted that there is presence of elevated chlorides at the level of the bottom mat of rebar at some locations which may result in corrosion of the bottom mat of steel.

It is understood that the City of Courtenay wishes to extend the service life of this structure nominally by 20 years, and are currently exploring options for rehabilitation. Given the desired extension of the service life, and the presence of chloride-contaminated concrete at the level of top mat of rebar throughout and at some locations at the level of the bottom mat, it is WSP's opinion that a new overlay should be installed within the next five to eight years.

WSP recommends that the new overlay be installed by removing chloride contaminated concrete to below the top mat of reinforcement over the entire deck and reinstating with a high quality concrete overlay. It is anticipated that the new overlay would significantly reduce or arrest corrosion of the bottom mat. In addition to a new overlay, full-thickness repairs may be necessary where pattern cracking is present in the deck soffit, primarily near the west abutment.

14.2 STEEL COATING ASSESSMENT

Based on WSP's visual assessment of steel coatings, WSP believes that the existing coatings have remaining service life of five to seven years. Practical service life is considered to be the time until 5 to 10% coating breakdown occurs (ASTM D610 Score 4) and active rusting of the substrate is occurring. From an aesthetic point of view the existing coatings are extensively degraded. Considerable surface roughness tends to retain deposits, enhance biological activity, and increase the time of wetness – all factors that will serve to hasten more widespread coating failure.

Although WSP did not assess coating adhesion for this review, WSP is in substantial agreement with the opinions express by Trans Canada Coating Consultants Ltd. in 2008, whereby it was assessed that the bridge is a poor candidate for overcoating owing to poor to fair existing adhesion and limited flexibility to tolerate further coating without the risk of major failures. In this regard, the bridge is considered a poor candidate for overcoating; however, it may be a candidate for encapsulation using calcium sluphonate alkyd (CSA) formulations. The performance of CSA systems are known to vary widely after approximately 10 years of exposure; and, although they are not aesthetically pleasing, CSAs tend to retain high levels of flexibility.

The alternate philosophy, which WSP is more inclined to support, is that of utilizing all of the practical service life of the existing coating (allowing it to weather for an additional five to seven years) before undertaking blast cleaning to bare metal, abatement, and recoating with a three coat system. Such intervention would only be considered prudent if the bridge was going to be utilized for at least 20 years beyond the recoat date.

15 OPINION OF PROBABLE COST

15.1 DECK

It is understood that the City requires approximate costs for recommended deck repairs. The estimate presented in Table 9 involves installation of a high quality overlay in five years, and full-thickness repairs in some portions of the deck at the time of new overlay installation. The estimate below is based on unit rates published in the BC Ministry of Transportation and Infrastructure's Construction and Rehabilitation Cost Guide 2013. Please note that the figures presented below are net present values and a discount rate of 3% was used.

