BRITISH COLUMBIA

February 03, 2021

Lesley Ballman
Executive Director, Major Projects and Alternate Procurement
Ministry of Transportation and Infrastructure
Major Projects, Infrastructure and Properties

Dear Lesley:

Re: George Massey Crossing Project – Use of Existing Immersed Tube Tunnel

In my role as Chief Engineer I have been asked to review the feasibility of re-using the existing George Massey Immersed Tube Tunnel as part of the new George Massey Crossing. Several items were considered as part of the review:

- Remaining service life;
- Geometrics (lane widths, shoulder widths, vertical clearance);
- Fire/Life/Safety/Ventilation;
- Seismic resiliency;
- Condition of the existing tunnel.

Regarding remaining service life, the Canadian Highway Bridge Design Code (CSA S6:19) and the AASHTO LRFD Road Tunnel Design and Construction Guide Specifications, 2017 are the primary documents the Ministry utilizes for setting tunnel design standards. These documents only provide broad guidance regarding the level of remaining service when retrofitting existing tunnels. Therefore, a jurisdictional review of other recently rehabilitated immersed tube tunnel structures around the world was also conducted to determine the service life these tunnels were retrofitted to. From the design guides and jurisdictional reviews, it was determined that a service life of at least 50 years would be acceptable for the re-use of the existing tunnel in the new crossing.

The next three items from the list above were determined to be acceptable if specific upgrades were undertaken (replacement/upgrading of fire suppression/ventilation/lighting, ground improvements to increase seismic resiliency etc.). However, a significant issue was identified during the investigation of the existing tunnel condition.

As part of a detailed tunnel condition review undertaken by COWI (George Massey Crossing Assessment – Existing Tunnel Condition Assessment Report, December 11 2020), sampling and testing of the precast concrete tunnel sections and cast-in-place

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concrete tunnel joints revealed a condition known as Alkali-Silica Reaction (ASR). This condition occurs when certain types of aggregates react with the cement and causes deterioration of the concrete.

The recent analysis concluded that the level of deterioration due to ASR is not an immediate safety concern. However, due to minimal historical data (detailed concrete testing of the George Massey Tunnel has only occurred once, back in 2000), the rate of concrete deterioration of the tunnel due to ASR cannot be determined.

To calculate the rate of ASR deterioration, additional testing over a three to five year period would be required to establish a deterioration rate and extrapolate the rate of progression over the next several years. Without this information, it is not possible at this time to determine if the existing George Massey Tunnel has a remaining service life of 50 years.

Noting the current timelines for the George Massey Crossing Project , the uncertainty of the remaining service life of the existing tunnel, the length of time it would take to determine that service life, and the probability that the additional required testing may reveal the existing tunnel to have less than a 50 year service life, it is recommended that the existing George Massey Tunnel not be considered for re-use as part of the new George Massey Crossing.

To ensure the existing tunnel remains serviceable until the new crossing is built, it is recommended to implement the recommendations from the COWI report in relation to future tunnel inspections and ASR testing to determine the rate of concrete deterioration. It is also recommended to undertake the rehabilitation work in the COWI report to address immediate short term issues that were identified during the condition review.

The COWI report will be provided to the Structural Asset Management Group to ensure these recommendations are integrated into the Structural Rehabilitation Program.

In addition, once a new George Massey Crossing has been established, it is recommended to remove the existing George Massey Tunnel as this structure does not meet current seismic standards and has the probability for displacement during a major seismic event, which could affect the integrity of the dyke system along both sides of the Fraser River.

If you have any questions about these recommendations or require further details or clarification, please do not hesitate to contact me at any time.

Yours truly,

Ian Pilkington, P. Eng
Chief Engineer

cc: Donald Trapp, Executive Director, Project Management, TI Corporation
Ed Miska, A/ADM, Highway Services, MoTI
Kevin Volk, ADM, Major Projects, Infrastructure and Properties, MoTI
Prerna Sohal, Manager, Structural Asset Management, Engineering Services, MoTI

Attachment: COWI Report: George Massey Crossing Assessment – Existing Tunnel Condition Assessment Report, December 11, 2020

DECEMBER 2020
TICORP

GEORGE MASSEY CROSSING ASSESSMENT – EXISTING TUNNEL

CONDITION ASSESSMENT REPORT - DRAFT

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CONDITION ASSESSMENT REPORT - DRAFT

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0C	11 Dec 2020	Draft	Brad J. Pease, Ph.D. Anne-Marie Langlois, P.Eng.	Neil Cumming, P.Eng. Carola Edvardsen	Darryl Matson, P.Eng.

CONTENTS

APPENDICES

1 Executive Summary

As part of the Business Case development for the George Massey Crossing Project, the Transportation Investment Corporation (TICorp) asked the COWI-Stantec Team (CST) to complete a preliminary Condition Assessment of the concrete structural components of the existing George Massey Tunnel (GMT). The objective of the Condition Assessment was to establish whether the structural components of the 61 year old tunnel have a remaining service life of at least 50 years. Potential maintenance and, if needed, rehabilitation requirements needed to achieve the additional service life duration were also to be briefly described.

A previous condition assessment of the structural reinforced concrete in the approach ramps and tunnel was completed in 2000. The current study, as well as the one carried out in 2000, included limited visual inspections and some laboratory testing, however neither included a detailed inspection of the tunnel. Similar methodologies and a similar extent of sampling were completed for both studies, and current results were compared to those previously recorded to assess whether any trends in the deterioration processes could be identified.

The study presented herein has identified two primary modes of deterioration in the structural reinforced concrete components of the tunnel: reinforcement corrosion and alkali-silica reactions (ASR). In addition, leaks through cracks and/or joints in the concrete were observed both in the immersed tunnel and the approach ramps, which has implications on durability. Of secondary importance, observations indicate freeze/thaw attack is likely a participatory deterioration mechanism in the approach ramps.

Corrosion of reinforcing steel embedded in concrete develops tensile stresses in the concrete due to the expansion of corrosion products ("rust" takes up more volume than the original steel). This results in concrete cracking, delamination and spalling, which exposes the reinforcing steel and accelerates the corrosion process.

The current corrosion and leaking situations are considered manageable through rehabilitation, injection of cracks and construction joints and continued maintenance, and the future rate of increase in both can be monitored through inspections. Based on the current level of corrosion damage, CST believes that with proper maintenance, the corrosion damage in the existing tunnel and approaches can be managed to achieve an additional 50-year service life.

Note that corrosion-induced spalling of concrete was observed from the soffit of the Northbound Roadway Tube, which presents a potential safety concern for the travelling public (the spalling concrete can fall into traffic). This should be addressed, and a Detailed Condition Inspection should be completed to assess whether additional areas of the soffit are delaminated or otherwise at risk for spalling.

Alkali-silica reactions can occur in concrete when aggregates of a certain chemical composition are used, potentially leading to internal damage to the concrete. This phenomenon was not well understood when the tunnel was built. The ASR reaction takes time to develop, and once it starts, it is not possible to stop (it can be slowed by removing the moisture from the concrete; however, this is not practical for an immersed tunnel). As such, the rate of progression of the ASR is a critical factor in being able to determine the remaining life of the existing tunnel.

CST understands that annual routine inspections of the existing tunnel have been carried out regularly, but detailed inspections have not. This resulted in limited information being available to CST regarding the rate of change in the deterioration levels of the existing tunnel, and in particular the rate of change of the ASR deterioration. While the deterioration mechanisms acting on the reinforced concrete components of the existing immersed tunnel and approach ramps are identified herein, the extent of damage and the rate of change of the damage resulting from these processes could not be determined.

The current level of ASR damage is not critical to the integrity and safety of the structure. However, due to a lack of historical information about the levels of ASR damage in the tunnel, CST is unable to predict the future expansion rate of ASR damage and its effect on the structural properties of the concrete. Future ASR damage might progress slowly enough to allow the tunnel to remain in service for an additional 50 years, however at this time it is not possible to confidently determine if this will be the case. If detailed inspections were carried out at regular intervals over the next 5 to 10 years, it is possible that the rate of ASR deterioration could be predicted, and the expected remaining life of the tunnel could be determined. However, without this information, it is not possible to determine if the tunnel has 50 years of remaining service life.

2 Introduction

In July 2019, COWI North America Ltd. and Stantec (the COWI-Stantec Team [CST]) were awarded an assignment to provide as and when technical services to the BC Ministry of Transportation and Infrastructure (Ministry) for the George Massey Crossing Project. The assignment was to provide technical support in response to questions asked by the Ministry (or the Mayors Task Force through the Ministry). Importantly, CST was specifically not to provide recommendations.

CST provided technical information to the Ministry, presented in a draft report in December 2019, and included a conceptual design for the Immersed Tube Tunnel (ITT) option and for two long-span bridge options. In February 2020, Transportation Investment Corporation (TICorp) initiated the development of a business case for the George Massey Crossing Project.

TICorp has asked CST to perform a preliminary Condition Assessment of the tunnel's structural reinforced concrete. In accordance with [2], the basic objectives of the preliminary Condition Assessment were to:

1) Establish an understanding of the likely progression of active deterioration process (likely corrosion and alkali-aggregate reaction (AAR)) of the existing tunnel's concrete structures during a specific time horizon. This time horizon is to be defined with TI Corp.

2) Propose a potential maintenance and (if needed) rehabilitation regime anticipated to achieve the time horizon.

The existing GMT was constructed between 1957 and 1959 and has been in service for 61 years, since May 1959. TICorp's specific time horizon is to maintain operation of the existing GMT for roadway traffic for an additional 50 years.

To ascertain the anticipated longevity of the structural reinforced concrete, a Condition Assessment Testing and Sampling Plan was developed by CST [3].

The Condition Assessment herein focuses primarily on the immersed precast concrete tunnel elements. Reinforced concrete components of the approach ramp structures are more easily accessible for maintenance and rehabilitation works and therefore not all components were assessed. In particular, the reinforced concrete struts (also referred to as T-beams) in the light attenuation structure are assumed to be replaceable components and were excluded from this assessment.

The focus of this report is to assess the condition of the structural reinforced concrete in the immersed tunnel and approach ramps and to establish an understanding of likely deterioration progression in a subsequent 50 years of service. It is however noted that a critical factor, outside of this focus of the study presented herein, but nonetheless significant in the consideration of remaining service life of any structure, is the controlling condition or situation that is considered (by the Ministry) to signify the end of the service life of the structure. In general, the service life duration of a structure may be greatly influenced by the definition of the 'end of service life', as well as the extent to which an appropriate maintenance and rehabilitation plan is funded and carried out. Further, additional factors beyond the condition of the structural reinforced concrete should be subject to due attention. Typical limit states to mark the end of service life of existing structures might include (to name but a few):

- The point of time at which the structure can no longer reliably resist applied loads (including extreme loads, e.g. seismic) based on structural evaluations,
- Inability for the structure to maintain some minimal functional requirement, and/or
- The costs and burden of continued maintenance needed to manage deterioration reaches a threshold level that is considered untenable.

The main report is divided into two parts, based on the main components inspected, namely:

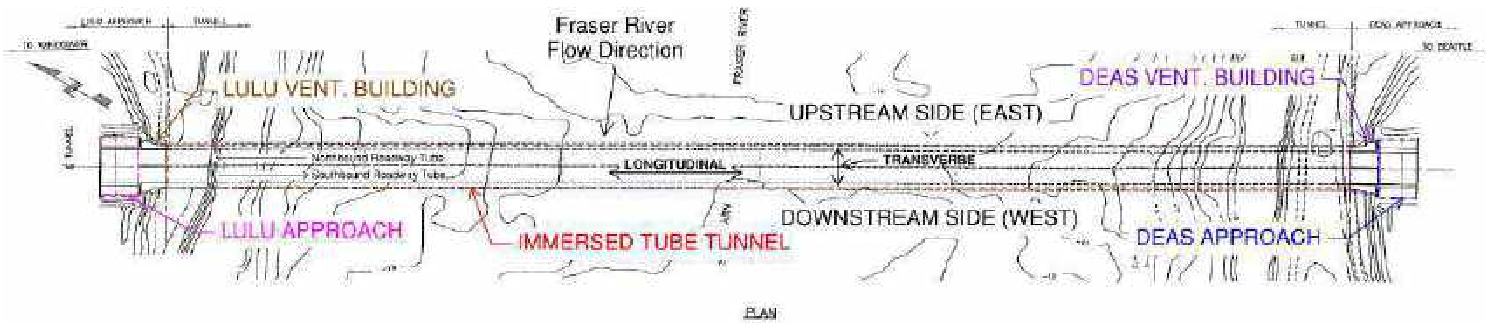
- Immersed Tunnel; and
- Approach Structures.

Drawings of the existing GMT are utilized and referenced herein. Referenced drawings are identified by drawing number.

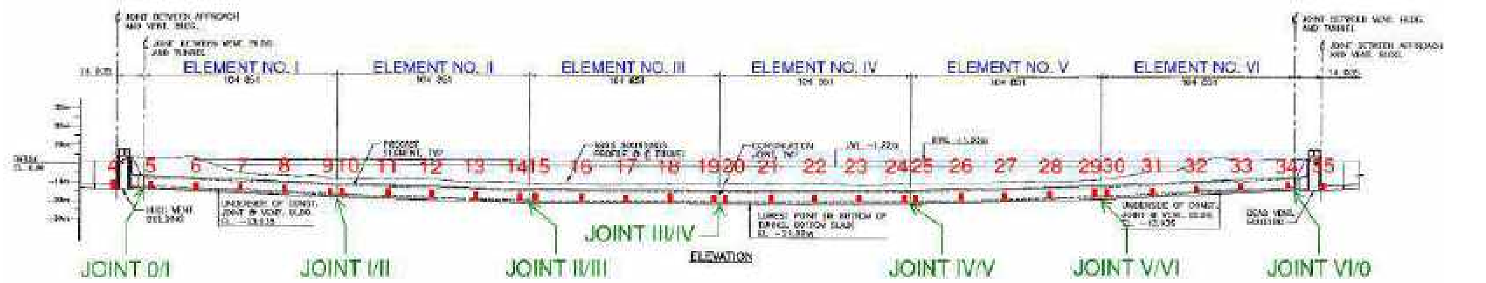
The following subsections to this Introduction provide aids to the reader, including pertinent background information on the structure, past inspections, and brief technical descriptions of the reinforced concrete deterioration mechanisms discussed within the body of this report.

2.1 Orientation and Terminology

For orientation, the following figures provide definitions of various components and locations in the existing GMT. Figure 2-1(a) illustrates an overall orientation map and defines the extents of the immersed tunnel, ventilation buildings and approaches and provides orientation including upstream (east) and downstream (west) sides of the tunnel. Figure 2-1(b) illustrates numbering systems used for location identification herein. The tunnel element numbering is provided in blue, infill joint numbering in green, and emergency door numbering in red. Element and door numbers are used throughout to indicate locations of observations and sampling.



(a)



(b)

Figure 2-1: Orientation maps of the existing GMT: (a) A plan view of the tunnel, indicating the upstream and downstream sides and extents of the various components of the structure and (b) An elevation view of the structure, with tunnel element numbering system given in blue, joint numbering system given in green and emergency door numbering system given in red. Emergency door locations are indicative.

Figure 2-2 defines terminology applied to the four tubes in the immersed tunnel.

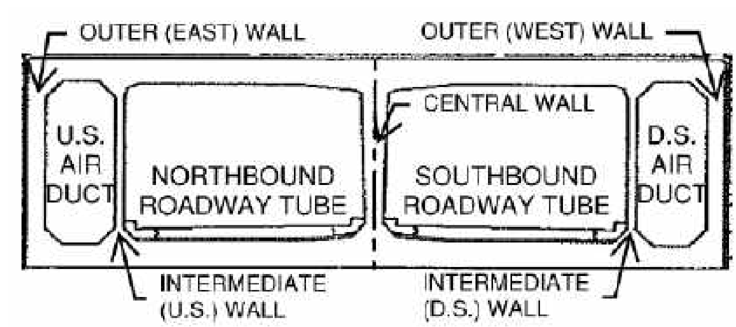


Figure 2-2: Typical Cross Section of the Existing GMT, as observed from the north, with the Upstream (U.S.) and Downstream (D.S.) Air Ducts and Northbound and Southbound Roadway Tubes labeled.

The emergency doors are located in the central wall, between the Northbound and Southbound Roadway Tubes, with numbers visible on each door.

The joints between the individual tunnel elements (i.e., element joints) are also readily visible within the tunnel as shown in Figure 2-3. This report refers to element joints as "infill joints" which were realized by cast-in-place infill joint concrete as described further in Section 3.1.1. The six individual tunnel elements were precast in a dry dock.

The 'bow-tie' shaped galvanized steel plates at the joints are not original to the structure and were installed between 2004-2006 on internal surfaces of the top and base slab in the roadway tubes and outer walls of the immersed tunnel as part of a seismic retrofit project [7]. The plates have therefore been exposed within the immersed tunnel for between 14-16 years.

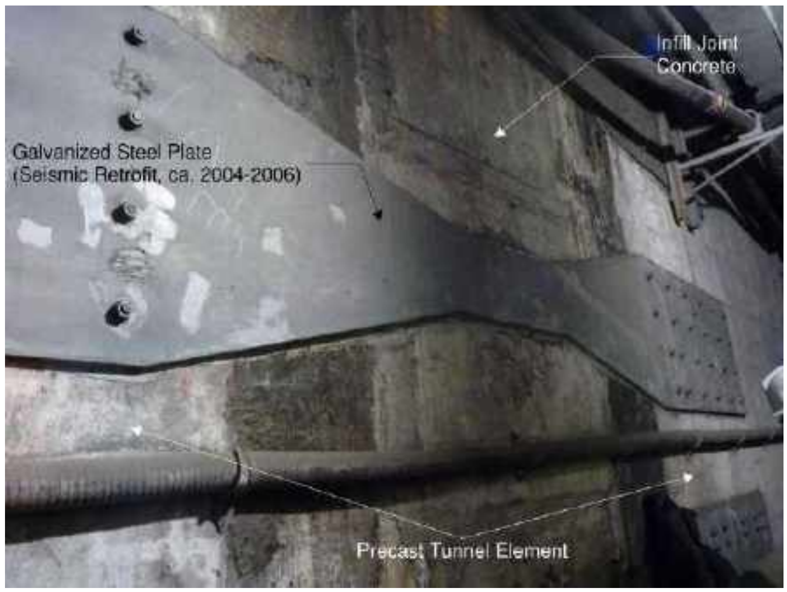
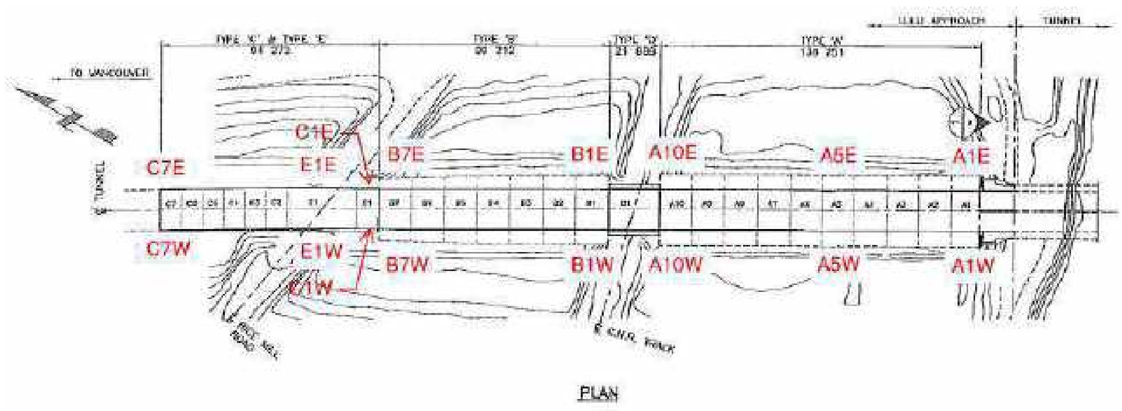
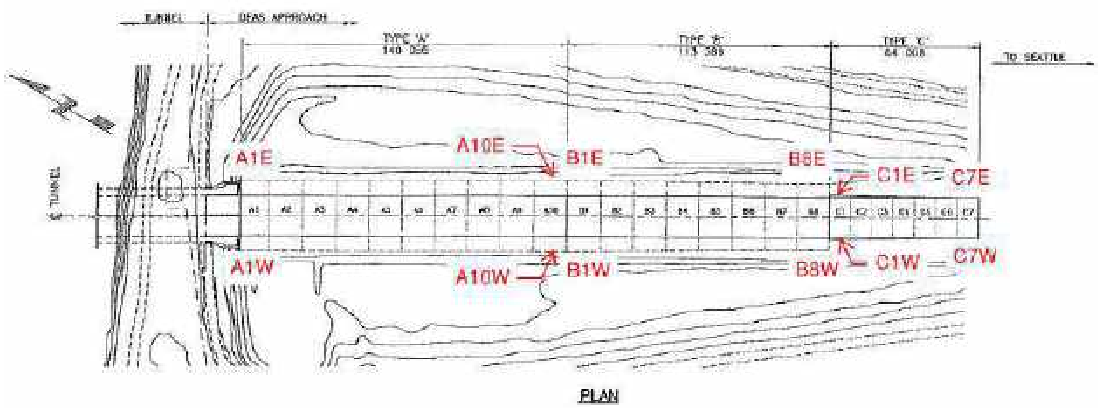


Figure 2-3: Typical example of joint location in outer wall of air duct tubes.

Figure 2-4 provides a numbering system for the approach structures, with A-type elements comprising the light attenuation structure with central partition wall, B-type elements comprising retaining structures with a base slab and external walls, and C-type elements comprising a base slab and curbs. A-type elements are the deepest portions of the ramp with B- and C-type used in progressively shallower sections of the approach ramps. Type D and E elements in Figure 2-4(a) represent the rail bridge and Rice Mill Road Overpass, respectively.



(a)



(b)

Figure 2-4: Plan views of the approaches providing numbering systems for components of (a) the Northern (Lulu) approach and (b) the Southern (Deas) Approach.

2.2 Overview of Previous Investigations and Inspections

Section 2.2 of [3] provides an overview of the investigations and inspections reported in the Condition Assessment in 2000 [1], 2018 Routine Condition Inspection report [4], and 2019 Routine Condition Inspection report [5], which is not repeated here. Detailed Condition Inspections appeared to not have been performed.

Key findings from the condition assessment performed in 2000 in the air duct tubes were [1]:

- Numerous cracks with widths in the range of <0.2mm to 0.6mm were observed in the external walls of both air ducts. Some cracks/joints were damp and weeping, with water leaking into the tunnel being brackish. Some cracks had been weeping in the past but at the time of inspection were plugged, in some cases by injection repairs, and various other causes for plugging leaks are theorized.

- There was evidence of tube movement.
- Corrosion-induced deterioration was reported as localized spalling of cover over corroding rebar in the outer walls in both air ducts.
- "*No significant concrete distress was identified. There were indications of early stage of alkali silica reactivity.*"
- Petrographic analysis of three cores from the infill joint concrete and precast tunnel concrete "*exhibited few signs of AAR*", with and uranyl acetate testing of the same cores showing "*an insignificant amount of AAR*" that was "*not considered to have been affected to a deleterious extent by Alkali-Silica Reaction*".
- Evidence showed that more than one source of aggregates was used.
- Measured concrete compressive strengths from cores between 34 and 51 MPa exceeded the design strength of 26 MPa (3,750 psi).
- This study recommended that remedial work be limited to conventional maintenance: injection of weeping cracks and patching of concrete spalls. It was expected that crack injection would be an ongoing process.

For the approaches, key findings were:

- Significant damage from alkali-silica reactions (ASR) was observed on the top portion of the approach ramp walls.
- No reinforcement corrosion was detected.

The 2000 Condition Assessment [1] reported findings from the BC Ministry of Transportation and Infrastructure (MoTI) archives that indicate leaks have been ongoing for a long time. Water was observed to be leaking through cracks in the air ducts as early as 1968. The flow of water was believed to be related to thermal movements, although crack measurements by MoTI were inconclusive. In 1984, water dripping from the ceiling was observed near the mid-river elements over the southbound slow lane at a construction joint. In 1999, trial epoxy injection of a weeping construction joint was completed and found to be only partially successful in stopping the leak, with water reported to continue leaking from the ceiling at the test site. It was concluded in 2000 that the concrete was exposed to diluted sea water with mildly aggressive sulphate and with chlorides at a level that could be contributory to reinforcement corrosion.

As reported in [1], various crack gauges were installed in the Air Duct Tubes. COWI was informed by TICorp that no data was available from these gauges. Gauges observed during the inspection reported herein were missing or damaged and no longer useable.

The 2000 Condition Assessment [1] did not include an estimate for a remaining service life of the structure.

2.3 Overview of Field Study and Sampling

For the purpose of the current study, a walk-through site inspection was conducted by Brad J. Pease, Neil Cumming, and Fatemeh Alapour from COWI between 27-30 July 2020 and on 23 September 2020.

Field testing and coring of concrete samples was completed to assess chloride ingress, carbonation depth, and petrographic analyses. The extent of the test program considered the work performed in 2000 [1], information included in the 2018 and 2019 routine condition assessment reports [4] and [5], and schedule and access constraints. Much of the sampling was intended to replicate analysis done in 2000 to allow assessment of any progression of adverse conditions.

All recorded field observations and original test reports are provided in the appendices as follows:

- Details of observations and obtained samples are provided in Appendix A;
- Reports from field testing, chloride profiles and carbonation depth measurements are provided in Appendix B; and
- Petrographic and damage rating reports are provided in Appendix C.

Eleven cores were obtained from the immersed tunnel; five in the Upstream Air Duct, and six in the Northbound Roadway Tube (which was closed to traffic for other maintenance activities during CST's field inspections). No cores or other samples were collected from the Southbound Roadway Tube, which was inaccessible for inspection. Similarly, the soffit of the top slab was also not subject to assessment in this investigation. Cores were not taken from the Downstream Air Duct due to access being limited to the ventilation building (which excluded access for the coring equipment). A series of powder samples were also collected from the air duct tubes obtained for chloride and pH profiles. Powder samples were collected by means of an impact drill with approximately ½" diameter and samples were collected in 15 mm increments.

It was considered to core through the entire thickness of the outer wall of the immersed tunnel to obtain samples of the outermost layer of reinforcement and the waterproofing membrane; however, this was ultimately excluded due to lack of specialty equipment locally and due to the risk of introducing permanent leaks into the tunnel.

A further three cores were collected from the approach ramps, with two obtained from the Northern (Lulu) Approach and one from the Southern (Deas) Approach. It is noted that at one coring location included in the original planned study provided in [3] a core was unable to be obtained. Core NAE-C7 was intended to be taken from the base slab of the Northern (Lulu) Approach Ramp. However, the thickness of the asphalt precluded collection of this core and an alternative location was selected as described in Table A-1.

2.4 Briefs on Discussed Reinforced Concrete Deterioration Mechanisms

The following deterioration mechanisms impacting reinforced concrete are discussed herein:

- Alkali-Silica Reaction (ASR);
- Reinforcement corrosion, induced by chloride and/or carbonation; and
- Freeze/thaw attack.

The following subsections provide short briefs on these deterioration mechanisms aimed to aid the layperson at understanding the basic processes, typical resulting damage, and overall significance of each deterioration mechanism.

It is noted that observations of water leakage discussed herein is not a concrete deterioration mechanism in itself; however, water leakage tends to accelerate deterioration mechanisms as discussed below.

2.4.1 Alkali-Silica Reaction

Alkali-silica reaction (ASR), one type of Alkali-Aggregate Reaction (AAR), is a chemical reaction between alkalis in the cement paste and reactive silica contained in some aggregate minerals. The product of this reaction is an expansive gel that can cause tensile stresses to develop in the concrete that may lead to cracking of the concrete. Moisture must be available for ASR to proceed and the reaction will cease when the internal relative humidity in the concrete drops below approximately 80%. Visible outward signs of extensive ASR include cracking of the concrete, often accompanied by a white gel precipitate on the concrete surface. Advanced cases of ASR can cause cracks in the concrete and reductions in strength (tensile and compressive) and elastic modulus.

Petrographic analysis of concrete cores, extracted from a structure thought to be suffering from ASR, is a common methodology to assess whether ASR is ongoing as it directly assesses the presences of potentially reactive minerals in the aggregate and visual signs of ASR (e.g., internal cracking, gel formation in the paste).

2.4.2 Reinforcement Corrosion

Reinforcement embedded in concrete is protected from corrosion by 1) a physical barrier against corrosion-causing substances in the form of the concrete cover provided and 2) an electrochemical protection as the high pH of concrete pore solution creates a passive film on the reinforcement surface that is highly resistant to corrosion. With time and exposure to chloride ions and/or carbon dioxide (CO_2) these protections can degrade, and reinforcement corrosion can occur.

Chloride ions from de-icing salt or brackish river water, in contact with the concrete surface, will slowly penetrate into concrete through various transport mechanisms like diffusion, capillary suction, and other processes. With time, the concentration and depth of penetration of chloride ions tends to increase and eventually a critical content is reached at the level of the reinforcement. At this critical content, the chlorides break down the passive film on the reinforcement surface and active corrosion can then occur in the presence of moisture and oxygen.

Carbonation occurs at concrete surfaces exposed to the atmosphere containing CO_2. Concrete carbonation is a reaction between CO_2 in the atmosphere and calcium found in cement hydration products. The product of this reaction (calcite ($CaCO_3$) lowers the pH of the concrete. The depth of carbonation progresses slowly into the concrete with time, and once the carbonation front reaches the depth of reinforcement, the reduced pH will allow the steel to corrode.

Areas of low concrete cover on the rebar will allow corrosion to initiate sooner than at areas of higher cover thickness. Cracks in the concrete can also accelerate the ingress of both chloride and carbonation. For tunnels and other underground structures specifically, leaks either from through-going cracks of construction joints are amongst the most critical durability concerns as corrosion of the transected rebars tends to initiate rapidly. Worsening the situation, the leaking water will typically wash away the corrosion products, leading to further corrosion of the rebar. As a result, leaking cracks are not desirable.

Corrosion of embedded steel reinforcement leads to local tensile stresses in the concrete as corrosion products expand compared to the initial steel volume. This process can result in cosmetic issues like formation of rust stains on the concrete surface and structurally significant issues like cracking, delamination, and spalling of the concrete cover and cross-sectional reduction of the corroding reinforcement.

Visual inspection and several non-destructive test methods, some of which are discussed further herein, are useful methods to detect reinforcement corrosion.

2.4.3 Freeze/Thaw Attack

Freeze/thaw attack is an attack on the concrete itself that involves cycles of freezing and thawing of moisture either contained within the concrete or on its surface. Freeze/thaw damage can include cracking of the concrete, scaling of the concrete surface, or both. Cracks caused by other mechanisms (e.g., ASR) can also be aggravated by freeze/thaw if these cracks allow ingress of water than subsequently freezes. Extensive freeze/thaw damage can lead to reduced mechanical properties (i.e., elastic modulus and strength characteristics) of the concrete.

To achieve a freeze/thaw resistant concrete it is common practice today to use freeze/thaw resistant aggregates, a concrete mix design with a low water/cement ratio to reduce water permeability, and to include air entrainment. Entrained air, introduced by air-entraining admixture, introduces a large numbers of small, closely spaced air voids that allows ice formation without corresponding stress build-up in the solid structure of the concrete.

3 Immersed Tunnel

3.1 General

3.1.1 Description of Structure

The existing George Massey immersed tunnel consists of six individual precast concrete tunnel elements with 104.85 m length as shown in Figure 2-1(b). As shown in Figure 2-2, the immersed tunnel consists of 4 individual tubes including two air duct tubes and two roadway tubes. Observations from the individual tubes inspected are provided in the following sections, noting that the Southbound Roadway Tube was not accessible for inspection.

Specified concrete cover thickness in the original as-built drawings (e.g., drawing No. 3-J-1007) of the precast concrete tunnel elements was:

- 1.5" (~38 mm) to main reinforcement; and
- 1" (~25 mm) minimum to all other bars.

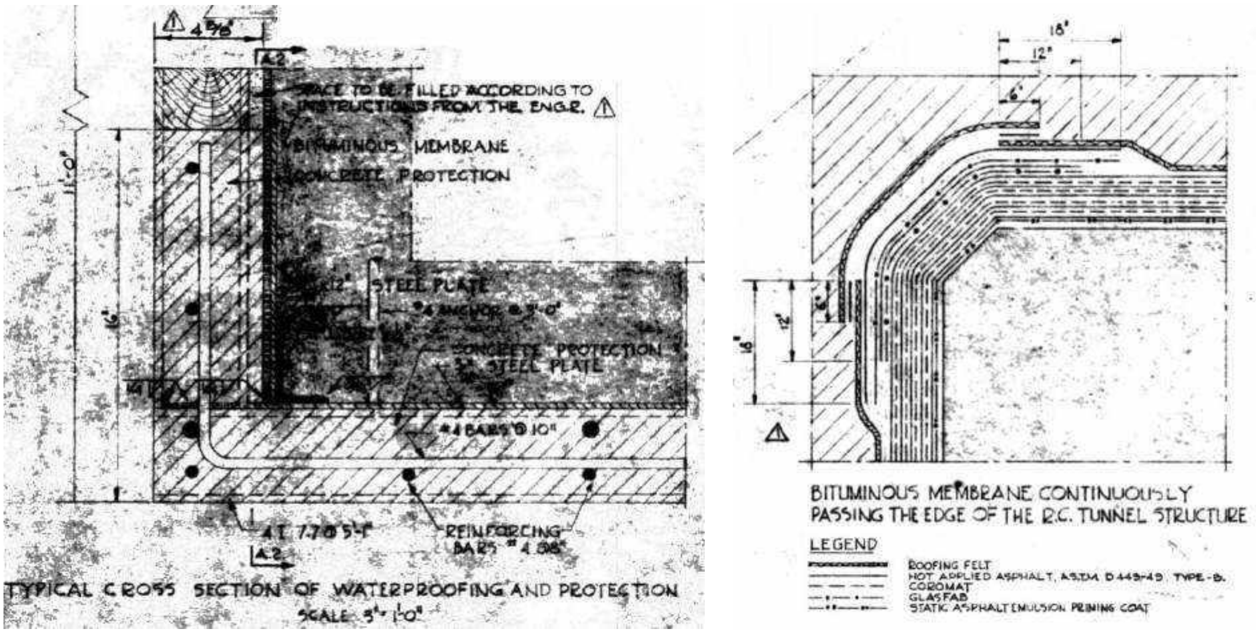
Measurements made in 2000 generally agree with design cover thicknesses [1].

The base slab of the elements was cast over a 5 mm thick steel membrane as shown in Figure 3-1(a), with a 100 mm layer of concrete protection covering (i.e., under) the steel membrane. The exterior surfaces of the external walls and the top slab were covered by a waterproofing system consisting of five layers of hot-applied asphalt with four layers of 'Coromat reinforcing membrane' and an outer layer of roofing felt as shown in Figure 3-1(c). Two additional layers of hot-applied asphalt was used at element corners, with the additional layers covered by 'Glasfab reinforcing membrane' under the outer layer of roofing felt. The membrane was covered by a 100 mm layer of protective reinforced concrete on the roof and by timber lagging on the walls as detailed in drawing 3-J-3048.

Drawing 3-J-3099 shows the precast tunnel elements were cast in 43'-5" (~13.2 m) long segments, with 6' (~1.8 m) gaps between individual segments (See Figure A-2). Horizontal construction joints in the segments were located between the outer walls and bottom/top slab as shown in Figure 3-2(a). The 6' gaps between individual sections were filled by a 'Contraction Pour', placed a minimum of 14 days after adjacent pours. Transverse construction joints[1] between the individual segments and contraction pours included a drain detail formed into the concrete as shown in Figure 3-2(b). The drain detail was continuous in the bottom slab, outer

[1] The original drawings refer to 'transversal construction joint'; however, the term 'transverse construction joint' is used throughout this report.

walls and top slab and connected to steel pipes protruding from the outer wall, located 3" above the lower chamfer in the air duct tubes.



(a) *(b)*

Figure 3-1: Details of the multiple layer waterproofing system from Drawing 3-J-1002 for the precast concrete tunnel elements showing (a) the base slab and exterior wall corner and (b) exterior wall and top slab corner.

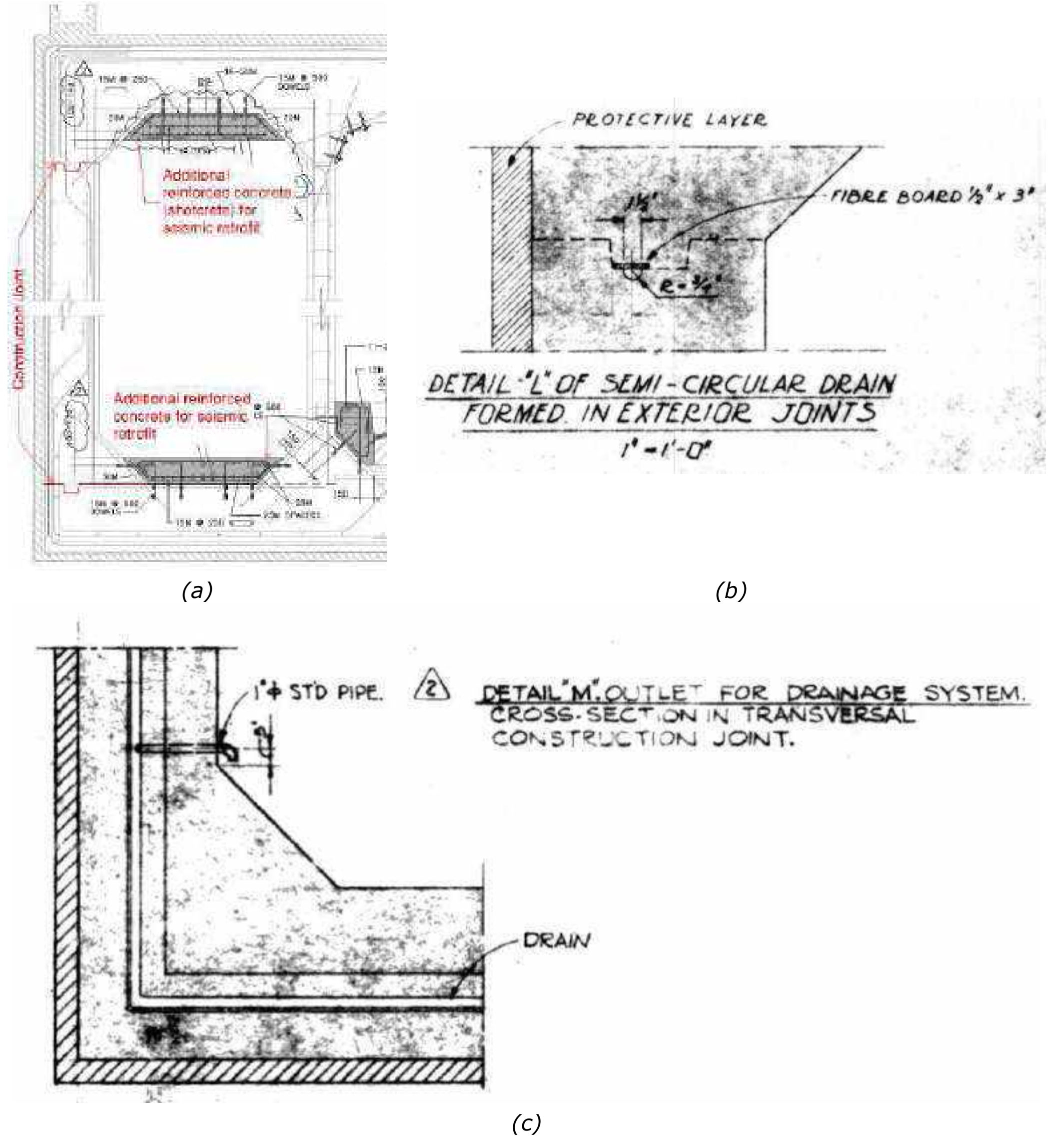
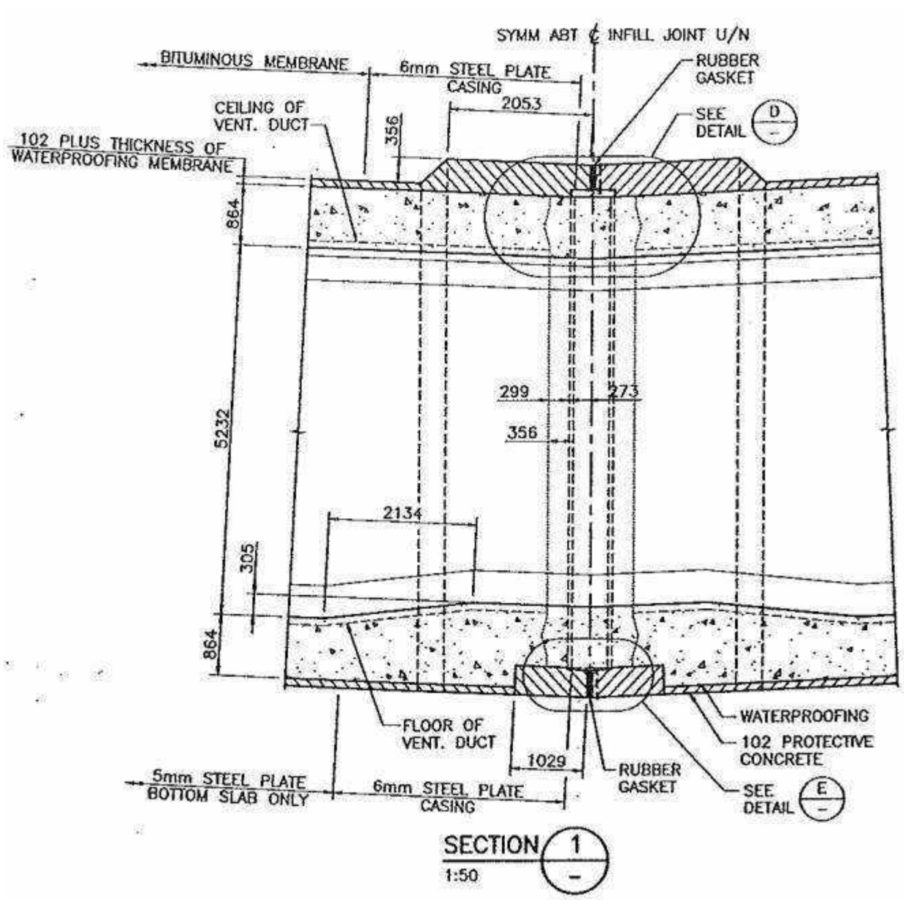


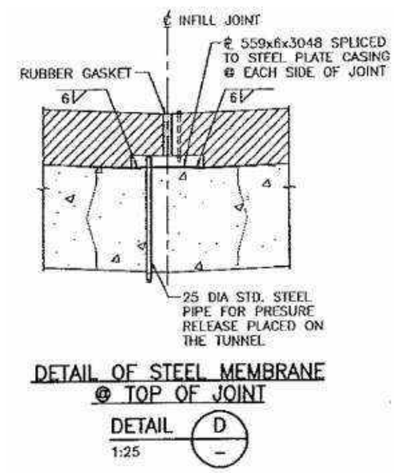
Figure 3-2: Construction joint details in precast concrete tunnel elements including (a) location of construction joints in outer walls (shown in red) from drawing 1509-39 (location of reinforced concrete added in air ducts as part of seismic retrofit is shown in grey), (b) drain detail used in the transverse construction joints between segment and contraction pours from drawing 3-J-3099, and (c) outlet pipe for drainage system in transverse construction joints.