Table 9: Opinion of Probable Cost – Deck Slab Overlay

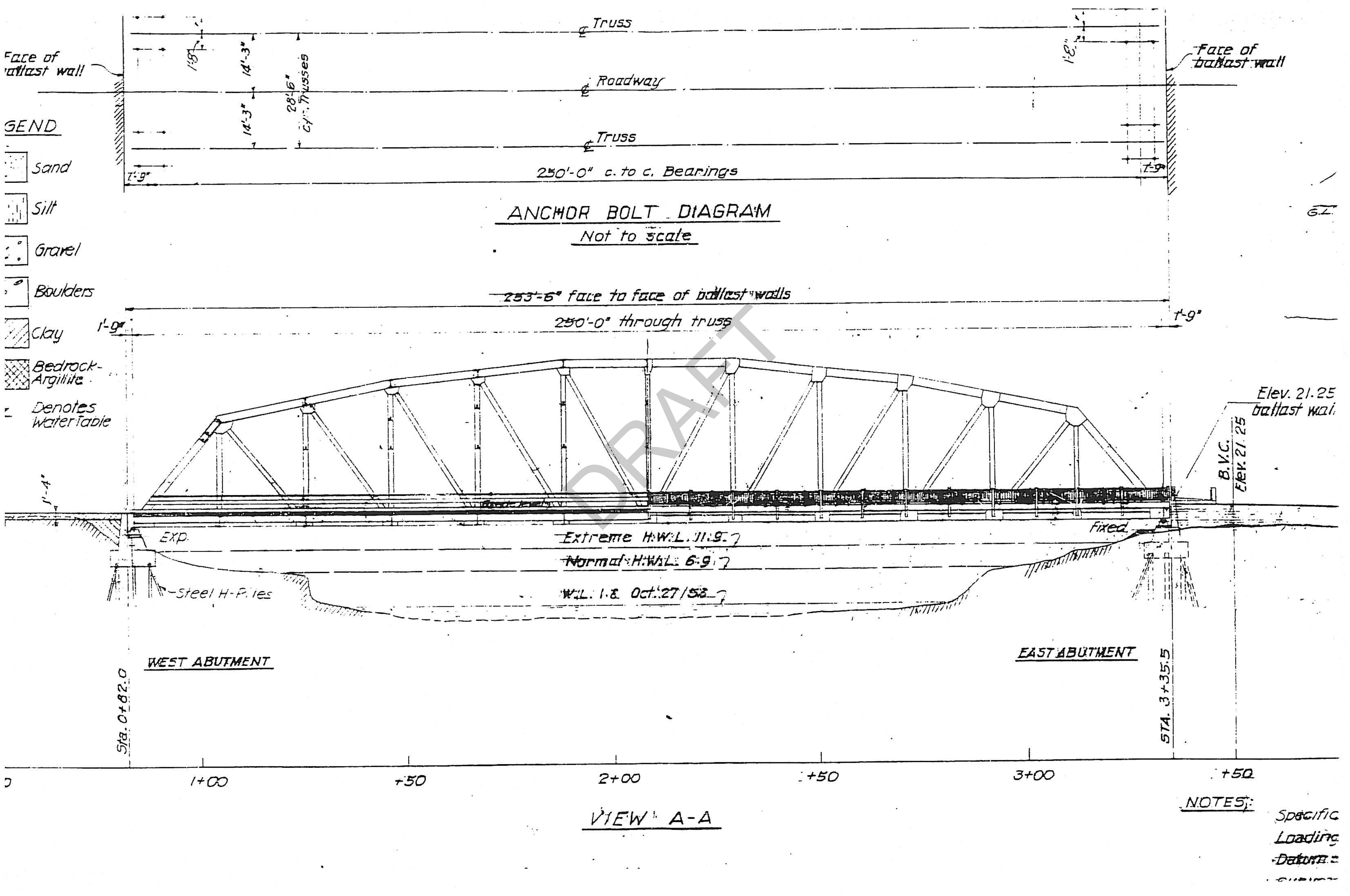
ITEM	UNIT COST	UNIT	QUANTITY	EXTENDED COST
Deck Overlay in 5 Years	$1,100	m^2	556	$709,012
Full-thickness repairs at the time of overlay (assume 10% of the deck area)	$1,650	m^2	56	$107,117
			Subtotal:	**$816,129**
Engineering and Contingencies @25%				$204,032
			Total:	**$1,020,161**