Connection of individual precast tunnel elements was achieved by cast-in-place infill joints at the base slab and walls after element immersion [6]. According to drawings 3-J-1512 and 3-J-1516, the top slab at the infill joint was realized by preplacing aggregate and injection with grout under pressure, a detail that is no longer visible due to added concrete layer shown in Figure 3-2(a) during seismic retrofit. As shown in Figure 3-3, the multi-layer bituminous membrane transitions to a 6 mm thick steel collar at the each end of the tunnel elements and a rubber gasket was affixed to the protective concrete at the joints. Compression of the rubber gasket was initially achieved by a hook and anchor system [6] and a final seal achieved by hydrostatic pressure. Per drawings 3-J-1512 and 3-J-1516, waterproofing steel sheets connected the adjacent collars by a continuous watertight weld and the infill concrete was then placed. A steel sheet waterstop, shown in drawing 3J-1015, was provided between the precast and infill joint concretes.

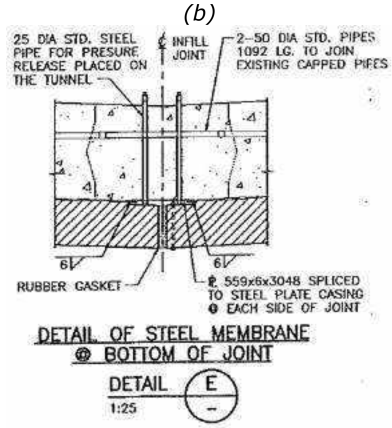
Figure 3-4 shows the reinforcement detailing at the infill concrete joints between individual elements. The longitudinal reinforcement was welded between the individual tunnel elements and at Joints 0/I and VI/0 (i.e., joint between tunnel and ventilation buildings, see Figure 2-1), indicating that effectively the immersed tunnel is a longitudinally continuous monolithic structure (ca. 660 m long) from the northern to the southern ventilation buildings. As shown in Figure 2-3 galvanized steel plates that were installed as seismic retrofit between 2004-2006 [7]. The steel plates were placed on the outer walls, soffit and base slab from Joint 0/I to Joint IV/0 [7]. As part of this seismic retrofit, an additional 300 mm thickness of structural reinforced concrete was placed over the original base slab top surface and top slab in the air duct tubes as shown in Figure 3-2(a).



(a)

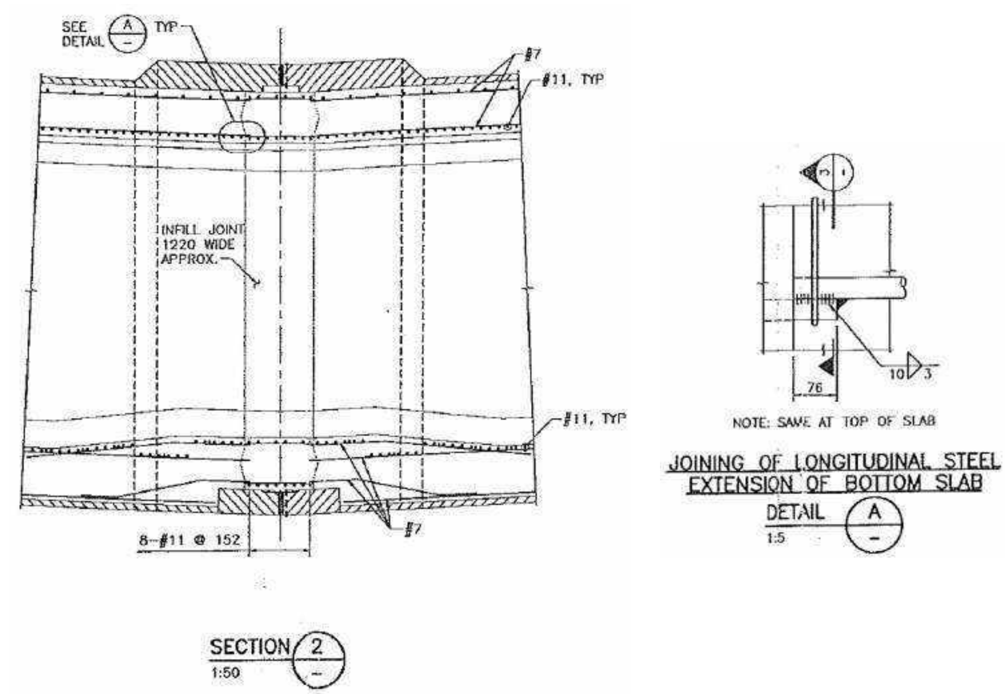


(b)



(c)

Figure 3-3: Waterproofing details at the infill joints for the immersed tunnel showing (a) the general arrangement of joints, (b) detail of steel membrane and infill *concrete dimension at top of joint, and (c) detail of steel membrane and infill concrete dimension at bottom of joint. * The added galvanized steel "bow-tie" plates at joints were not part of the original design and are therefore not shown in these details.*



(a) *(b)*

Figure 3-4: (a) Reinforcement detailing at the infill joint with (b) weld detail utilized at the joint.

3.1.2 Concrete Mix Design

Key details on the concrete for both the precast tunnel elements and approach ramps is summarized as follows, from [10] and [11]:

- Aggregate for the approach ramps was sourced from Deeks McBride Sand and Gravel Company, from a pit on the north shore of Burrard Inlet (presently the Maplewood Conservation Reserve [25]). Aggregate for the precast tunnel concrete originated from Gilley Brother's Mary Hill Pit on the north shore of the Fraser River near Douglas Island. Both aggregate sources are now closed.
- Testing completed on the aggregates included an assessment of the potential for alkali-reactivity by means of a rapid chemical test that was standardized as ASTM C 289 [13]. Results indicate the aggregates would be classed as innocuous per the standard. However, the ASTM C 289 approach, which measured the aggregate's dissolved silica concentration and reduction in alkalinity, was later found to not accurately predict potential reactivity of slowly expanding aggregate and many other aggregates and was therefore withdrawn in 2016. The results are considered unreliable;

- Plasticizing (i.e., water-reducing) admixture was used which was reported to entrain 2-3% air, which was "*considered sufficient for the exposure conditions to which the concrete would be subjected*";
- A filler from a single source was used to supplement fines in the concrete mixes for both the approach ramp and precast concrete elements. Limited detail on the filler is included in [11] to its source and gradation. The source is reported as "Delson Estate" (likely Delsom Estate) on the south shore of the Fraser River, opposite of Annacis Island. The maximum particle size was <300 µm with 100% passing the no. 50 size sieve;
- Normal Portland cement conforming to Canadian Specifications A 5-1951 was used. The cement was supplied by the British Columbia Cement Company's Bamberton facility on Vancouver Island [11], [23]. The same cement was used for the approach ramp and precast tunnel elements; and
- Mixing water consisted of Vancouver drinking water.

Table 3-1 provides the reported concrete mix design from [10] for the precast tunnel concrete.

The concrete mix design for the infill joints pours was not discussed in [10] and limited detail is provided in [11]. Per [11], the infill joints were part of a separate contract and a 'pumpcrete' was used for floors and walls. For the top slab, a crushed granite coarse aggregate was pre-placed in forms and grout was subsequently pumped into the joint. Findings from [1] indicate a differing aggregate size and type for the joints and even between different joints; concrete from the infill joints in the outer wall is distinctly different from the majority of the structure. Concrete mixes for the infill joints likely differ from the precast tunnel concrete mix design shown in Table 3-1.

Table 3-1: Trial testing determined concrete mix design for precast concrete tunnel elements, from [10].

	Imperial units (as reported in [10])	SI Units (converted)
Design Compressive Strength	3,750 psi	26 MPa
Cement ("Standard Portland cement")	525 lb/cy	311 kg/m^3
Water	242 lb/cy	144 kg/m^3
Fine Aggregate incl. filler	1440 lb/cy	854 kg/m^3
Coarse Aggregate	1900 lb/cy	1127 kg/m^3
Maximum Size Aggregate	1½"	38 mm
Air, %	2%	2%
w/c Ratio	0.48	0.48

3.2 Air Duct Tubes Condition Assessment

3.2.1 Visual Observations

Figure 3-5 provides photos of the typical condition of the soffit, internal/external walls and base slab top surface from the air duct tubes taken during present inspections (summer 2020). Visible surfaces of the concrete in air duct tubes generally had a similar appearance as shown in Figure 3-5, except at distinct sites in the air duct tubes where various indications of localized deterioration were observed in the structural reinforced concrete. The dark spots seen on the chamfer and bottom slab in Figure 3-5(b) and (d) are from water leaks from the drainpipes as discussed further in Section 3.4.

The following subsections discuss the signs of deterioration and other items of interest, divided by observation type including:

- Cracks (with and without signs of weeping or leakage);
- Reinforcement corrosion-induced damage of concrete;
- Condition of infill joints; and
- Other observations.

Figure 3-6 provides a schematic overview of the approximate location and extent of deterioration and other observations from the Outer (East) Wall of the Upstream Air Duct Tube. A similar frequency of issues was observed in the Downstream Air Duct Tube. Appendix A provides a complete compilation of visual observations from the air duct tubes.

(a) *(b)*

(c) *(d)*

Figure 3-5: Photos of the typical condition of (a) soffit, (b) outer wall, (c) intermediate wall, and (d) floor of the air duct tubes during summer inspection 2020. Dark spots on chamfer and bottom slab seen in (b) and (d) are from water seeping from drainpipes.

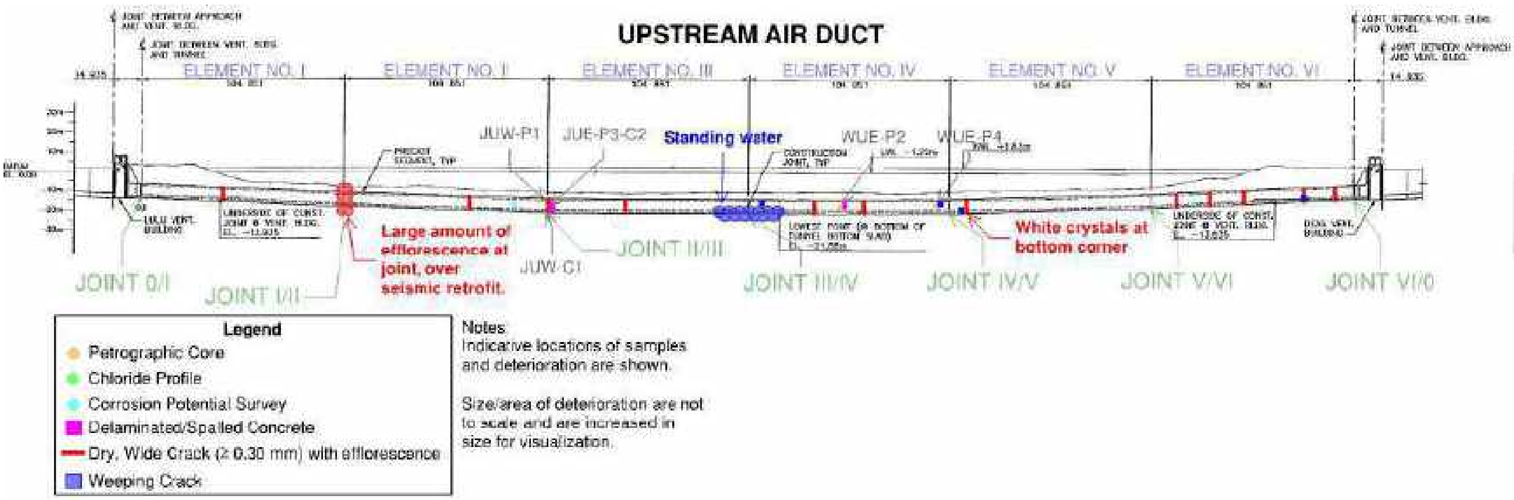


Figure 3-6: Schematic of approximate locations of observed deterioration at the Outer (East) Wall of the Upstream Air Duct Tube during summer inspection 2020.

3.2.1.1 Observations of Cracks

Cracks observed in the air ducts tubes can generally be categorized as either relatively wide transverse cracks or finer map cracks. See Figure 2-1(a) for definition of transverse.

Transverse cracks observed in the Outer (East) wall of the Upstream Air Duct are mapped in Figure 3-6. Cracks were observed only by the unaided eye and it is likely that not all transverse cracks were observed. The notable transverse cracks were typically 0.30-0.40 mm wide, appeared to be continuous at the intermediate wall and to a lesser extent (i.e., narrower) on the base and top slabs running relatively straight. Certain transverse cracks on the outer walls of the air ducts had efflorescence and/or corrosion staining, as shown in Figure 3-7(a, b), indicating the waterproofing membrane may not be watertight locally. Other transverse cracks were observed to have no staining of efflorescence or corrosion as shown in Figure 3-7(c, d). Each element contained at least one such transverse crack with a transverse crack also observed at the Joint II/III outer wall in the Upstream Air Duct.

While the vast majority of transverse cracks were dry at the time of inspection, weeping was observed at certain transverse cracks in the outer walls of the air duct tubes as shown in Figure 3-8. Repeat inspection of the air duct tubes during winter months would be useful to assess whether additional leaks are present in cold weather (i.e., when the concrete may contract due to lower ambient temperature conditions). White efflorescence was observed at all weeping cracks, with corrosion staining observed at the Downstream Air Duct, Element V, Door 25 as shown in Figure 3-8(b). As discussed further in Section 5, weeping cracks can promote rapid reinforcement corrosion and are therefore typically proactively injected to mitigate long-term durability concerns.

(a) (b)

(c) (d)

Figure 3-7: Transverse cracks observed in the outer walls of the air duct tubes, including (a) a dry crack with efflorescence and spalling at a location with ~35 mm cover (Downstream Air Duct, Element III, Door 17/18, powder samples collected), (b) a dry crack with efflorescence and corrosion staining (Downstream Air Duct, Element IV, Door 23/34), (c) a dry crack without staining (Downstream Air Duct, Element IV Door 20), (d) a dry crack without staining after core drilling (Upstream Air Duct, Joint II/III, Core JUE-P3-C2 collected, coring slurry seen on concrete surface).

(a) (b) (c)

Figure 3-8: Examples of weeping transverse cracks observed at (a) Upstream Air Duct, Element IV, Door 24, (b) Downstream Air Duct, Element V, Door 25, and (c) Downstream Air Duct, Element VI, Door 33.

As shown in Figure 3-9, distinct areas with fine map cracks were observed in both air duct tubes. Efflorescence was not observed at these locations. These fine cracks were observed locally and were not systemic throughout the air duct tubes. Core WUE-P4 was collected at the location shown in Figure 3-9(a) and findings from this core are presented in Section 3.2.3.

(a) (b) (c)

Figure 3-9: Examples of observed map cracking on outer walls of the air duct tube including at (a) Upstream Air Duct, Element IV, Door 24 (core WUE-P4 taken at this location), (b) Downstream Air Duct, Element II, Door 14, and (c) Downstream Air Duct, Element IV, Door 24.

3.2.1.2 Observations of Reinforcement Corrosion

Visual signs of reinforcement corrosion were observed locally at a limited number of locations in the air duct tubes. These visual signs included corrosion staining and small areas of delamination and/or spalling, examples of which are shown in Figure 3-10.

(a)

(b)

(c)

(d)

Figure 3-10: Examples of visual signs of reinforcement corrosion including (a) spalling and corrosion staining (Upstream Air Duct, Element IV, Door 22, Outer wall), (b) cracking, delamination and spalling (Downstream Air Duct, Element III, Door 17, Outer wall), (c) corrosion stains on concrete surface (Downstream Air Duct, Element V, Door 29, Outer wall), and (d) spall and exposed reinforcement (Downstream Air Duct, Element II, Intermediate wall).

3.2.1.3 Condition of Infill Joints

Figure 3-11 shows a typical infill joint. Galvanized steel plates located at the joints in the outer walls were observed to be free from corrosion.

The infill joints were observed to be dry. However, as shown in Figure 3-12, two infill joints (out of seven total joints) showed signs of possible leakage. At these locations, efflorescence is seen near the corner between the soffit and outer wall on both the surface of the concrete and the galvanized steel plate. Joint II/III of the

Upstream Air Duct had a transverse crack that was dry and without efflorescence on the surface of the concrete. Core JUE-P3-C2 was extracted from this location.

(a) *(b)*

Figure 3-11: Joints between individual immersed tunnel elements as seen in the air duct tubes including at (a) the outer wall (Downstream Air Duct, Joint II/III, Outer wall) and (b) the intermediate wall (Upstream Air Duct, Joint II/III, Intermediate wall).

(a) *(b)*

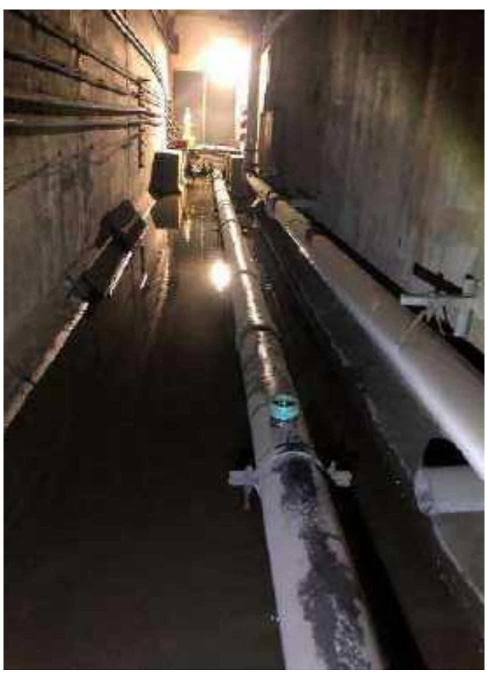
Figure 3-12: Efflorescence in the outer wall joints at (a) Upstream Air Duct, Joint I/II and (b) Downstream Air Duct, Joint IV/V.

Typically, the intermediate wall joint concrete was intact; however, the infill concrete at Joint II/III shown in Figure 3-11(b) was found to have delamination in both the Upstream Air Duct and Northbound Roadway Tube surfaces. Cores JUW-P1, JUW-C1, and JNE-P6 were taken from this joint to investigate.

3.2.1.4 Other Observations

Additional observations of note from the air duct tubes included the collection of water on the floor in the deepest parts of the immersed tunnel (i.e., near Joint III/IV) shown in Figure 3-13 and observed weeping of water from pipes protruding from the lower portion of the outer wall with associated crystal deposits as shown in Figure 3-14. Water collection was more pronounced in the Upstream Air Duct with both Elements III and IV have pooled water with maximum depth of approximately 10 cm towards the deepest point of the tunnel. The Downstream Air Duct was drier with the floor of Element IV being damp with some small puddles as shown in Figure 3-13(b) and the floor of Element III being dry (Obs. No. DS-18, Table B-2 of Appendix A).

Numerous drainpipes associated with the transverse construction joints were observed to be damp or slowly leaking, commonly with a buildup of crystalline deposits under and in the drainpipes. Figure 3-14 shows examples of this and sample S3 was collected at the location shown in Figure 3-14(a). Water leaking from the drainpipes could be from any location along the transverse construction joint between the segments and contraction pours and indicates the waterproofing membrane has likely failed locally along the joint. A detailed survey of the drainpipes was completed as reported in Appendix A, with additional discussion provided in Section 3.4.



(a)

(b)

Figure 3-13: Water collection in the air duct tubes: (a) Upstream Air Duct, (b) Downstream Air Duct.

(a) *(b)*

Figure 3-14: Examples of drainpipes in the outer wall of air ducts with leaking water and crystal deposits: (a) Downstream Air Duct, Element 1, Door 6/7, (b) Downstream Air Duct, Element I, Door 5/6.

3.2.2 Non-Destructive Test Results

Non-destructive testing completed in the air ducts included:

- Half-cell corrosion potential measurements per ASTM C876;
- Corrosion current density measurements made using Galvapulse equipment; and
- Cover measurements.

This testing was completed at one area in each of the air duct tubes at sites believed to be similar to those examined in 2000 [1]. Additional details on testing and classification system are provided in Appendix B.

Table 3-2 compiles the interpreted results from the corrosion potential, Galvapulse current density measurements, ground penetrating radar (GPR) measured concrete cover thickness[2], and observed condition of the reinforcement. The interpreted results from LPR measurements, completed in 2000 are also included in the table. Testing was completed at locations with no outward signs of corrosion or corrosion-induced deteriorations. All results indicate corrosion was likely not occurring at the

[2] As described in the original test report provided in Appendix B, "*The depth of the exposed bar was measured and used to calibrate the GPR device at each location to ensure accuracy of the nondestructive test results at other locations.*"

two measurements locations at the time of the measurements. Measured cover thicknesses are higher than specified in the design.

Table 3-2: Overview of non-destructive test results in the air ducts. Note 1 – Measurements provide indication only at the time of measurement.

Location	Measured Cover	Visual condition of exposed reinforcement	Corrosion Half-cell Potential Classification [1]	Galvapulse - Interpretation of corrosion rate [1]	2000 LPR Measurement classification [1] [1]
Downstream Air Duct Element IV, Door 20/21 Outer (West) wall	Average: 60 mm Minimum: 45 mm Maximum: 75 mm	No corrosion	Corrosion unlikely	Negligible to Slow	Passive to Low
Upstream Air Duct Element II, ~5 m north of Door 13 Outer (East) wall	Average: 38 mm Minimum: 36 mm Maximum: 42 mm	No corrosion on the exposed embedded bar	Corrosion unlikely	Slow	Passive

3.2.3 Observations from Cores and Powder Samples

Cores and powder samples extracted from the air duct tubes were subjected to chemical and/or petrographic analysis as summarized in Table A-1. Detailed results are provided in Appendix B and Appendix C, and are summarized in the following sections.

3.2.3.1 Carbonation Depths

Table 3-3 presents measured carbonation depths. Minimal carbonation depth was measured for the outer wall infill joint concrete and precast tunnel concrete. Carbonation of the precast tunnel concrete was measured to range from 2-12 mm, while a range of 5-13 mm was measured for the infill joint concrete at the outer wall.

The intermediate wall infill concrete had a maximum measured carbonation depth of 45 mm and generally the measured carbonation depth from the infill joint concrete at the intermediate wall has reached the design cover thickness. Measurements in the following section confirm the pH in the cover concrete was <11 for core JUW-P1, which was taken near an area of delamination in the joint concrete.

Considering the design cover thicknesses described in Section 3.1.1 and measured cover thicknesses, carbonation-induced corrosion is a potential cause for the delamination observed at the intermediate wall of Joint II/III.

Table 3-3: Measured carbonation depths in the air duct tubes by phenolphthalein spray.

Core ID	Concrete Type/Location		Carbonation Depth
JUW-P1	Infill joint concrete – Intermediate wall	Upstream Air Duct Joint II/III	Average: 38 mm Range: 33-45 mm
JUW-C1		Upstream Air Duct Joint II/III	30 mm
JUE-P3-C2	Infill joint Concrete – Outer wall	Upstream Air Duct Tube Joint II/III	Average: 5 mm Range: 3-13 mm
WUE-P2	Precast Tunnel Concrete	Upstream Air Duct Tube Element IV, Door 22	Range: 7-12 mm
WUE-P4		Upstream Air Duct Tube Element IV, Door 24	2 mm

3.2.3.2 Chloride and pH Profiles

Table 3-4 compiles the measured chloride and pH profiles from samples collected from the air duct tubes. The sample type, core or powder sample (PS), is indicated in the table. The measured chloride content can be compared to the theoretical critical chloride threshold concentration, that is the concentration required to break down the passive layer protecting the steel reinforcement, which may lead to corrosion initiation. While this threshold value varies based on the composition of the steel, concrete mix, internal humidity conditions of the concrete, among other factors, typically cited threshold values range from 0.025 to 0.075% wt. concrete. A threshold value of 0.025% wt. concrete was assumed in [1]. The passive corrosion layer on reinforcement can also become unstable due to carbonation of the concrete cover, with a pH of 9-10 typically considered sufficient to initiate depassivation.

To facilitate interpretation of the chloride and pH profiles, a colour code is introduced in Table 3-4 based on the above values to provide a qualitative assessment of the potential for corrosion status from the measured pH and chloride content: green (below 0.025%), yellow (between 0.025 and 0.075%), and red (above 0.075%). The lowest recorded pH value was 9.9 and therefore all pH values are categorized as 'green'.

Sample PS-S1 described in Table 3-4 was taken approx. 30 mm from a transverse crack in the outer wall. Core JUE-P3-C2 includes a transverse crack in the outer wall. Both of these samples have chloride profiles indicating chloride exposure. Reinforcement corrosion, spalling, and dry efflorescence were observed at the location of S1 (Obs. No. DS-15 in Table B-2).

Powder samples PS-CL1 to PS-CL10 presented in Table 3-4 were collected by means of an impact drill at 15 mm increments, resulting in limited sample size (i.e., volume of the individual sample). Individual results may be influenced by natural fluctuations in the aggregate/paste content in the obtained powder and therefore the powder sample results are considered collectively by concrete type. The samples were collected from locations with a surface appearance similar to that of the general appearance of the tunnel elements or infill joints.

Results from the powder samples shows limited chloride ingress and carbonation penetration. Chloride contents in the six samples from the precast tunnel concrete are slightly elevated near the surface; however, the 0.025%wt. concrete threshold is not reached. A similar trend is observed in three of the four powder samples from the infill joints, with slightly elevated chloride contents observed near the surface. Powder sample PS-CL4 shows heightened chloride contents in the 'yellow' range at the three deepest measurement locations, whereas the two shallowest measurements depths have chloride content <0.025%wt. concrete.

Table 3-4: Water Soluble Chloride and pH profiles measured from select cores and samples from the air duct tubes.

Core/Sample ID	Concrete Type/Location		Depth (mm)	Chloride content (%wt. concrete)	pH
PS-S1 (powder sample)	Precast Tunnel Concrete	Downstream Air Duct Element III, Door 17/18 Outer (West) wall	0-20	0.120	12.3
			20-40	0.102	11.6
			40-60	0.136	12.4
			60-80	0.076	12.4
PS-CL1 (powder sample)		Downstream Air Duct Element I, Door 6 Contraction Pour 2 Outer (West) wall	0-15	0.016	11.1
			15-30	0.022	*
			30-45	0.017	12.1
			45-60	0.021	*
			60-75	0.024	*
PS-CL2 (powder sample)		Downstream Air Duct Element III, Door 16/17 Outer (West) wall	0-15	0.021	12.1
			15-30	0.016	*
			30-45	0.014	12.3
			45-60	<0.01	12.3
			60-75	<0.01	*

Core/Sample ID	Concrete Type/Location		Depth (mm)	Chloride content (%wt. concrete)	pH
PS-CL3 (powder sample)	Precast Tunnel Concrete	Downstream Air Duct Element V, Door 25/26 Outer (West) wall	0-15	0.013	12.3
			15-30	<0.01	11.9
			30-45	<0.01	12.3
			45-60	<0.01	12.3
			60-75	<0.01	11.6
PS-CL6 (powder sample)		Upstream Air Duct Element VI, Door 32 Outer (East) wall	0-15	0.013	12.0
			15-30	<0.01	12.1
			30-45	<0.01	12.3
			45-60	<0.01	12.2
			60-75	<0.01	12.3
PS-CL9 (powder sample)		Upstream Air Duct Element IV, Door 23/24 Outer (East) wall	0-15	0.012	12.1
			15-30	0.012	11.9
			30-45	0.011	12.3
			45-60	<0.01	12.3
			60-75	<0.01	12.0
PS-CL10 (powder sample)		Upstream Air Duct Element II, Door 13/14 Outer (East) wall	0-15	0.015	12.2
			15-30	0.015	12.4
			30-45	<0.01	11.8
			45-60	<0.01	12.3
			60-75	<0.01	12.2
JUE-P3-C2 (core)	Infill joint Concrete – Outer wall	Upstream Air Duct Tube Joint II/III	0-15	0.050	12.6
			15-30	0.068	12.7
			30-45	0.056	12.6
			45-60	<0.01	12.7
			60-75	0.023	12.6

Core/Sample ID	Concrete Type/Location		Depth (mm)	Chloride content (%wt. concrete)	pH
PS-CL7 (powder sample)	Infill joint Concrete – Outer wall	Upstream Air Duct Joint IV/V	0-15	0.012	11.9
			15-30	0.010	12.2
			30-45	<0.01	12.0
			45-60	<0.01	12.1
			60-75	<0.01	12.2
PS-CL4 (powder sample)		Downstream Air Duct Joint V/VI	0-15	0.012	11.1
			15-30	0.020	11.7
			30-45	0.052	12.2
			45-60	0.037	12.2
			60-75	0.028	12.3
JUW-P1 (core)	Infill joint concrete – Intermediate wall	Upstream Air Duct Joint II/III	0-15	<0.01	10.8
			15-30	<0.01	9.9
			30-45	<0.01	12.2
			45-60	0.021	12.5
			60-75	0.03	12.6
PS-CL5 (powder sample)		Downstream Air Duct Joint VI/0	0-15	0.022	11.4
			15-30	0.010	11.7
			30-45	0.016	12.2
			45-60	0.019	12.3
			60-75	<0.01	12.2
PS-CL8 (powder sample)		Upstream Air Duct Joint IV/V	0-15	0.029	11.3
			15-30	<0.01	11.9
			30-45	0.013	12.4
			45-60	<0.01	12.4
			60-75	<0.01	12.4
* Laboratory reported insufficient powder sample remained for pH testing after processing for chloride ion content.					

3.2.3.3 Petrographic Analysis

Petrographic examination was conducted on the cores JUW-C1, WUE-P2, JUE-P3-C2, and WUE-P4 extracted from the Upstream Air Duct. Petrography reports, included in Appendix C, document that all cores consisted of concrete that was non-air-entrained, generally dense and well-consolidated with well-hydrated cement paste. Table 3-5 provides an overview of details on the individual cores with the summary text from the original petrography reports. Bold text in the summary column is added to emphasize defects observed.

As indicated in Table 3-5, the four cores extracted from the air duct show signs of ASR, with more significant signs of ASR observed in core JUE-P3-C2 from the outer wall at Joint II/III compared to the precast tunnel concrete. Due to observed ASR in the cores, damage rating indices are computed in Section C.1 of Appendix C. These results indicate that the relative severity of ASR is maximal at the infill joint concrete from the outer wall (i.e., core JUE-P3-C2). A remaining portion of core JUE-P3-C2 was tested for compressive strength. While the sample's aspect ratio of 0.72 (L/D) was not ideal, results provide an indication that unit weight and compressive strength of the concrete remains high at 2482 kg/m^3 and ~58 MPa, respectively.

Additional thin and plane sections of the outermost portion of core WUE-P4 were prepared and analyzed to assess whether potential cause(s) for the fine map cracking on the concrete surface could be ascertained. Evidence of ASR at this location is somewhat inconclusive with trace amounts indicated and the cause or causes of these surface cracks was not determined with certainty.

Table 3-5: Summaries from petrography reports for Upstream Air Duct Tube cores.

Core ID	Concrete type/Location & Notes		Summary
WUE-P2	Precast tunnel concrete Outer wall Element II	7-8 cm deep core taken adjacent to an area of spalled concrete (Obs No. US-8 in Table A-3)	The concrete is well-consolidated with adequate proportioning and distribution of aggregates in the paste matrix. Paste interface is good. The **outer 7-12 mm** of the concrete is **carbonated**. ASR-related effects are minimal: minor microcracking, reaction rims, **ASR gel in a few locations**.
WUE-P4 (inner portion of core)		~30 cm deep core taken at fine map cracking on wall surface.	The concrete contains a **moderate amount of open and fine cracks, some lined/filled with ASR gel**, some devoid of fillings. Reaction rims are common on some particles. **ASR gel fills some voids**. Minor **carbonation** in **outer 2 mm** of concrete.
WUE-P4 (outer portion of core)			**Cracks** are observed throughout the core, most/many **lined wholly or in part with ASR gel**. Some cracks are absent of ASR gel – those that appear in the outer portion of the core (e.g., outer 50 mm) and are oriented parallel to the core surface **may be considered as resulting from freeze-thaw activit**y. Other cracks that are absent of ASR gel may not

Core ID	Concrete type/Location & Notes		Summary
			conclusively be considered to have resulted from other mechanisms. Some cracks contain ASR gel in only a portion of the visible crack expression; ASR gel, having dried and hardened may have been removed in sample preparation. An overview estimate suggests that at least 80% of the cracks may contain some amount of ASR gel
JUW-C1	Infill joint concrete Joint II/III	Intermediate wall ~10 cm deep core taken adjacent to delamination in joint	The **outer ~30 mm** of the paste is **carbonated**. Localized patches of softer, more absorbent paste are observed in the sample. **Frequent indications of Alkali-Silica Reaction** (ASR) observed in the concrete. Cracking in paste is moderate. Reaction rims on some aggregate types are common.
JUE-P3-C2		Outer wall ~30 cm deep core taken at transverse crack in joint	The concrete contains a variety of **open (larger) and fine cracks, some lined/filled with ASR gel**, some devoid of fillings. Reaction rims are common on aggregate particles. **ASR gel fills some voids**. **Partially debonded aggregates** observed. **Corroded aggregates** are observed, mostly in the fine aggregate size range.

3.2.4 Comparisons with 2000/2001 Inspection Findings

Comparative photographs were taken from similar areas of photographs included in the 2000/2001 inspection [1]. Figure 3-15 shows that a series of horizontal cracks in the outer wall of the Downstream Air Duct in Element V near Door 29 have undergone limited change from 2000 to 2020; however, additional signs of corrosion staining near the vertical joint are observed.

(a) *(b)*

Figure 3-15: Comparison photos from area on outer wall in Downstream Air Duct, Element V, Door 29 with (a) showing the condition during the current inspection and (b) the condition during the 2000 inspection [1].

Comparison of carbonation depth measurements shows progression of the carbonation front, which is to be expected. Depth of carbonation increased from a range of 0 to 10 mm in 2000 [1] for the precast tunnel concrete to a range of 2 to 12 mm in 2020. For the infill joint concrete at the outer walls, a similar carbonation depth is measured, with approx. 15 mm carbonation reported in 2000 and up to 13 mm in 2020. Carbonation depth of the infill joint concrete at the intermediate walls has the most significant increase, with 0-2 mm carbonation reported in 2000 and up to 45 mm carbonation depth observed in 2020. The observed depth of carbonation in the infill joint concrete at the intermediate walls exceeds the design cover thickness and can initiate reinforcement corrosion.

Comparison of chlorides levels from concrete in the air duct tubes from 2000 and 2020 show similar profiles in general, with the majority of chloride contents found to be below the threshold required to initiate corrosion. However, the two samples in the 2020 investigation taken at or near surface cracks (i.e., PS-S1 and JUE-P3-C2) presented sufficient chloride concentration at the depth of the reinforcement to potentially initiate reinforcement corrosion.

The extent of ASR damage observed in concrete cores may have progressed since 2000, with cores taken in this study (2020) showing cracks, reaction rims, and gel due to ASR. The 2000 assessment reported "insignificant" signs of ASR from tunnel concrete. However, both assessments are based on a very limited numbers of samples and should be treated with caution.

3.3 Northbound Roadway Tube Condition Assessment

3.3.1 Visual Observations

Figure 3-16 illustrates the typical condition of the soffit, central wall between the Northbound and Southbound Roadway Tubes, wall between the Northbound Roadway and Upstream Air Duct tubes, and the roadway surface of the Northbound Roadway Tube. A yellowish surface coating or paint was present on both the central and intermediate wall, which is partially peeling/chipping and the walls and emergency doors were covered by a built-up layer of soot/dirt (both not present in the air duct tubes) that hindered the visibility of cracks and other indication of deterioration.

The visible surfaces of the concrete in the Northbound Roadway Tube generally had a similar appearance as shown in Figure 3-16, except at distinct sites where localized indications of various deterioration mechanisms were observed in the structural reinforced concrete. The following subsections discuss the signs of deterioration and other items of interest, divided by observation type including:

- Reinforcement corrosion-induced damage of concrete;
- Condition of infill joints; and
- Other observations.

Figure 3-17 provides an overview of the approximate locations of the signs of deterioration and other observations in the Northbound Roadway Tube reported in Appendix A.

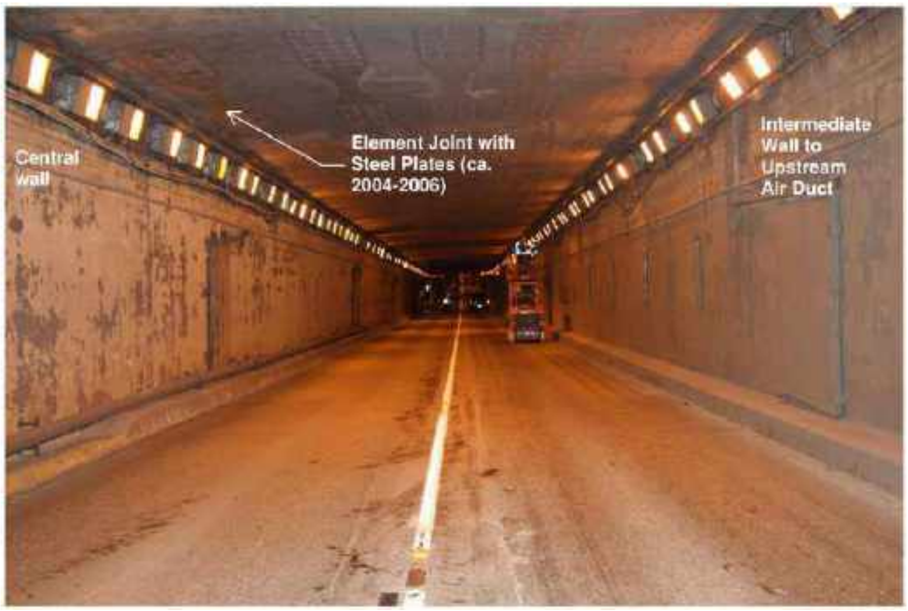


Figure 3-16: Overview of the Northbound Roadway Tube at Joint V/VI, looking north.