15.2 STEEL COATING

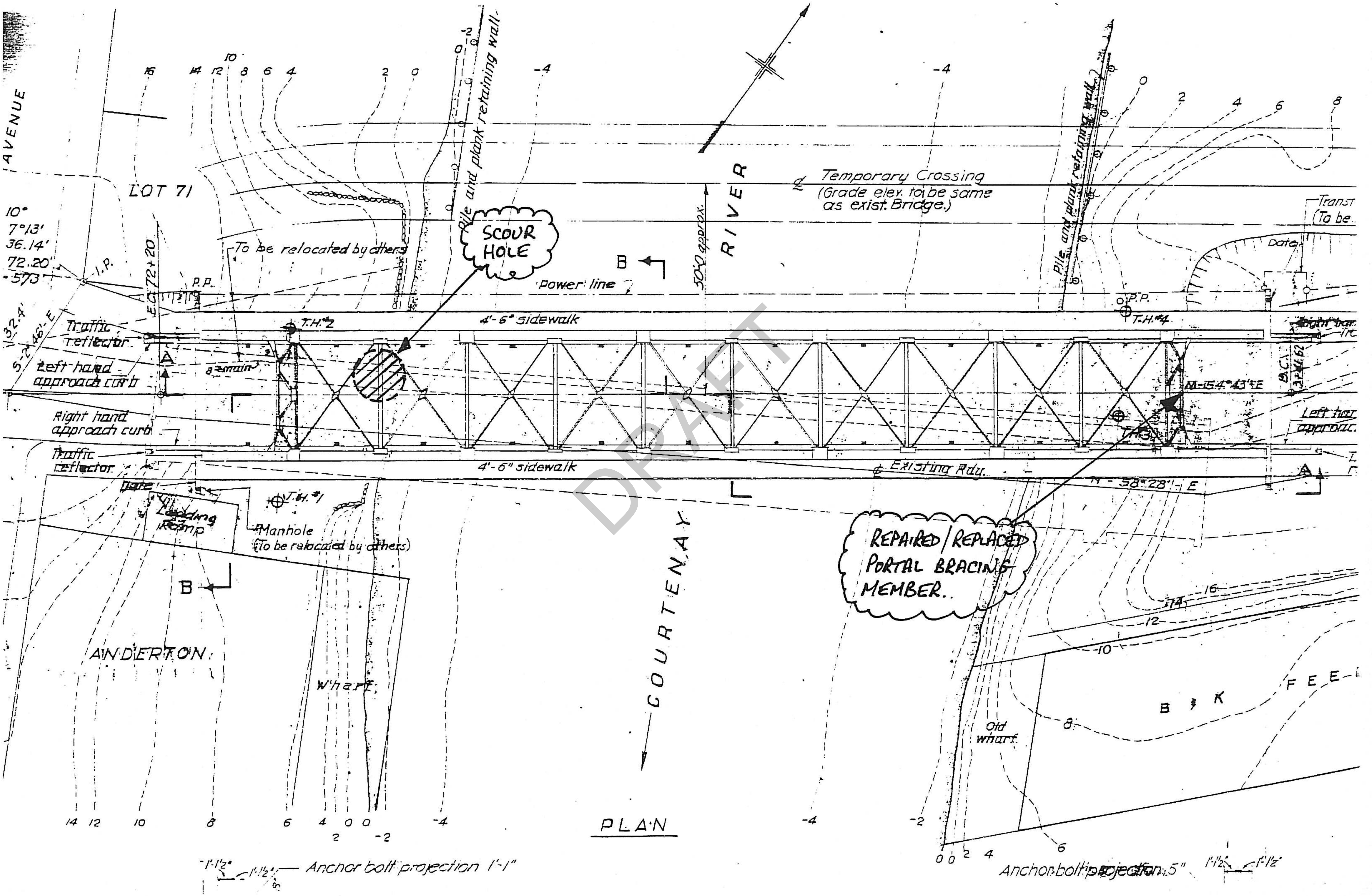
Very preliminary estimates of costs to clean, abate, and recoat the bridge a minimum of five years are in the order of $2.45 million to $3.26 million. This estimate should be considered at best within ± 30%.

Appendix A

GENERAL ARRANGEMENT



Truss
Roadway
Truss
Face of ballast wall
Face of ballast wall
14'-3"
14'-3"
28'-6" c/c Trusses
1'-8"
1'-8"
1'-9"
1'-9"
250'-0" c. to c. Bearings
ANCHOR BOLT DIAGRAM
Not to scale
GEND
Sand
Silt
Gravel
Boulders
Clay
Bedrock-Argillite
Denotes Water Table
253'-6" face to face of ballast walls
250'-0" through truss
1'-9"
1'-9"
1'-4"
Elev. 21.25 ballast wall
B.V.C. Elev. 21.25
Exp.
Fixed
Extreme H.W.L. 11.9
Normal H.W.L. 6.9
W.L. 1.8 Oct. 27/58
Steel H-Piles
WEST ABUTMENT
EAST ABUTMENT
Sta. 0+82.0
STA. 3+35.5
1+00
+50
2+00
+50
3+00
+50
VIEW A-A
NOTES:
Specific
Loading
Datum
DRAFT



AVENUE
LOT 71
Pile and plank retaining wall
RIVER
Temporary Crossing
(Grade elev. to be same as exist. Bridge.)
SCOUR HOLE
To be relocated by others
Power line
50'0 approx.
P.P.
T.H.#2
T.H.#4
T.H.#3
T.H.#1
4'-6" sidewalk
4'-6" sidewalk
Traffic reflector
Left hand approach curb
Right hand approach curb
Existing Rdy.
N-58°28'-E
N-15°4'43"E
REPAIRED/REPLACED PORTAL BRACING MEMBER.
Loading Ramp
Manhole (To be relocated by others)
COURTENAY
ANDERTON
Wharf
Old wharf
B K FEE
PLAN
Anchor bolt projection 1'-1"
Anchor bolt projection 5"
DRAFT

Appendix B

PHOTOS

Photo 1: North elevation.

Photo 2: South elevation.

Photo 3: Deck view facing east.

Photo 4: Deck view facing west.

Photo 5: Delaminations in eastbound lane near the west abutment.

Photo 6: Delamination in eastbound lane near the east abutment.

Photo 7: Transverse crack near chainage 58 m in eastbound lane.

Photo 8: Close-up of a typical transverse crack in the eastbound lane with epoxy.