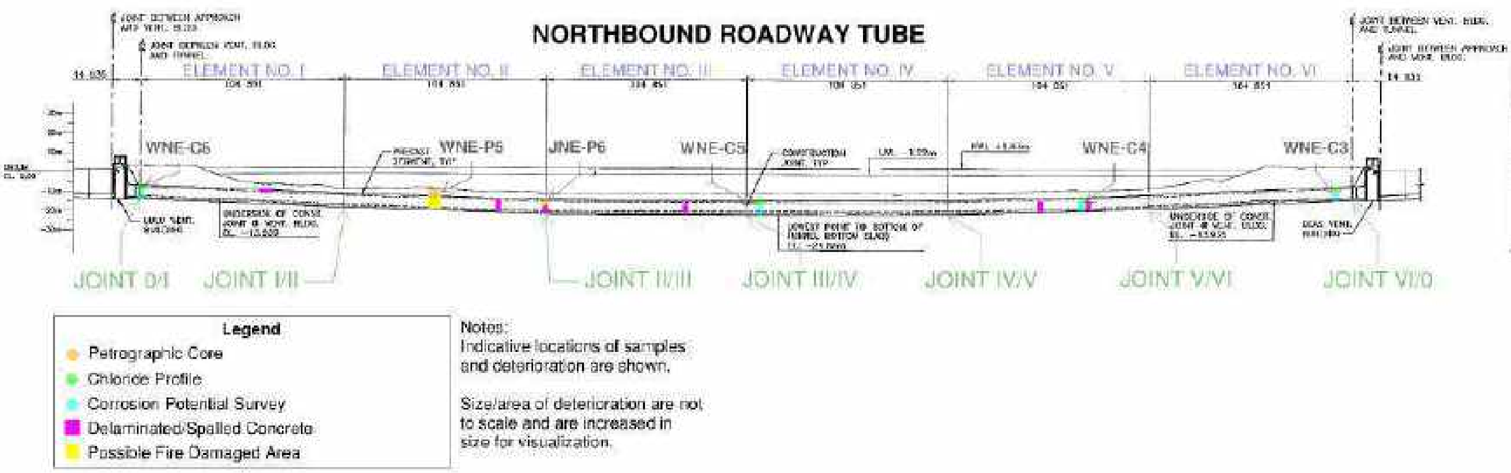
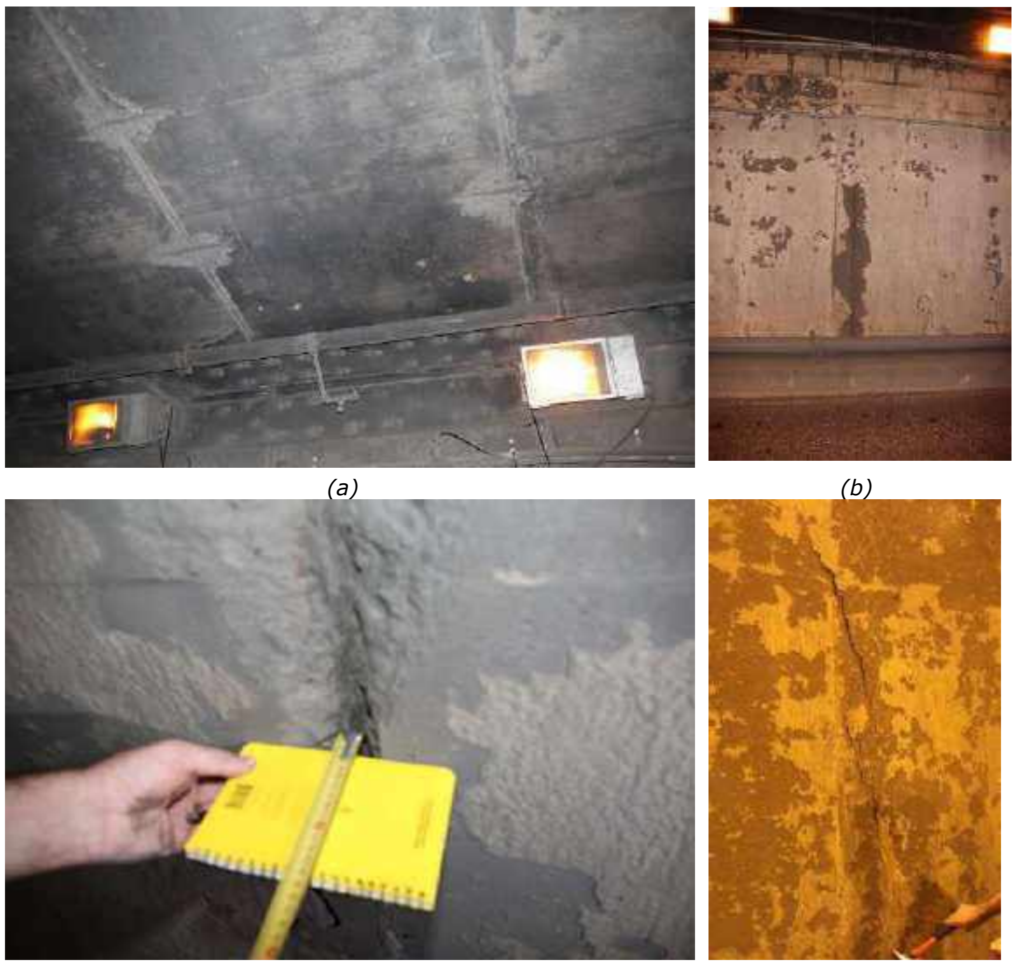


Figure 3-17: Overview of the approximate locations of ongoing deterioration in the Northbound Roadway Tube.

3.3.1.1 Observations of Reinforcement Corrosion

Visual signs of potential concrete damage by reinforcement corrosion were observed locally at a limited number of locations in the Northbound Roadway tube. These visual signs included areas of delamination and/or spalling, examples of which are shown in Figure 3-18. Figure 3-18(a) shows that spalling has occurred at two transverse construction joints on either side of a Contraction Pour. The fracture and delamination shown in Figure 3-18(d) could potentially be caused by sources other than reinforcement corrosion (e.g., settlement) and the location should be subject to follow-up scrutinization during Detailed and Routine Condition inspections described in Section 5.3.

Observed spalling from the soffit in the roadway tube is critical as it presents a potential safety concern, as discussed further in Section 5.1.2.



(a) (b)

(c) (d)

Figure 3-18: Examples of visual signs of reinforcement corrosion including delamination and/or spalling at (a) the soffit in Element I, Door 7/8 showing spalling at the transverse construction joints, (b) the central wall in Element V north of Door 28, (c) the central wall in Element II, South of Door 13, and (d) the central wall in Element III north of Door 18.

3.3.1.2 Condition of Infill Joints

Figure 3-16 includes a typical infill joint as observed in the Northbound Roadway Tunnel. Galvanized steel plates were located at the soffit of the top slab and were observed to be free from corrosion on visible surfaces. Leaks or weeps were not observed at infill joints and no signs of past leakage were noted.

At the central and intermediate walls, the joint was observed to be in similar condition as the remainder of the walls in the Northbound Roadway Tube, except for the Joint II/III at the intermediate wall. As shown in Obs. No. NB-5 in Table A-6 in Appendix A, at this location the joint was found to have a large area of delaminated concrete, similar to the deterioration observed at opposite side of this joint in the Upstream Air Duct (see Section 3.2.1.3).

3.3.1.3 Other Observations

Possible fire damage was observed on the intermediate wall and soffit in Element II near Door 12 as shown in Obs. No. NB-3 in Table B-6 in Appendix A. Core WNE-P5 was collected from the intermediate wall at this location for petrographic evaluation with results presented in Section 3.3.3.

3.3.2 Non-Destructive Test Results

Table 3-6 compiles interpreted results from field testing completed in the Northbound Roadway tube as well as the visual condition of the reinforcement exposed for testing. Again, field testing was completed at locations with no outward signs of corrosion or corrosion-induced deterioration. While results from the individual tests provide varying indications at certain locations, it can be seen that at three out of four locations at least one measurement indicates a potential for active corrosion. Actual corrosion products were observed on the exposed reinforcement at two out of the four locations with corrosion products observed at locations with covers larger than the design covers. This means that there is a risk of corrosion in areas with the design covers and even in areas with larger covers.

Table 3-6: Overview of non-destructive test results in the Northbound Roadway Tube. Note 1 – Measurements provide indication only at the time of measurement.

Location	Measured Cover	Visual condition of exposed reinforcement	Corrosion Half-cell Potential Classification [1]	Galvapulse - Interpretation of corrosion rate [1]
Core WNE-C3 Element VI, Door 34 Intermediate (U.S.) wall, 85-90 cm above roadway level	Average: 43 mm Minimum: 39 mm Maximum: 46 mm	Minimal signs of corrosion on exposed bar	Corrosion unlikely	Slow

Location	Measured Cover	Visual condition of exposed reinforcement	Corrosion Half-cell Potential Classification [1]	Galvapulse - Interpretation of corrosion rate [1]
Core WNW-C4 Element V, Door 27 Central wall, ~130 cm above roadway level	Average: 32 mm Minimum: 31 mm Maximum: 33 mm	~25 mm cover and corrosion products observed	Potential for corrosion	Moderate
Core WNE-C5 Element IV, Door 20 Intermediate (U.S.) wall, ~100 cm above roadway level	Average: 54 mm Minimum: 50 mm Maximum: 57 mm	Some corrosion products observed	Potential for corrosion*	Negligible
Core WNE-C6 Joint 0/I Intermediate (U.S.) wall, ~125 cm over roadway level	Average: 44 mm Minimum: 43 mm Maximum: 45 mm	No signs of corrosion	Corrosion unlikely	Moderate
* Based on average of the measurements, minimum measured value is in the "Potential for corrosion" range				

3.3.3 Observations from Cores

Cores extracted from the Northbound Roadway Tube were subjected to chemical and/or petrographic analysis as summarized in Table A-1. Results from the data reports, which are provided in Appendix B and Appendix C, are summarized in the following sections.

3.3.3.1 Carbonation Depths

Measured carbonation depths from the Northbound Roadway Tube are compiled in Table 3-7. Core JNE-P6 was extracted from the opposite face of the same joint (Joint II/III) as cores JUW-P1 and JUW-C1, with carbonation depths ranging from 30-45 mm. Carbonation depth from core JNE-P6 is slightly less, ranging from 20-25 mm, but is nevertheless still higher than typically observed carbonation depth from other concrete types in the tunnel.

Carbonation measurements of the precast tunnel concrete is more varied than as measured in the air duct, with values ranging from 0 to 18 mm.

Considering the design cover thicknesses described in Section 3.1.1 and measured cover thicknesses, carbonation-induced corrosion is a potential cause for the delamination observed at the intermediate wall of Joint II/III.

Table 3-7: Measured carbonation depths in the Northbound Roadway Tube by phenolphthalein spray.

Core ID	Concrete Type/Location		Carbonation Depth
WNE-C6	Infill joint concrete – Intermediate wall	Joint 0/I ~125 cm over roadway level	Average: 11 mm Range: 6-16 mm
JNE-P6		Joint II/III	20-25 mm
WNE-C3	Precast Tunnel Concrete	Element VI, Door 34 Intermediate wall 85-90 cm over roadway level	Average: 0 mm
WNW-C4		Element V, Door 27 Intermediate wall ~130 cm over roadway level	Average: 8 mm Range: 5-18 mm

3.3.3.2 Chloride and pH Profiles

The colour code introduced in Section 3.2.3.2 is applied to the compiled chloride and pH profiles measured from the Northbound Roadway Tube presented in Table 3-8. As shown, elevated chloride levels are observed in the cores, with the threshold value approached or exceeded at the depth of the design cover thickness. Average cover thicknesses measured in the roadway tube in Table 3-6 range from 32 to 44 mm the these locations, indicate the chloride threshold is approached or reach at the actual cover thicknesses as well.

Table 3-8: Chloride and pH profiles measured from select cores from the Northbound Roadway Tube.

Core ID	Concrete Type/Location		Depth (mm)	Chloride content (%wt. concrete)	pH
WNE-C3 (core)	Precast Tunnel Concrete	Element VI, Door 34, Tunnel Entry Intermediate wall 85-90 cm over roadway level	0-15	0.058	12.6
			15-30	0.027	12.6
			30-45	<0.01	12.6
			45-60	<0.01	12.6
			60-75	<0.01	12.7
			0-15	0.394	12.4

Core ID	Concrete Type/Location		Depth (mm)	Chloride content (%wt. concrete)	pH
WNW-C4 (core)		Element V, Door 27, ¼ way through tunnel Intermediate wall ~130 cm over roadway level	15-30	0.214	12.6
			30-45	0.053	12.5
			45-60	0.014	12.6
			60-75	-	-
WNE-C6 (core)	Infill joint concrete – Intermediate wall	Joint 0/I, Tunnel Exit ~125 cm over roadway level	0-15	0.154	12.5
			15-30	0.019	12.6
			30-45	<0.01	12.6
			45-60	<0.01	12.6
			60-75	<0.01	12.6

3.3.3.3 Petrographic Analysis

Petrographic examination was conducted on the cores WNE-P5 and JNE-P6 extracted from the Northbound Roadway Tube. Petrography reports, included in Appendix C, document that both cores consisted of concrete that was non-air-entrained, generally dense and well-consolidated with well-hydrated cement paste. Table 3-9 provides an overview of details on the two cores with the summary text from the original petrography reports. Bold text in the summary column is added to emphasize the observed defects.

As indicated in Table 3-9, both cores show signs of ASR. Petrographic assessment of core WNE-P5 appear to confirm that this section of the tunnel was exposed to fire (Obs. No. NB-3 in Table A-6), with the outer ~15 mm presenting characteristics associated with fire damage. Due to observed ASR in the cores, damage rating indices were determined for the cores, which are presented in Section C.1 of Appendix C.

Table 3-9: Summaries from petrography reports for Northbound Roadway Tube cores.

Core ID	Concrete type/Location & Notes		Summary
WNE-P5	Precast tunnel concrete Intermediate wall	Element II, Door 12 ~110 cm over roadway level at location of suspected fire damage (Obs. No. NB-3 in Table A-6)	The **outer ~15 mm** of the core has **characteristics** that are **consistent with fire damage**. Additionally, and possibly due to the presence of cracks in the fire-damaged zone, **carbonation follows a few cracks into the concrete**. **ASR-related indications are present but are minimal** in nature, persistence and level of damage.

Core ID	Concrete type/Location & Notes		Summary
JNE-P6	Infill joint concrete Joint II/III	~60 mm deep solid core (plus ~30 mm of shards) take at location of apparent delamination ~120-130 cm above roadway level	The concrete contains a moderate number of **open and fine cracks, some lined/filled with ASR gel**, some devoid of fillings. Reaction rims are common on some volcanic particles. **ASR gel fills some voids and cracks**. **Outer ~20-25 mm of paste is carbonated.**

3.4 Watertightness

The tunnel's waterproofing characteristics were reported as good in the 2000 investigation [1]. However, during the current inspection, indirect evidence of failures of the waterproofing membrane were observed at the following locations:

- Certain infill joints;
- Numerous transverse construction joints; and
- Certain transverse cracks in the precast concrete tunnel.

As shown in Figure 3-13, water was present in the sump in both air duct tubes, with the Upstream Air Duct having up to approximately 10 cm of standing water in Elements III and IV (i.e., deepest elements) as shown in Figure 3-13. This water level is considered significant as standing water is not typically observed outside of sumps in other immersed tunnels. The water level did not visibly change between the July and September site visits, with significant rain falling during the September visit. Personal communication on site with an engineer familiar with the existing GMT gave an indication that the observed water level is the required water height to activate pumps [19]. Nevertheless, the water level and possible source of the water should be confirmed as discussed in Section 5.3.1.

3.4.1 Infill Joints

No active leaks were observed in infill joints during the current inspection in summer 2020 (i.e., the joint were dry), however there were signs of past leakage at Joint IV/V in the Downstream Air Duct (Obs No. DS-30 in Table A-2) and Joint I/II in the Upstream Air Duct (Obs. No US-1 in Table A-3). At these locations, efflorescence was observed over top of the seismic retrofit, indicating localized leakage has continued at these joints during the past ~15 years (i.e., since installation of seismic retrofit).

3.4.2 Transverse Construction Joints

As discussed in Section 3.1.1, transverse construction joints include a drain detail that is connected to drainpipes visible in the air duct tubes. The condition of these drainpipes was recorded in detail during the September follow-up walkthrough, which is documented in Table A-7, included in Appendix A. Overall, water or dampness was present at 44% of the drainpipes. Further, 47 of the 71 (66%) observable transverse construction joints had at least one drainpipe with water or dampness present. Under many of the drainpipes with leaks, crystalline deposits were observed as shown in Figure 3-14. Test results from a sample of the crystalline deposit (Sample S3) and was found to have high salinity as reported in Appendix B.

Review of the drain detail drawings indicates that 1) for water to be dripping from drainpipes it appears that the entire concrete formed drain below drainpipe level needs to be water-filled, and 2) sources of water with potential to fill the concrete formed drain to this level would be ingress of water from the outer surface of the tunnel[3]. It is therefore likely that local failures of the waterproofing membrane have occurred above the transverse construction joints, allowing external saline or brackish water to penetration through the joint.

3.4.3 Transverse Cracks

While some of the wide transverse cracks seen in the air duct tubes were dry during current inspection, others had evidence of leaks including damp spots, and efflorescence as shown in Figure 3-8. Chloride profiles measured at two transverse cracks showed increased chloride contents compared to 'background' chloride profiles measured in uncracked concrete.

[3] Water could also enter the concrete formed drains by seeping through the wearing course in the roadway tubes; however, there is insufficiently head to cause the water level in the drain to reach the drainpipes.

4 Approach Structures

4.1 General

4.1.1 Description of Structure

The approach structures consist of cast-in-place reinforced concrete retaining walls integrated with bottom reinforced concrete slabs, sloped to provide a ramp transition from the normal highway level down to the immersed tunnel. The specified concrete cover thickness in the approach structures was 2" (~50 mm) according to drawing 6-J-1121. The ramps have a total length of 355 m for the Northern (Lulu) Approach and 317 m for the Southern (Deas) Approach. The surfaces in contact with soil of the approach ramp walls and base slab are covered by a waterproofing membrane as shown in Figure 4-1. Reinforcement, shown in drawing 1540-R-19 in [8], is discontinuous between the ventilation buildings and the approach ramps. The waterproofing membrane consisted of four layers of hot-applied asphalt, with 'Coromat Reinforcing Membrane' based between inner layers of the asphalt. The outermost layer of hot-applied asphalt was covered by 'Glasfab Reinforcing Membrane'.

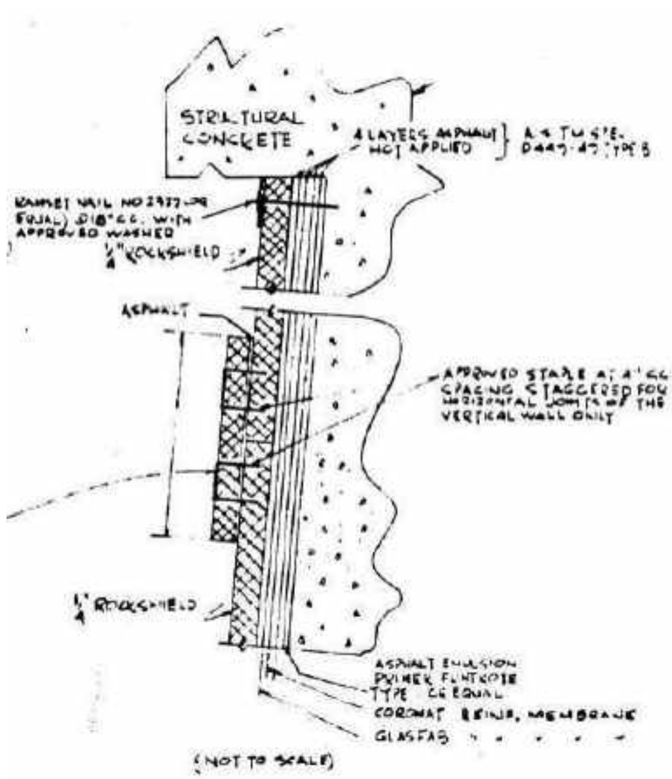


Figure 4-1: Lulu and Deas Approach Waterproofing Details from Drawing 6J-1158.

For the base slab, the membrane was applied to a 4" layer of concrete and the structural reinforced concrete was cast into the pre-placed membrane. A single external waterstop of stainless steel was applied for expansion joints in the base slabs.

For the walls, the membrane was applied to the formed surface of the structural reinforced concrete and covered by a ¼" protective 'rockshield'. Two waterstops were placed in expansion joints including an internal PVC waterstop embedded in the concrete and an external stainless steel waterstop.