Photo 9: Partial width transverse crack near chainage 25 m in the eastbound lane.

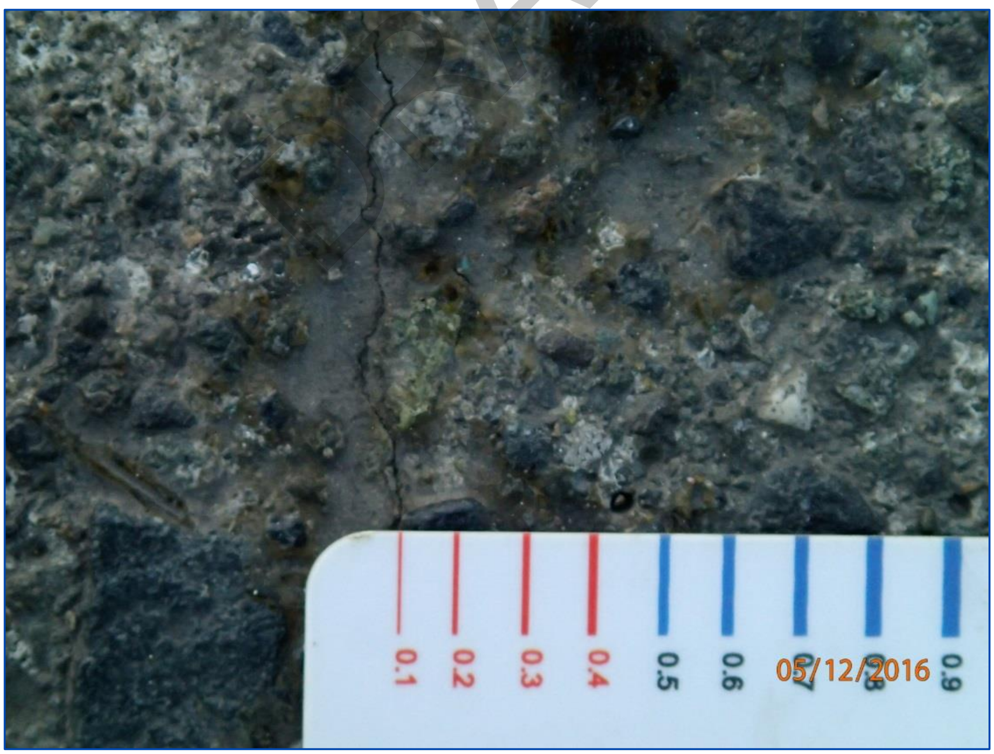


Photo 10: Close-up of Photo 9 showing 0.1 mm wide crack with failed epoxy.

Photo 11: Minor pop out in the eastbound lane near the east end of the bridge.

Photo 12: Moderate loss of paste resulting in exposed coarse aggregate in the wheel paths (typical).

Photo 13: Close-up of deck surface showing minor loss of paste (typical).

Photo 14: North curb and sidewalk facing east.

Photo 15: South curb and sidewalk facing east.

Photo 16: Close-up of Photo 15 showing 0.3 mm crack.

Photo 17: Minor impact damage to the north rail.

Photo 18: Condition of the drain (typical).

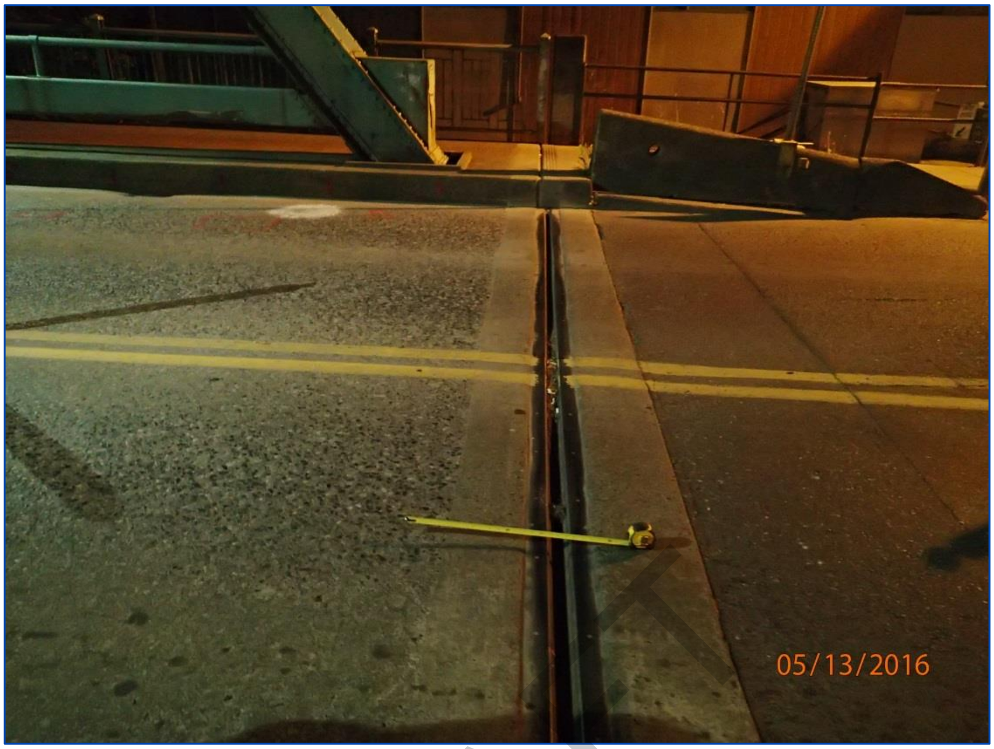


Photo 19: Expansion joint at the west abutment facing south.

Photo 20: Fixed joint at the east abutment facing north.

Photo 21: West asphalt approach.

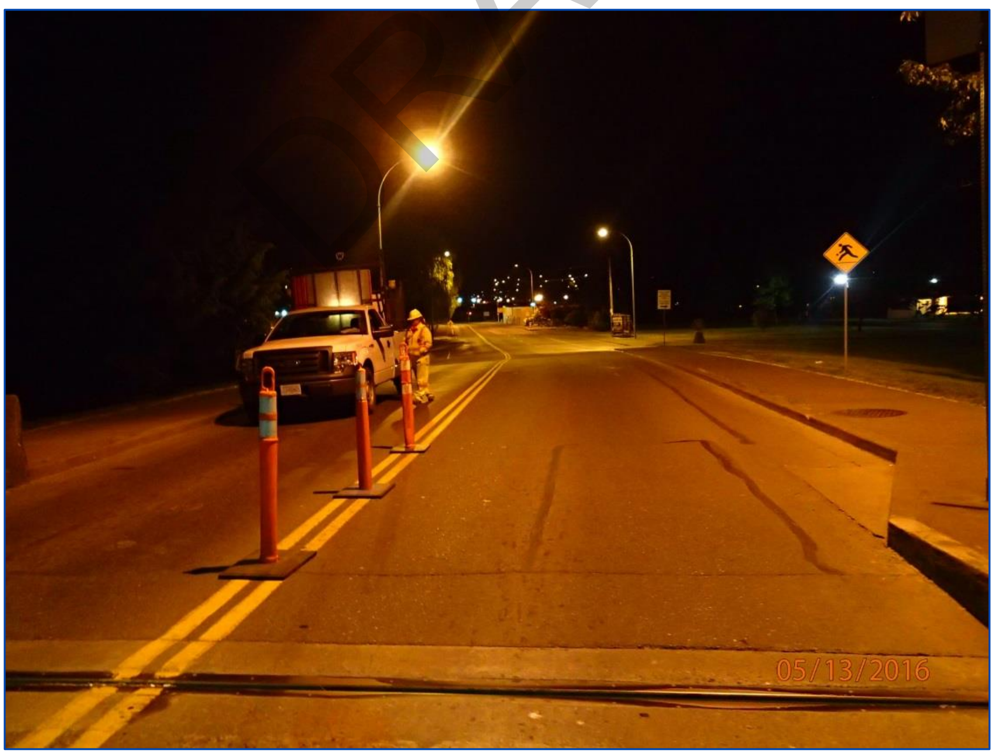


Photo 22: East asphalt approach.

Photo 23: Structural steel and deck soffit between the river shores facing west.

Photo 24: Structural steel and deck soffit between the river shores facing east.

Photo 25: Close-up view of flange edge at concrete soffit.

Photo 26: Soffit adjacent to the east abutment showing pattern cracking and efflorescence.

Photo 27: Soffit adjacent to the west abutment showing pattern cracking and efflorescence.

Photo 28: Soffit near the west abutment showing pattern cracking.

Photo 29: Spall due to low cover corroding rebar (Close-up of Photo 28).

Photo 30: Spalls due to low cover corroding rebar at the north sidewalk soffit adjacent the west abutment.

Photo 31: Close-up of Photo 30.

Photo 32: Low cover corroding rebar at the south curb soffit near the east abutment.

Photo 33: Northern half of deck soffit and sidewalk near the east abutment.

Photo 34: Southern half of deck soffit and sidewalk near the east abutment.