4.1.2 Concrete Mix Design

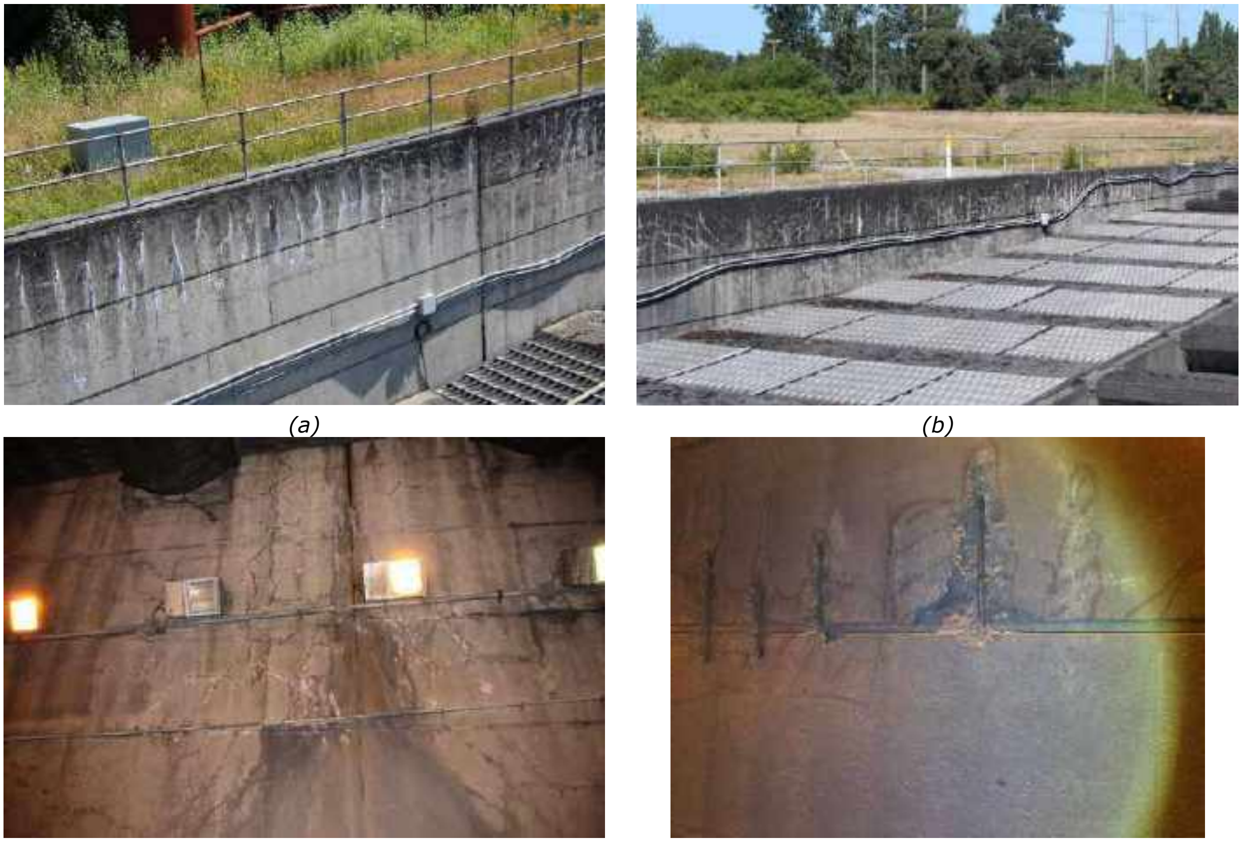
In addition to project-wide detail on concrete provided in Section 3.1.2, Table 4-1 provides the reported concrete mix design from [10]. Aggregate for the approach ramps was sourced from Deeks McBride Sand and Gravel Company, from a pit on the north shore of Burrard Inlet [11]. In general, the concrete quality of the approach ramps is lower than the precast concrete due to a lower cement content and higher water-to-cementitious materials (w/c) ratio.

Table 4-1: Trial testing determined concrete mix design for cast-in-place concrete for the approach ramps and ventilation buildings, from [10].

	Imperial units (as reported in [10])	SI Units (converted)
Cement ("Standard Portland cement")	481 ¼ lb/cy	285 kg/m^3
Water	250 lb/cy	148 kg/m^3
Fine Aggregate incl. filler	1370 lb/cy	813 kg/m^3
Coarse Aggregate	1975 lb/cy	1172 kg/m^3
Air, %	2%	2%
w/c Ratio	0.52	0.52

4.2 Visual Observations

The approach ramps were reported to be suffering from deterioration induced by ASR along the upper portions of the retaining walls in [1]. Classical signs of ASR, including pattern cracks with exuding gel, were observed at both the Northern (Lulu) and Southern (Deas) Approach Ramps as shown in Figure 4-2(a-c).



(a) *(b)*

(c) *(d)*

Figure 4-2: Signs of deterioration on the approach ramps include AAR at the upper portions of the retaining walls at (a) the Southern (Deas) Approach Ramp, and (b) Northern (Lulu) Approach Ramp, (c) map cracking with efflorescence and corrosion staining from a joint in the Southern (Deas) Approach Ramp (Element A1, Door 37) and (d) delamination, spalling and corroding reinforcement in the Southern (Deas) Approach Ramp (Element A8, Door 38).

As was reported in [1], outward signs of AAR were observed to a far lesser extent lower in the approach ramps, near the roadway level. As shown in Figure 4-2(a, b), signs of ASR continue to be somewhat less pronounced at the roadway level. There are, however, local signs of reinforcement corrosion and potential ASR near the roadway, as shown in Figure 4-2(c, d). The more advanced damage at the top portion of the retaining wall may be caused by the horizontal surface at the top of the wall allowing increased water saturated compared to the vertical walls. Fine surface cracks, formed due to ASR throughout the retaining wall, may have been preferentially subject to additional freeze/thaw attack of the non-air-entrained concrete near the top of the retaining walls.

At the time of inspection, work was ongoing to rehabilitate movement joints in the base slabs of the approach ramps. This work permitted visual assessment of the reinforcement in the base slab near the joints. As shown in Figure 4-3, corrosion products were observed on the base slab reinforcement, clearly showing corrosion (likely chloride-induced) is ongoing at least near to movement joints.

(a) *(b)*

Figure 4-3: Representative examples of reinforcement exposed in the base slab of the Southern Approach Ramp at locations of ongoing rehabilitation of the movement joints.

4.3 Observations from Cores

Cores extracted from the approach ramps were subjected to chemical and/or petrographic analysis as summarized in Table A-1. Results from the original data reports, which are provided in Appendix B and Appendix C, are summarized in the following sections.

4.3.1 Carbonation Depths

Measured carbonation depths from the approach ramps are compiled in Table 4-2 and indicate minimal penetration of carbonation. Measured carbonation depths have not yet reached the design cover thickness described in Section 4.1.1.

Table 4-2: Measured carbonation depths in the approach ramps by phenolphthalein spray.

Core ID	Concrete Type/Location		Carbonation Depth
NAE-C7 (core)	Northern (Lulu) Approach Ramp	Element A10 Curb concrete	Average: 6 mm Range: 5-8 mm
SAE-C8	Southern (Deas) Approach Ramp	Element A10, East wall ~90 cm over roadway level	5-15 mm

4.3.2 Chloride and pH Profiles

The colour code introduced in Section 3.2.3.2 is applied to the compiled chloride and pH profiles measured from the Approach Structures presented in Table 4-3.

The chloride profile indicates that de-icing salts are applied in the Northern (Lulu) Approach. Elevated chloride levels are also observed from the core taken from the Southern (Deas) Approach. Chloride contents in the approach concrete are on average higher and have penetrated deeper than what is seen in the tunnel concrete as shown in Table 3-8.

Table 4-3: Chloride and pH profiles measured from select cores from the approach ramps.

Core ID	Concrete Type/Location		Depth (mm)	Chloride content (%wt. concrete)	pH
NAE-C7	Northern (Lulu) Approach Ramp	Element A10 Curb concrete	0-15	0.078	12.3
			15-30	0.215	12.5
			30-45	0.133	12.5
			45-60	0.132	12.6
			60-75	0.084	12.6
SAE-C8	Southern (Deas) Approach Ramp	Element A10 East wall ~90 cm over roadway level	0-15	0.154	12.5
			15-30	0.172	12.5
			30-45	0.105	12.5
			45-60	0.065	12.6
			60-75	0.040	12.6

4.3.3 Petrographic Analysis

Petrographic examination was conducted on the cores NAE-P7 and SAE-C8 extracted from the approach ramp structures. Petrography reports, included in Appendix C, document that both cores consisted of concrete that was non-air-entrained, generally dense and well-consolidated with well-hydrated cement paste. Table 4-4 provides an overview of details on the two cores with the summary text from the original petrography reports. Bold text in the summary column is added to emphasize the observed defects.

As indicated in Table 4-4, both cores show signs of ASR. Due to observed ASR in the cores, damage rating indices were determined for the cores, which are presented in Section C.1 of Appendix C.

Table 4-4: Summaries from petrography reports for Approach Ramp cores.

Core ID	Concrete type/Location & Notes		Summary
NAE-P7	Northern (Lulu) Approach Ramp	Top of Element A5E retaining wall (East side)	The concrete contains a variety of **open and fine cracks, some lined/filled with ASR gel**, some devoid of fillings. **Reaction rims are common** on some particles. **ASR gel fills some voids**.
SAE-C8	Southern (Deas) Approach Ramp	Element A10, East wall, roadway level	The concrete is well-consolidated with adequate proportioning and distribution of aggregates in the paste matrix. **Paste interface** is generally good, except in **localized zones where debonding of aggregate has occurred**. The **outer 5-15 mm** of the concrete is characterized by **carbonated** paste. **Signs of Alkali-Silica Reaction (ASR) are observed throughout the sample** and include(a) reaction rims on some aggregates, (b) presence of ASR gel in cracks (c) cracks in aggregate and in paste (d) localized debonding of aggregate from paste.

Core NAE-P7, collected from the same location in the Northern (Lulu) Approach as the core "A5" reported in [1], was found to contain numerous ASR-related features through visual inspection and petrographic analysis. Figure 4-4 shows core NAE-P7 with signs of AAR observed including white gel deposits on horizontal cracks encountered by the core and reaction rims around aggregates. Petrographic analysis of the core confirms the presence of ASR gel filling air void, cracks in aggregate and paste, and at the interface between the two. It is noted that the extensive cracking observed along the top of the retaining walls (see Figure 4-2) may not be exclusively caused by ASR and freeze-thaw cycling of the non-air-entrained concrete is probably a major contributing factor.

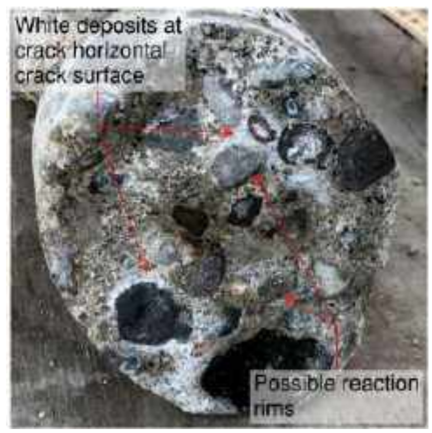
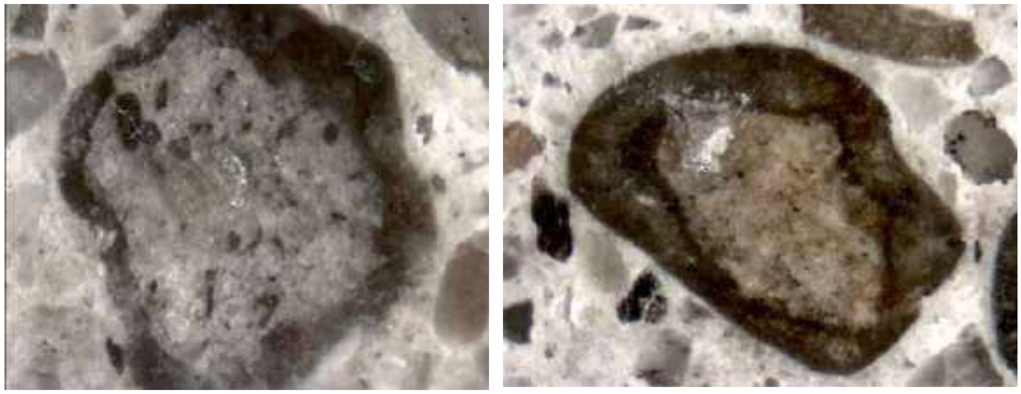


Figure 4-4 Surface of a horizontal crack in Core NAE-P7 from Northern (Lulu) Approach Ramp

Core SAE-C8, collected near the roadway level of the Southern (Deas) Approach, was taken as an example of the approach ramp concrete without outward signs of ASR induced deterioration or other visible cracking. Petrographic analysis reports similar signs of ASR are observed throughout the sample. A further observation of interest from this core is shown in Figure 4-5. At two locations, ASR gel was found to have formed on the surface of the sample between polishing of the surface (completed on a Friday) and determination of the damage rating index (completed on the following Tuesday). This observation may indicate ASR is still active in this core.



(a) (b)

Figure 4-5: Photos showing ASR gel that formed on aggregates with reaction rims in the laboratory subsequent surface polishing of the surface: (a) 38x mag., field of view approx. 3 mm, (b) 25x mag., field of view approx. 4 mm.

Both cores therefore provide indication that ASR is acting at the top and near the roadway level of the retaining walls (possibly to a similar degree as discussed in Appendix C). The outward appearance of the two locations however differs. Freeze/thaw damage and ASR therefore appear to both be acting on the upper portions of the retaining walls. It is possible that the water ponded on the flat top surface of the retaining wall entered into fine cracks induced by ASR. Subsequent freezing and thawing of the water in these fine cracks may then cause propagation of cracks, leading to the increased visual damage typically seen at the top of the retaining walls.

4.4 Comparisons with 2000/2001 Inspection Findings

While not a conclusive method to assess the advancement of ASR, comparative photographs were taken of images presented in [1] as shown in Figure 4-6 to provide a qualitative assessment of progression of ASR. There are modest indications of progression of the deterioration near the vertical cables, with additional gel and/or efflorescence seen in the image from 2020 on the concrete surface adjacent to the vertical cables.

Core NAE-P7 was collected from approximately 300 mm to the north of Core "A5" presented in [1] as an additional assessment of ASR progression in the 20 years subsequent to findings presented in [1]. Damage ratings presented in Appendix C indicate a limited proliferation of ASR has occurred at the core location since 2000.

(a)

(b)

Figure 4-6: Comparison of photo 32 from [1] with (a) showing the condition during the current inspection and (b) the condition during the 2000 inspection – Northern (Lulu) Approach, Element A1, West wall.

4.5 Watertightness

The joint between the Southern (Deas) Approach and ventilation building was observed to have a significant leak, with the leak appearing to originate from damage in the joint seal in the eastern wall. As shown in Figure 4-7, the joint seal is locally pulled out and broken as seen in Figure 4-7(b).

During the inspection, rehabilitation of joints in the base slab was ongoing. While not observed during the inspection, our team's experience as users of the tunnel indicates that some of these joints were sites of leakage in the past.

Leaks were not observed through the concrete elements.

(a) *(b)*

Figure 4-7: Joint leakage at site of damage to the joint seal, with (a) providing an overview and (b) showing the area of joint seal pulled out and broken.

5 Discussion and Future Work

5.1 Discussion of Immersed Tunnel Condition

Generally, results and observations from the immersed tunnel indicate that two reinforced concrete deterioration processes are ongoing that negatively affect the future service life of the tunnel: alkali-silica reaction (ASR) and reinforcement corrosion. Water leakage, which can exacerbate these and freeze-thaw damage, was also observed. Outward indications of these deterioration processes are not widespread, with significant areas of the tunnel not showing outwards signs of deterioration.

5.1.1 Alkali-Silica Reaction (ASR)

ASR in the joint infill concrete and precast tunnel concrete is confirmed, with variability in the extent of ASR observed in the cores. The precast tunnel concrete shows minor signs of ASR, while the infill joint concrete was observed to have more significant signs of ASR (particularly at the outer wall) that do not yet appear to significantly impact its compressive strength. Other mechanical properties that may be more readily affected by ASR than compressive strength (e.g., tensile strength and elastic modulus) were not evaluated.

Forecasting of future course of ASR damage through analysis and evaluation is not possible with the available data. No significant signs of ASR were observed 20 years ago in the precast tunnel concrete and infill joint concrete [1], whereas clear signs of ASR were observed from all cores for petrographic analysis extracted from the tunnel during this investigation. Therefore, there is a likelihood that ASR has progressed in the last 20 years. As discussed further in Section 5.3, industry practice calls for longer-term study should a prognosis on ASR development be desired.

The cores used for petrographic analyses were subjected to a detailed aggregate lithology examination with the aims of potentially ascertaining:

- Insight on the content of potentially reactive aggregate in the concrete types represented by extracted cores,
- Insight on the field performance history of the aggregates used, and
- Confirmation of the aggregates' provenance.

The resulting Aggregate Provenance Evaluation is included in Section C.3 of Appendix C, which confirms that discernable differences exist in the aggregates found in concrete from the precast elements, approach ramps, and infill joints. However, the differences are not strongly apparent and are somewhat inconsistent.

Results of the lithology examination indicate that the content of potential reactivity aggregate is significant in all cores.

Comparison of the observed lithology of aggregates in the concrete cores with knowledge of aggregate sources in British Columbia and review of geological reports on the South Coast does not support a conclusion on the precise provenance of the aggregates. However, as noted in the appended report, "*review and research on aggregate sources indicates that the Seymour Deeks-McBridge and Gilleys Bros May Hill pits mentioned in the Christiani and Nielsen report are among the possible sources.*"

Based on the lithology examination and petrography reports, the future performance of the concretes represented by the cores is not expected vary significantly (assuming a common exposure situation). As potentially reactive aggregates are present, future expansion will be controlled by exposure conditions and the amount of available residual alkali in the concrete paste. It remains unclear whether ASR-induced deterioration can be managed over an additional 50 years of service. Should ASR stabilize or progress very slowly, then this may not be a significant issue for the ongoing use of the tunnel. However, if ASR develops to a level similar to what is observed on the approach ramps, then this can become unmanageable deterioration and, potentially, a safety issue.

Options to mitigate ASR are limited in this situation. A common method to slow down ASR in structures is to reduce the exposure to moisture. This is not considered practical in an immersed tunnel with exposure to water on its outer surface and high humidity internally. The use of lithium salts systems to control ASR is not well proven. Repair and rehabilitation using conventional methods will not stop ASR although could help with slowing down the process depending on how it is performed. For example, as indications are that ASR is more significant in the infill joints, the feasibility of replacement of the entirety or significant portion of the infill joint concrete could be investigated (since there may be an external seal in place from the original submersion process of the tunnel segments). Replacement or rehabilitation of the precast tunnel elements is not considered possible if more significant signs and impact of ASR occur in the future.

Additional discussion on possible monitoring and management of damage induced by ASR is provided in Section 5.3.

5.1.2 Reinforcement Corrosion

Chloride-induced reinforcement corrosion is confirmed, with chloride levels sufficient to initiate corrosion and signs of corrosion damage were observed. If left untreated, chlorides levels in concrete are expected to increase with time and promote further and more widespread corrosion.

Carbonation-induced reinforcement corrosion is confirmed in the infill concrete as the carbonation depth has reached the design cover thickness and beyond, and signs of corrosion damage were observed. Carbonation-induced corrosion is not a concern for the next 50 years for the precast tunnel concrete based on observed carbonation depths : the carbonation front generally advances in proportion to the square root of time, extrapolation of carbonation depth data show that penetration front is expected to remain less than the specified cover over the next 50 years.

Observed outward signs of reinforcement corrosion-induced deterioration are not widespread. Observations in the 2000 report [1] also described localized spalling of cover over corroding rebar in the outer walls in both air ducts, however a detailed mapping of locations showing outward signs of corrosion in 2000 is not included in [1]. It was therefore not possible to quantify a progression rate of observed corrosion-induced deterioration. Based on comparison of descriptions in [1] and present observations, it appears the corrosion process has progressed slowly between 2000 and 2020. The significance of comparisons to the 2000 observations is limited considering the roadway tubes (with generally more significant chloride ingress) were not part of the 2000 investigation.

Chloride profiles and carbonation depths indicate that initiation of reinforcement corrosion could occur at additional locations in the future. Generally in the following 50 year period, corrosion will likely initiate at more locations and, where corrosion has already initiated, cracks and spalls will appear in increasing amount. The time between corrosion initiation and cracking/spalling is highly variable and can range from a few years to 10 years or more. In cases where the corrosion is not widespread, such as is believed to be the case currently in the tunnel, repair and rehabilitation using conventional methods can slow the process and keep the deterioration to manageable levels for extended period of time. For GMT, proactive repairs and planned rehabilitation programs could keep corrosion-induced deterioration to manageable levels for a 50-year period.

It is noted that the condition of the outer layers of reinforcement in the exterior walls was not assessed as part of this inspection and constitute a limitation to this study.

5.1.3 Water Leakage

In addition to deterioration of the structural reinforced concrete, observations provide indications of failures in the waterproofing membrane, including leaking cracks and water dripping from pipes connected to the drain system in transverse construction joints as described in Section 3.1.1.

The drain detail utilized at the transverse construction joints may have had a good intention of collecting leaking water; however, allowing for leaks through construction joints is now known to be counter-productive and presenting significant concerns for corrosion of the reinforcement crossing the joints.

Injecting cracks and leaking joints on a regular basis could keep the leaks under control and help control the deterioration rates of the tunnel.

5.2 Discussion of Approach Structures Condition

The Approach Ramps are generally showing more advanced deterioration, mainly from ASR, than the immersed tunnel, particularly along the top 2-3 m of the retaining walls.

Results and observations from the approach structures provide indications of similar ongoing deterioration processes as were observed in the immersed tunnel including ASR and reinforcement corrosion. Water leakage was also observed at joints and freeze/thaw attack appears to be contributing to the deterioration of the top 2-3 m of the retaining walls.

5.2.1 Alkali-Silica Reaction (ASR) with Local Freeze/Thaw Attack

Petrographic analyses have confirmed ASR in concrete both from the top of the retaining wall (where ASR was already observed in 2000) and at the roadway level. Indications from the two cores collected is that a similar extent of ASR has occurred at both locations. Freeze/thaw attack is likely contributing to the more advanced deterioration observed in upper portions of the retaining wall.

Signs of propagation of ASR-induced damage on the approach ramps were inconclusive as to whether damage is increasing or staying the same based on comparison of data between 2000 and 2020. Additional discussion on indications of the progression of ASR is found in Appendix C. Similar to the tunnel, analysis or evaluation to forecast the progression of ASR cannot be completed with currently available information and it therefore remains unclear whether ASR-induced deterioration can be managed over an additional 50 years of service. Industry practice calls for monitoring should a prognosis be desired, as discussed in Section 5.3.

The top 2-3 m of the wall is showing more advance deterioration and can reasonably be expected to require replacement within the next 50 years, most likely in the next 10 to 20 years.

5.2.2 Reinforcement Corrosion

Chloride-induced reinforcement corrosion is confirmed, with the retaining wall and curb adjacent to the roadway containing sufficient chloride at the level of the reinforcement to initiate corrosion. Localized signs of corrosion were observed in the northbound lanes of the approach ramp include in the retaining walls and the base slab. If left untreated, chlorides levels in concrete are expected to increase with time and more widespread corrosion and/or increased corrosion-induced damages to the concrete.

Carbonation of the concrete is not yet problematic and is not expected to be for the next 50 years.

Repair and rehabilitation using conventional methods can slow the deterioration process and keep the associated damage to manageable levels for an extended period of time. Proactive repairs and planned rehabilitation programs could keep deterioration to manageable levels for a 50 years period.

5.2.3 Water Leakage

Significant leaks were observed between the ventilation building and the approach ramps. The leaks were generally associated with damaged waterstops at joints. If left untreated, persistent exposure to leaking groundwater will promote accelerated deterioration due to reinforcement corrosion, freeze/thaw attack and ASR.

5.3 Future Work

This study has identified two primary modes of deterioration in the structural reinforced concrete components of the tunnel structure: corrosion and alkali-silica reactions (ASR). Leaks through cracks and/or joints in the structural reinforced concrete were observed both in the immersed tunnel and the approach ramps, which has critical implications on durability. Of secondary importance, observations indicate freeze/thaw attack is likely a participatory deterioration mechanism in the approach ramps.

The inspection completed to inform this study was not a detailed condition inspection and therefore close scrutinization of all surfaces of reinforced concrete was not completed. The deterioration mechanisms acting on the reinforced concrete components of the existing immersed tunnel and approach ramps are identified herein; however, the extents (i.e., total area) impacted by these deterioration processes was not determined.

Further, leaks are present in the tunnel and approach ramps, which can promote additional deterioration. The current corrosion and leaking situations are considered manageable through rehabilitation as listed in Table 5-2, injection of cracks and construction joints and continued maintenance, and the future rate of increase in both can be assessed through routine inspections.

Note that corrosion-induced spalling of concrete was observed from the soffit of the Northbound Roadway Tube, which presents a potential safety concern for the travelling public (the spalling concrete can fall into traffic). This should be addressed, and a Detailed Condition Inspection should be completed to assess whether additional areas of the soffit are delaminated or otherwise at risk for spalling.

The current level of ASR damage is not critical to the integrity and safety of the structure, however due to a lack of historical information about the levels of ASR damage in the tunnel, there is an uncertainty regarding the future expansion rate of ASR damage and its effect on the structural properties of the concrete. If ASR develops to a significant degree, there is little that can be done to slow or remedy the deterioration other than to remove and replace the affected components once they reach an unacceptable level of damage. This is not considered feasible for the tunnel elements although replacement of infill joint concrete may be feasible.

Although guidance is available for laboratory and in-situ investigations to estimate the potential for further expansion from ASR [14], these methodologies rely on exposing samples of the concrete for up to a year under laboratory conditions or even longer term in-situ investigations. Assessment of further expansion from ASR is also considered to be a developing field of materials engineering and therefore, the reliability of these methods is not well established. Based on the current state of knowledge, long term monitoring of ASR symptoms, using techniques that result in quantifiable parameters, is required over a period of at least 3 to 5 years, preferably longer. This commonly involves techniques such as the Damage Rating index, Field Damage Index, Cracking Index, Stiffness Damage Test and conventional testing of tensile strength and elastic modulus at intervals of 1 to 3 years [14]. Once the progression of such parameters is observed over a sufficient period of time it then becomes possible to extrapolate the results and make prognostications on the development of future deterioration, though as mentioned, the reliability of such prognostications is not well established.

Due to the lack of information available about the progression rate of the ASR in the tunnel, at this time it is not possible to determine if the existing tunnel has a remaining service life of 50 years. It is possible that the ASR damage will not be decisive in limiting the service life during the next 50 years, however there is insufficient data to support a conclusion of the remaining service life of the tunnel, and therefore it is not possible to conclude whether the existing tunnel has a remaining service life of 50 years or not.

The controlling condition or situation that is considered by MoTI to signify the end of the service life of the GMT, which is potentially not controlled by the condition of the structure reinforced concrete, will also strongly impact the ability of the structure to remain in service for an additional 50 years. Similarly, the extent to which an appropriate maintenance and rehabilitation plan is funded and carried out will also strongly influence damage progression from the reinforced concrete deterioration mechanisms observed.

The following sections provide overview of commonly used investigations, inspection & monitoring and rehabilitation & maintenance per the following industry standards, guides, and manuals to help maximize the service life of the tunnel:

- BC MoTI Bridge Inspection Condition: Field Book 2013 [20];
- FHWA Tunnel Operations, Maintenance, Inspection, and Evaluation (TOMIE) Manual [21];
- CSA A864 Guide to the Evaluation and Management of Concrete Structures Affected by Alkali-Aggregate Reaction [16];
- FHWA Report on the Diagnosis, Prognosis, and Mitigation of Alkali-Aggregate Reaction (ASR) in Transportation Structures [14]; and
- Report on rehabilitation in tunnels prepared for the AASHTO Technical committee for tunnel entitled "Development of Guidelines for Rehabilitation of Existing Highway and Rail Transit Tunnels" [22].

5.3.1 Investigations, Inspections & Monitoring

Table 5-1 provides an overview of common industry practice investigations, inspection and monitoring procedures that may be applied to GMT to further assess its current condition and to gain further insight on deterioration rates and prognosis. The scope and frequency of the various condition assessment approaches are included in the table. Typical conditions inspections from [20] and [21] are listed first, followed by specific investigations and monitoring methodologies to assess progression of ASR-induced damages. It is noted that the following discussion is based on current standards, some of which are not specifically developed for immersed tunnels. As such, a more detailed program for investigations, inspections and monitoring should be elaborated.

A Detailed Condition Inspection of the tunnel (all four tubes) and approach ramps, typically to be completed once every 5 years [20], would be the first task per industry practice. The level of visible damage would be quantified, which would provide information for the development of an initial rehabilitation program to repair deteriorated areas. Detailed inspections are typically performed using access equipment where the engineer can observe closely all surfaces to document cracks, delamination, spalls, and other defects. CST noted that the concrete surfaces were dirty, particularly in the roadway tubes, making observations of cracks and defects difficult. Therefore, cleaning of concrete surfaces prior to the detailed inspection would facilitate the observation of defects. The observed spalling of concrete from the soffit of the Northbound Roadway Tube shall be a focus of this Detailed Condition Inspection to assess whether additional areas of the soffit are delaminated or otherwise at risk for spalling.

For years in which a Detailed Condition Inspection is not completed, a Routine Condition Inspection should be completed per [20].

Partial Condition or Damage Inspections should be completed per [20] and [21] on an ad-hoc basis in response to specific events.

Special Inspections, described in [21], are typically performed after another inspection type discovers significant deficiencies that warrant monitoring. The following deficiencies observed could be monitored through Special Inspections to gain further insights on their significance and effect on the service life of the tunnel:

- Transverse cracking;
- Leakage and collection of water through transverse cracks, transverse construction joints and infill joints in the tunnel and leakage of water at movement joints in the approach ramps; and
- Ongoing ASR in the structural reinforced concretes at the approach ramps and in the immersed tunnel.

A number of transverse cracks were observed during the inspection presented herein as well as the 2000 inspection, with site visits completed for both assessments during warmer months (between July and October). A Detailed Condition Inspection would help produce a map of transverse cracks; however, the number and width of observable cracks may be influenced by the time of year during which the inspection is completed. Repeated wintertime inspection and temperature measurements (both ambient and internal concrete temperature) would help assess possible increases in the number/width of cracks due to possible thermal contraction of the tunnel during colder months. Increasing width of transverse cracks with time (i.e., across several years) may be an indication of local cross-sectional reductions in the longitudinal reinforcement. Repeated visual inspections should therefore be completed at least during winter months on a yearly basis. Further, a common approach to more accurately and frequently quantify movements in the tunnel is to install and monitor crack gauges or similar at select transverse cracks, transverse construction joints and infill joints.

Likewise, the presence and extent of leaks can be impacted by temperature changes throughout the year. Leaks and the level of water collected in the air duct tubes can be monitored to ascertain a more complete picture of leaks and, assess the source of the water collecting (particularly in the Upstream Air Duct Tube), and verify that the level of collected water is consistent. A less concerning source for the water would be rain water entering through the tunnel portals; however, if suspicion is raised that the source of collected water is (partly) from leakage through transverse or horizontal construction joints, prompt injection of the joint should be completed to minimize long-term impacts. Special Inspection of leaks can continue after repair/injection discussed in the following section, until the leaks are observed to be stopped at which time Routine Condition Inspections would provide indication of new/reopened leaks.

The final row of Table 5-1 provides basic information on an ASR-specific investigation, referred to as a FHWA Level 3 Detailed Investigation Program [1]. A complete FHWA Level 3 Detailed Investigation Program is not developed here. Supplementary evaluations of concrete cores and monitoring of the concrete structure can help assess the likely progression of ASR and its impact on the structural capacity and watertightness of the tunnel and approach ramps. A key aspect of this process is to ascertain the current expansion rate from ASR. One such in-situ method to estimate the expansion rate, described in Appendix B of [14], involves setting up and monitoring the "Cracking Index" measured at several locations on the structure for a number of years. Once a potential for further expansion is found, an analysis of the impact this expansion has on the structure may be completed. It is noted that a typical approach taken on other structures of significance impacted by ASR (e.g., dams) is to form a review panel of experts in both ASR and the structure type to aid in guiding investigation and monitoring of the structure as well as in the interpretation of results. Additionally, if other concrete structures that utilized the same aggregate sources can be identified, a field performance history as described in [24] could provide additional insights.

Additionally, future investigations of the tunnel may include core through the entire thickness of the outer wall of the immersed tunnel to obtain samples of and assess the condition of the outermost layer of reinforcement and waterproofing membrane.

Table 5-1: Overview of condition assessment processes from common industry practice.

Condition Assessment Type	Scope	Frequency
Routine Condition Inspection [20]* or Routine Inspection [21]	Combination of hands on and visual inspection (where access is difficult) [20].	Not less than once per year [20]
Detailed Condition Inspection [20]* or In-Depth Inspection [21]	Close-up, hands on inspection of all parts of a structure, aided by access equipment. May be supplemented with physical testing.	5-year interval typically appropriate [20]
Partial Condition Inspection [20]* or Damage Inspection [21]	Performed on selected component(s) due to significant condition change due to collision damage or rehabilitation [20]. Damage from motor vehicle impact, fire, flood, earthquake, vandalism, or explosion [21].	No pre-set frequency, as needed in response to natural disasters or human activities.
Special Inspection [21]	Inspection completed when significant deficiencies are discovered during other types of inspection.	Continue at adjusted interval until deficiency is repaired.
ASR Specific Investigations:	Extensive sampling and laboratory investigation can include: › Petrography › Mechanical testing	Frequency of complete detailed investigation program not stated.

Condition Assessment Type	Scope	Frequency
FHWA Level 3 Detailed Investigation Program [14] or Assessment of Future Behavior of the Concrete Structure [16]**	› Expansion tests on cores › Water soluble alkali content of concrete In-situ investigations that can include: › Surface crack mapping › Monitoring of expansion/movements › Assessment of internal stresses by overcoring method or stress measurement from removed reinforcing bar (describe in Section 5.2.4 of [14]) › Temperature and humidity measurement (from air/ambient and internally in concrete) The laboratory and in-situ investigation results can be used in collective assessments on the need for mitigation. Collective assessments can include: › Expansion to date (in-situ estimated/monitored expansion, surface cracking, stiffness damage testing) › Current expansion rate (in-situ monitoring, core expansion, surface cracking) › Potential for further expansion due to ASR › Structural integrity › Public safety › Effect of other mechanisms on progress of deterioration. Section 5.0 of [21] provides additional detail on scope and methods.	Once every 10 years appears appropriate based on maximum extent of extrapolation. As a general guideline, in-situ monitoring of crack index is repeated yearly for 3-5 years and then every 5 years if evolution is slow/nil.

* It is noted that text in [20] "*was written for Pedestrian Tunnel Inspections*". The FHWA TOMIE Manual [21] provides descriptions of similar inspection types, also listed in the table, with additional context regarding inspection of highway tunnels.
** A somewhat comparable investigation program is provided in Section 7 of [16]. The descriptions here focus on the process from [14], which reflects current state-of-the-industry practices.

5.3.2 Rehabilitation & Maintenance

Table 5-2 provides an overview of the currently anticipated scope and frequency of maintenance and rehabilitation of structural reinforced concrete in the existing GMT. Maintenance, repairs and rehabilitation should be performed following industry best practices including e.g., [21] and [22]. Repairs should be performed as recommended by Routine and Detailed Condition Inspection reports. Additional information on typical maintenance operations for immersed tunnels, included in [15], is also considered in the following.

An Initial Rehabilitation Program is expected to repair current damaged areas. An indicative scope for this program is included in the table, consisting of basic types of rehabilitation that are anticipated based on observed deterioration reported herein. The extent of areas needing rehabilitation is not currently known due to the limited information available; a Detailed Condition Inspection would inform the development of an Initial Rehabilitation Program. As part of the development of this Initial Rehabilitation Program, cores should be drilled from sites of leaking cracks that include or at minimum expose reinforcement to assess the conditions and degree of corrosion at leaking or previously leaking cracks. Cracks and construction joints currently leaking should be injected promptly during winter months with an appropriate injection material (e.g., acrylic gel) to mitigate potential accelerated deterioration at leak sites. As described in Section 5.3.1, repaired leaks should be subject to additional Special Inspection to verify repair and subsequent Routine/Detailed Condition Inspections to check for signs of new or restarted leaks (which should also be promptly injected).

Additionally, possible mitigation measures could be implemented as part of the rehabilitation program. One potentially suitable and useful mitigation to extend service life could involve cleaning and applying a silane sealer on roadway tube walls in an effort to slow ingress of additional chlorides, together with the aforementioned injection of leaking cracks. The sealer would need reapplication at intervals of 6-8 years to maintain its effectiveness.

Table 5-2: Overview of maintenance and rehabilitation processes for highway tunnels, based on [15], [21] and [22].

Activity	Component	Scope	Est. Frequency
Initial/Future Rehabilitation Program	Precast Tunnel Elements Construction Joints Infill joint concrete Approach Ramps	Injection of leaking cracks and/or construction joints Repair of 'dry' cracks Repair of delaminated areas and spalls Repair of concrete damages from impact, mechanical wear (Possible) application of silane sealer in roadway tube walls	Generally, approx. every 20 years Silane sealer reapplication every 6-8 years.
Maintenance/ Repair	Precast Tunnel Elements Construction Joints Infill joint concrete Approach Ramps	Simple/minor repair of concrete Reinjection of observed leaks Reinjection of leaking cracks	As require per Condition Inspection observations, min. once per year

6 Acknowledgements

Input and review of sections of this report related to observations and future assessment of alkali-silica reaction (ASR) by Christopher Rogers, an internationally recognized expert on the diagnosis and prognosis of ASR in reinforced concrete structures is gratefully acknowledged.

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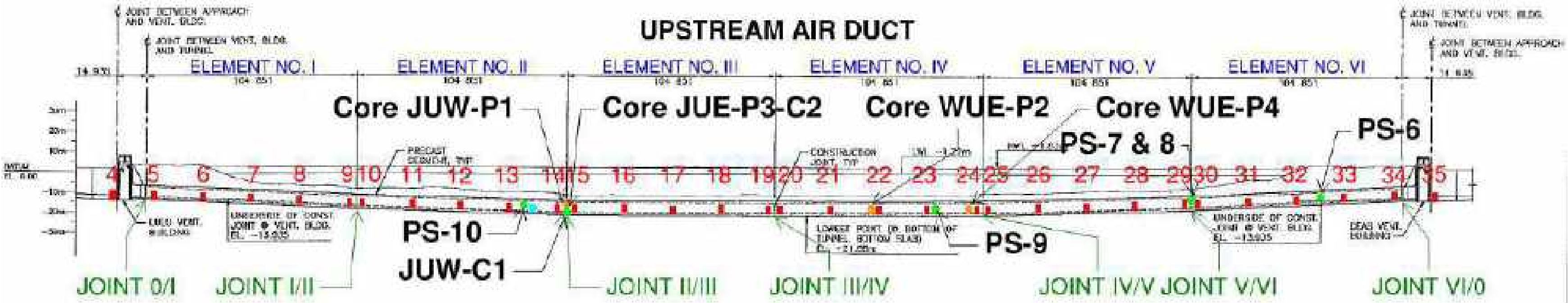
Appendix A Field Observations

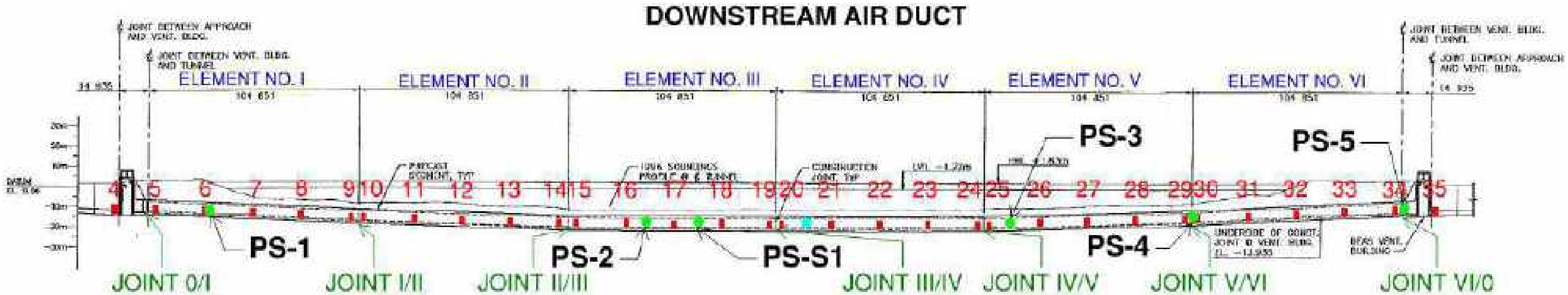
The site inspection was conducted by Brad J. Pease, Neil Cumming, and Fatemeh Alapour from COWI between 27-30 July 2020 and on 23 September 2020.

Weather before and during the July inspection was dry with no accumulation of precipitation (as reported at Vancouver International Airport) for two weeks prior to the inspection. Outdoor temperatures varied from 13-25° C during July inspection dates. The September walkthrough took place during a heavy rain event, with 29.6 mm of rainfall accumulated on September 23. Outdoor temperatures varied from 13-20° C on September 23.