Photo 35: Leakage staining in the curb soffit adjacent to the vertical steel members (typical).

Photo 36: General view of the west abutment.

Photo 37: General view of the east abutment.

Photo 38: East abutment facing south.

Photo 39: South bearing of east abutment.

Photo 40: South bearing of west abutment.

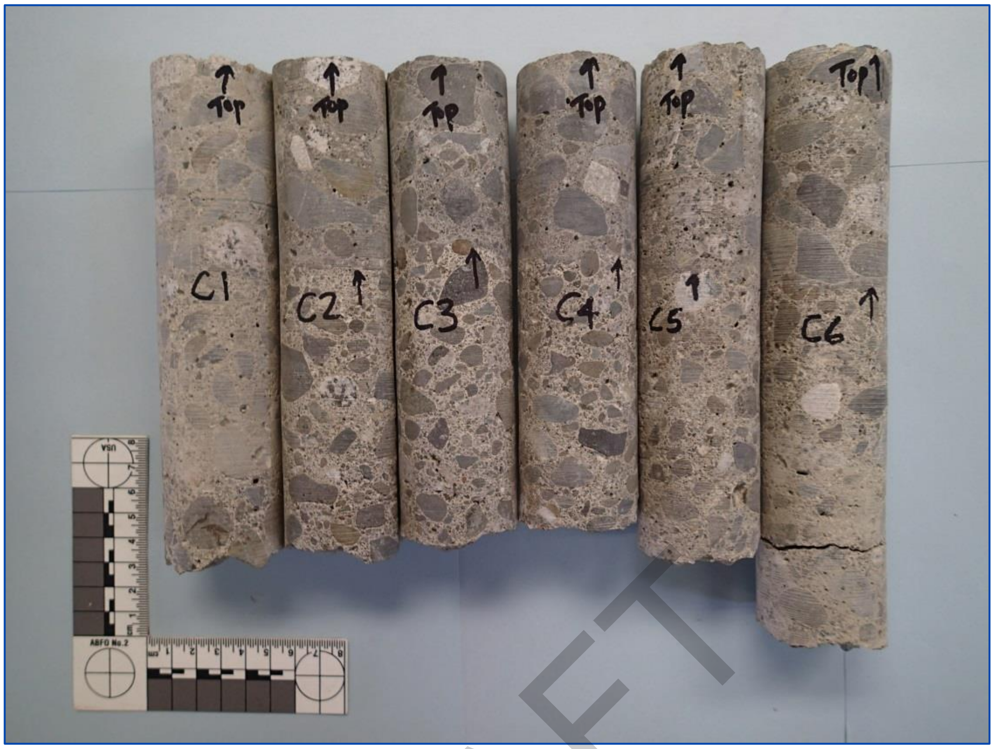


Photo 41: Extracted cores for chloride ion determination.

Photo 42: Condition of bottom mat longitudinal bar at Core 4.

Photo 43: Condition of transverse wire mesh at Core 6.

Photo 44: Condition of top mat of rebar at Core 8.

Photo 45: General view of south end.

Photo 46: Oblique view of top chord at southeast corner.

Photo 47: Oblique view of top chord at southwest corner

Photo 48: Typical transverse bracing near southwest corner of bridge.

Photo 49: Close-up view of coating failure on webs of built up elements

Photo 50: General view of truss “hanger” at approximately mid-span on the west (upstream) side. Note the corrosion at the toe of the angle.

Photo 51: Close-up view of typical stringer showing corrosion of flange edge at concrete soffit.

Photo 52: Oblique view of diagonal bracing in truss, east side.

Photo 53: Oblique view of diagonal brace, west side.

Photo 54: Close-up view of lattice in Photo 9, showing failure of green vinyl topcoat.

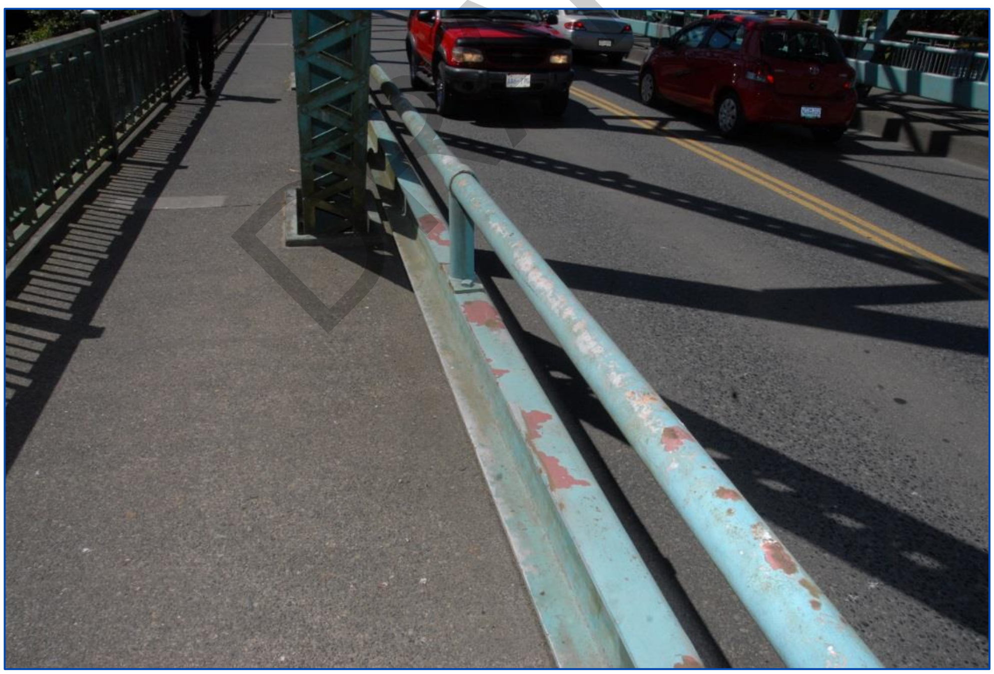


Photo 55: Oblique view of traffic lane railing/rub rail as seen looking approximately south.

Photo 56: Typical pedestrian railing, east (downstream) side.

Photo 57: General view of deck framing as seen looking south.

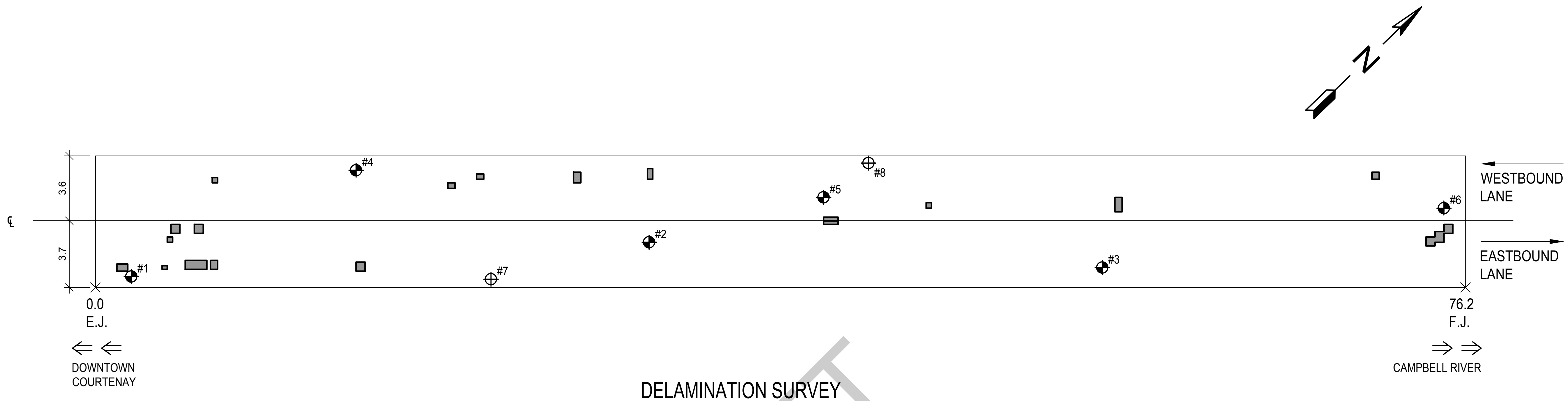
Photo 58: Typical touch-up coating, east side

Photo 59: Services, west side.

Photo 60: East side soffit below deck.

Appendix C

DELAMINATION SURVEY



DELAMINATION SURVEY

SCALE: 1:250

NOTES:

BRIDGE SCALE: METERS

YEAR BUILT: 1960

DATE TESTED: MAY 12, 2016

WEATHER: CLEAR, ≈20°C

LEGEND:

- DELAMINATED CONCRETE
- DELAMINATED PATCHES
- SOUND PATCHES
- ⊕ CORE SAMPLE
- CHLORIDE SAMPLE
- E.L. EXPANSION JOINT
- F.J. FIXED JOINT

Lane	Deck Area (m²)	Delaminations (excluding patches)		Delaminated Patches		Sound Patches		Total Delaminations (including delaminated patches)		Total Patches and Delaminations	
		Count	Area (m²)	Count	Area (m²)	Count	Area (m²)	Area (m²)	%	Area (m²)	%
Westbound Lane	274.3	9	1.5	0	0.0	0	0.0	1.5	0.5	1.5	0.5
Eastbound Lane	281.9	10	2.9	0	0.0	0	0.0	2.9	1.0	2.9	1.0
Total	556.3	19	4.4	0	0.0	0	0.0	4.4	0.8	4.4	0.8

No.	Description	Date

WSP

WSP CANADA INC.
150-12791 Clarke Place
Richmond, B.C.
Tel: 604 278-1411
Fax: 604 278-1042

TITLE: DELAMINATION SURVEY

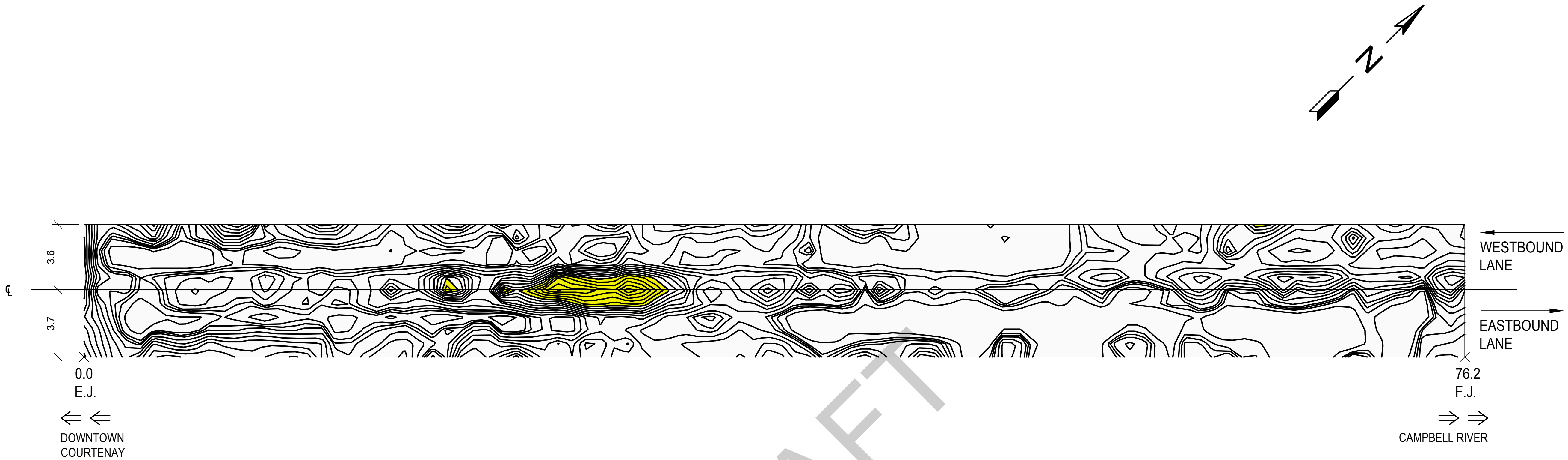
PROJECT: 5TH ST. BRIDGE

CLIENT: HATCH MOTT MACDONALD (HMM)

DES: MXA	DRWN: CWH
CH. MXA	SCALE: 1:250
APP. MXA	DATE JUNE 13, 2016
FILE NO. 161-05488-00	
DWG. NO. DWG 01	

Appendix D

HALF-CELL SURVEY



HALF-CELL SURVEY

SCALE: 1:250

NOTES:

BRIDGE SCALE: METERS

YEAR BUILT: 1960

DATE TESTED: MAY 12, 2016

WEATHER: CLEAR, ≈20°C

LEGEND:

E.J. EXPANSION JOINT

F.J. FIXED JOINT

Probability of Corrosion	Potential Range mV $_{CSE}$	Percentage of Deck Area
10%	More +ve than -199	96.0
Uncertain	-199 to -350	4.0
90%	More -ve than -350	0.0

No.	Description	Date

WSP

WSP CANADA INC.
150-12791 Clarke Place Richmond, B.C.
Tel: 604 278-1411
Fax: 604 278-1042

TITLE: HALF-CELL SURVEY

PROJECT: 5TH ST. BRIDGE

CLIENT: HATCH MOTT MACDONALD (HMM)

DES: MXA	DRWN: CWH
CH. MXA	SCALE: 1:250
APP. MXA	DATE JUNE 13, 2016
FILE NO. 161-05488-00	
DWG. NO. DWG 02	