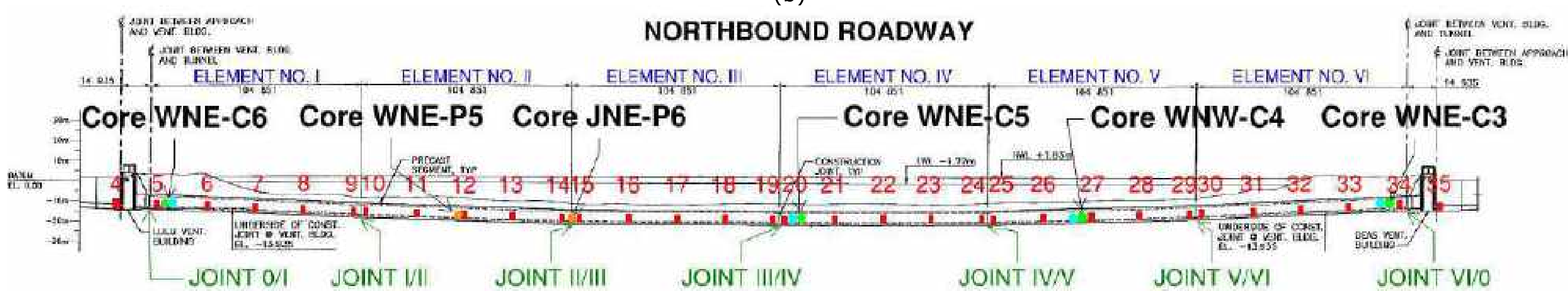
A.1 Overview of Sample and Testing Locations


UPSTREAM AIR DUCT
ELEMENT NO. I
ELEMENT NO. II
ELEMENT NO. III
ELEMENT NO. IV
ELEMENT NO. V
ELEMENT NO. VI
Core JUW-P1
Core JUE-P3-C2
Core WUE-P2
Core WUE-P4
PS-7 & 8
PS-6
PS-10
JUW-C1
PS-9
JOINT 0/I
JOINT I/II
JOINT II/III
JOINT III/IV
JOINT IV/V
JOINT V/VI
JOINT VI/0

(a)


DOWNSTREAM AIR DUCT
ELEMENT NO. I
ELEMENT NO. II
ELEMENT NO. III
ELEMENT NO. IV
ELEMENT NO. V
ELEMENT NO. VI
PS-3
PS-5
PS-1
PS-2
PS-S1
PS-4
JOINT 0/I
JOINT I/II
JOINT II/III
JOINT III/IV
JOINT IV/V
JOINT V/VI
JOINT VI/0

(b)


NORTHBOUND ROADWAY
ELEMENT NO. I
ELEMENT NO. II
ELEMENT NO. III
ELEMENT NO. IV
ELEMENT NO. V
ELEMENT NO. VI
Core WNE-C6
Core WNE-P5
Core JNE-P6
Core WNE-C5
Core WNW-C4
Core WNE-C3
JOINT 0/I
JOINT I/II
JOINT II/III
JOINT III/IV
JOINT IV/V
JOINT V/VI
JOINT VI/0

(c)

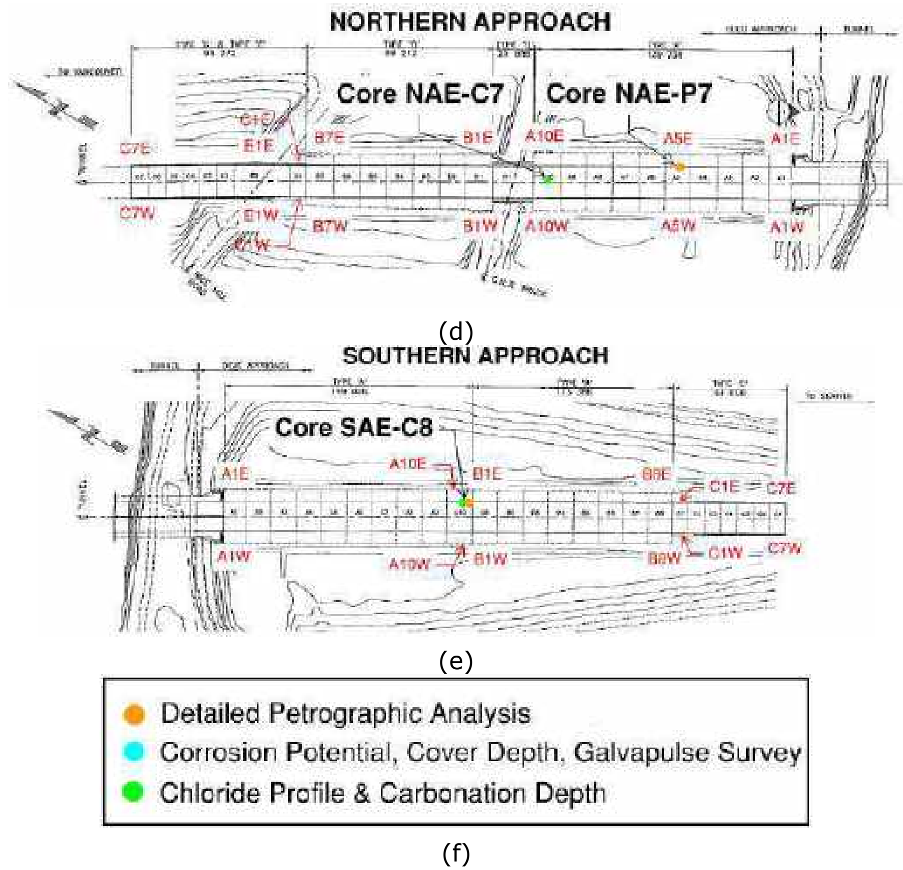


Figure A-1: Indicative locations of field testing and sampling in (a) the Upstream Air Duct, (b) the Downstream Air Duct, (c) the Northbound Roadway Tube, (d) the Northern (Lulu) Approach, (e) the Southern (Deas) Approach, and (f) provides a legend of the testing conducted.

Table A-1: Overview of details on collected cores and samples including locations, observation notes, field and laboratory tests methods used, and description of situation this location represents.

Core/Sample ID	Concrete Type/Location		Description of Core/Sample			General Observations Notes on Location	Tests methods used	Sample/Location Representative of:
			Core Length	Crack?	Rebar?			
JUW-P1 (core)	Infill joint concrete – Intermediate wall	Upstream Air Duct Joint II/III	90-100 mm	No	No	Large area of delamination in joint.	Chloride profile & carbonation depth	Infill joint at Intermediate wall Air duct exposure Near delamination
JUW-C1 (core)		Upstream Air Duct Joint II/III	~100 mm	No	No	Large area of delamination in joint. Companion core to JUW-P1	Petrographic analysis	Infill joint at Intermediate wall Air duct exposure Near delamination
JNE-P6 (core)		Northbound Roadway Tube Joint II/III ~120-130 cm above roadway level	~60 mm plus 30 mm of shards	Delami nation	No	Audible delamination at core location. Horizontal delamination observed at ~6 cm depth, with ~10 mm gap at delamination.	Petrographic analysis	Infill joint at Intermediate wall Roadway tube exposure Delaminated concrete
WNE-C6 (core)		Northbound Roadway Tube Joint 0/I ~125 cm over roadway level	80-90 mm	No	No*	Element joint near exit of tunnel * Rebar was encountered at core used to expose rebar for NDT. No signs of corrosion.	Chloride profile & carbonation depth Corrosion potential, Galvapulse, and cover measurement survey	Infill joint at intermediate wall Roadway exposure

Core/Sample ID	Concrete Type/Location		Description of Core/Sample			General Observations Notes on Location	Tests methods used	Sample/Location Representative of:
			Core Length	Crack?	Rebar?			
JUE-P3-C2 (core)	Infill joint Concrete - Outer wall	Upstream Air Duct Tube Joint II/III	~300 mm	Transverse Crack	No	Transverse crack in joint, dry and without efflorescence Clear color change near crack in extracted core (likely carbonation) Crack appears to penetrate ~1/2 depth of core	Petrographic analysis Chloride profile & carbonation depth	Infill joint at outer wall Air duct exposure Dry transverse crack without efflorescence
WUE-P2 (core)	Precast Tunnel Concrete	Upstream Air Duct Element IV, Door 22 Outer (East) wall	80 mm	No	No	Core near delamination site for petrographic analysis Rims seen on 2 aggregate particles (photos taken)	Petrographic analysis	Precast Tunnel Concrete Air duct exposure Near spalled concrete
WUE-P4 (core)		Upstream Air Duct Tube Element IV, Door 24, Outer (East) wall	~300 mm	No	Yes	Fine map cracks observed on concrete surface 4" core started but encountered rebar and therefore core continued with a 3" core. No signs of corrosion. Some rims on aggregates observed.	Petrographic analysis	Precast Tunnel Concrete Air duct exposure Fine map cracking
WNE-P5 (core)		Northbound Roadway Tube Element II, Door 12 Intermediate Wall ~110 cm over roadway level	~80-100 mm	No	No	Petrographic core at area of interest in northbound roadway tube, possibly fire damage. Flexible coating, ~1 mm thick on concrete surface	Petrographic analysis	Precast Tunnel Concrete Roadway exposure Possible fire damage site

Core/Sample ID	Concrete Type/Location		Description of Core/Sample			General Observations Notes on Location	Tests methods used	Sample/Location Representative of:
			Core Length	Crack?	Rebar?			
WNE-C3 (core)		Northbound Roadway Tube Element VI, Door 34 Intermediate wall 85-90 cm over roadway level	~130 mm	No	No*	Intact concrete at tunnel entry * Rebar was encountered at core used to expose rebar for NDT. Minimal signs of corrosion on exposed bar. Coating/paint on core surface.	Chloride profile & carbonation depth Corrosion potential, Galvapulse, and cover measurement survey	Precast Tunnel Concrete Roadway exposure Intact concrete at tunnel entry
WNW-C4 (core)		Northbound Roadway Tube Element V, Door 27 Intermediate wall ~130 cm over roadway level	~90 mm	No	No*	Intact concrete at ¼ length into tunnel * Rebar was encountered at core used to expose rebar for NDT. Low cover (~25 mm) and corrosion seen.	Chloride profile & carbonation depth Corrosion potential, Galvapulse, and cover measurement survey	Precast Tunnel Concrete Roadway exposure Intact concrete at ¼ length into tunnel
WNE-C5 (A & B) (cores)		Northbound Roadway Tube Element IV, Door 20 Intermediate wall ~100 cm over roadway level	70-100 mm		No*	Intact concrete at ½ length into tunnel * Rebar was encountered at core used to expose rebar for NDT, some signs of corrosion products. At least 3 materials encountered. Porous brown material at center of core under ~5-20 mm of what may be a steel fiber reinforced concrete. Possibly original concrete at sides of core. Rebar access core (for NDT) also kept (Core B) which seems to be entirely an outer layer of repair material. Determined to complete chloride profile on Core B, which is likely not original concrete and with shorter-term exposure situation.	Chloride profile & carbonation depth Corrosion potential, Galvapulse, and cover measurement survey	Infill joint at Intermediate wall Roadway exposure Intact concrete at ½ length into tunnel Likely a repair material

Core/Sample ID	Concrete Type/Location		Description of Core/Sample			General Observations Notes on Location	Tests methods used	Sample/Location Representative of:
			Core Length	Crack?	Rebar?			
NAE-P7 (core)	Approach Concrete	Northern (Lulu) Approach Element A5E Wall (East side)	290 mm	Vertical and horizontal (transverse) cracks in core	Yes	Vertical core from top of wall immediately next to (~1 ft away from) past core taken in the 2000/2001 study. Core centered on a vertical crack in the wall Core came out in 3 pieces – horizontal cracks in concrete (did not appear to be fractures from coring process) Signs of AAR. White deposits on horizontal cracks in the core. Rebar encountered, smooth bar, ~21 cm deep. Some corrosion stains.	Petrographic analysis	Northern Approach Concrete Atmospheric exposure Same location as core in [1] that detected AAR.
NAE-C7 (core)		Northern (Lulu) Approach Ramp Element A10, Curb concrete	~90 mm	No	Yes*	Core from central curb adjacent to partition wall. * Possibly cast-iron embedment, no active corrosion noted.	Chloride profile & carbonation depth	Curb Concrete Roadway exposure
SAE-C8 (core)		Southern (Deas) Approach Ramp Element A10, East wall	~90 mm	No	No	Indication of AAR at the top of the wall, limited indications lower in wall, core take lower in wall. Some rims and rust colored staining around aggregate and a porous aggregate seen Core cut in half, with half used for tests petrographic analysis and half for chloride & carbonation testing	Petrographic analysis Chloride profile & carbonation depth	Southern Approach Concrete Roadway exposure Typical lower portion of approach wall

Core/Sample ID	Concrete Type/Location		Description of Core/Sample			General Observations Notes on Location	Tests methods used	Sample/Location Representative of:
			Core Length	Crack?	Rebar?			
PS-S1 (concrete powder samples)	Precast Tunnel Concrete	Downstream Air Duct Element III, Door 17/18 Outer (West) wall	N/A	Adjacent to transverse crack	N/A	Powder samples collected for chloride profiles adjacent to transverse crack with efflorescence and near location of spalled concrete.	Chloride profile	Precast Tunnel Concrete Air duct exposure Transverse crack with efflorescence
S2 (efflorescence sample)	N/A	Downstream Air Duct Element II, Door 13 Intermediate wall	N/A	No	N/A	White efflorescence at spall	Chloride content and pH	Precast Tunnel Concrete Air duct exposure White efflorescence at spall
S3 (crystalline sample)	N/A	Downstream Air Duct Element I, Door 6/7 Outer (West) wall	N/A	No	N/A	Crystalline deposits at pipe in outer wall	Chloride content and pH	Precast Tunnel Concrete Air duct exposure Crystalline deposits at pipe in outer wall
S4 (efflorescence sample)	N/A	Southern (Deas) Approach Ramp Element A2, Door 37, East wall	N/A	Vertical crack	N/A	Efflorescence built up at a crack in approach ramp	Chloride content and pH	Southern (Deas) Approach Ramp Efflorescence built up at a crack in approach ramp

Core/Sample ID	Concrete Type/Location		Description of Core/Sample			General Observations Notes on Location	Tests methods used	Sample/Location Representative of:
			Core Length	Crack?	Rebar?			
PS-CL1 (concrete powder samples)	Precast Tunnel Concrete	Downstream Air Duct Element I, Door 6 Outer (West) wall Contraction Pour 2	N/A	No	N/A	Appearance similar to general appearance	Chloride content and pH	Precast Tunnel Concrete, Contraction Pour Air duct exposure General condition
PS-CL2 (concrete powder samples)	Precast Tunnel Concrete	Downstream Air Duct Element III, Door 16/17 Outer (West) wall	N/A	No	N/A	Appearance similar to general appearance	Chloride content and pH	Precast Tunnel Concrete Air duct exposure General condition
PS-CL3 (concrete powder samples)	Precast Tunnel Concrete	Downstream Air Duct Element V, Door 25/26 Outer (West) wall	N/A	No	N/A	Appearance similar to general appearance	Chloride content and pH	Precast Tunnel Concrete Air duct exposure General condition
PS-CL4 (concrete powder samples)	Infill joint Concrete - Outer wall	Downstream Air Duct Joint V/VI Outer (West) wall	N/A	No	N/A	Appearance similar to general appearance	Chloride content and pH	Infill joint at Outer wall Air duct exposure General condition
PS-CL5 (concrete powder samples)	Infill joint concrete – Intermediate wall	Downstream Air Duct Joint VI/0 Intermediate (East) wall	N/A	No	N/A	Appearance similar to general appearance	Chloride content and pH	Infill joint at Intermediate wall Air duct exposure General condition

Core/Sample ID	Concrete Type/Location		Description of Core/Sample			General Observations Notes on Location	Tests methods used	Sample/Location Representative of:
			Core Length	Crack?	Rebar?			
PS-CL6 (concrete powder samples)	Precast Tunnel Concrete	Upstream Air Duct Element VI, Door 32 Outer (East) wall	N/A	No	N/A	Appearance similar to general appearance	Chloride content and pH	Precast Tunnel Concrete Air duct exposure General condition
PS-CL7 (concrete powder samples)	Infill joint Concrete - Outer wall	Upstream Air Duct Joint IV/V Outer (East) wall	N/A	No	N/A	Appearance similar to general appearance	Chloride content and pH	Infill joint at Outer wall Air duct exposure General condition
PS-CL8 (concrete powder samples)	Infill joint concrete – Intermediate wall	Upstream Air Duct Joint IV/V Intermediate (West) wall	N/A	No	N/A	Appearance similar to general appearance	Chloride content and pH	Infill joint at Intermediate wall Air duct exposure General condition
PS-CL9 (concrete powder samples)	Precast Tunnel Concrete	Upstream Air Duct Element IV, Door 23/24 Outer (East) wall	N/A	No	N/A	Appearance similar to general appearance	Chloride content and pH	Precast Tunnel Concrete Air duct exposure General condition
PS-CL10 (concrete powder samples)	Precast Tunnel Concrete	Upstream Air Duct Element II, Door 13/14 Outer (East) wall	N/A	No	N/A	Appearance similar to general appearance	Chloride content and pH	Precast Tunnel Concrete Air duct exposure General condition

A.2 General Observations from Visual Inspection

The table on the following pages provides an overview of observations from the visual inspection. The table is divided by components of the structure (i.e., Immersed Tunnel and Approach Ramps) and location details for the individual observations are provided.

Table A-2: Downstream Air Duct observations

Obs. No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
DS-1	Element I, Door 5	Overview	North end of downstream airduct, looking south	
DS-2	Element I, Door 5/6	Outer (West) Wall	Similar location as in photo #11 of 2000 inspection. Moisture and salt crystallization at bottom of the wall, adjacent to a pipe protruding from outer wall	 

Obs. No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
DS-3	Element I, Door 6/7	Outer (West) wall	Similar location as in photo #11 of 2000 inspection. Pipe protruding from outer wall. Moisture and crystalline deposits. Sample taken (S3)	 
DS-4	Element I	Inner (East) wall	Similar location as photos 5 and 6 in 2000 inspection. Exposed rebar but not concerning.	

Obs. No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
DS-5	Element I	Outer (West) wall	Minor Cracking	

Obs. No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
DS-6	Joint I/II	Outer (West) wall	Sign of delamination	  

Obs. No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
DS-7	Joint I/II	Corner of Outer (West) wall and floor	White crystals collected on floor, possibly chlorides	
DS-8	Element II, Door 12/13	Inner (East) wall	White efflorescence at surface spall	

Obs. No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
DS-9	Element II, Door 13	Inner (East) Wall	White efflorescence at surface spall. Sample collected (S2)	  
DS-10	Element II, Door 14	Outer (West) wall	Map cracking	

Obs. No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
DS-11	Element II	Inner wall	Spall and exposed rebar	
DS-12	Joint II/III	Outer (West) wall		 

Obs. No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
DS-13	Element III, Door 17	Outer (West) wall	Small area of delamination with large cracks	
DS-14	Element III, Door 17	Inner (East) wall	Crack in inner wall at louvre cut out.	

Obs. No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
DS-15	Element III, Door 17/18	Outer (West) wall	Spalled concrete. Cover was approximately 35 mm locally. Powder samples collected (S1). Sample location indicated in annotation.	   

Obs. No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
DS-16	Element III, Door 18	Outer (West) wall	Duplicate photo #16 in 2000 inspection report	
DS-17	Element III, Door 19	Outer (West) wall	Pattern cracking	

Obs. No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
DS-18	Element III	Overview	Looking North into the airduct	 

Obs. No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
DS-19	Element III, Joint III/IV, Sump in Element IV	Outer wall and base slab near sump	Water collected in sump, none in walking areas	 
DS-20	Element IV	Overview near sump	Looking South into the airduct Walkway damp Photo taken in front of sump	

Obs. No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
DS-21	Element IV, Door 20	Outer (West) wall	Crack, ~0.40 mm, no sign of leaks.	 
DS-22	Element IV, Door 20	Outer (West) wall	Duplicate photo #13 in 2000 inspection report In 2000, there was exposed rebar which is patched now	

DS-23	Element IV, Door 20/21	Outer (West) wall	NDT on element IV by WSP ~5 m north of door 21 Rebar exposed at a past core repair location. Likely the location of 2000/2001 LPR measurement. No visual signs of corrosion on exposed rebar.	 

Obs. No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
				  

Obs. No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
DS-24	Element IV, Door 20/21	Outer (West) wall	Area of delamination. Salt deposit nearby.	  

Obs. No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
DS-25	Element IV, Door 23/24	Outer (West) wall	Rust deposit high on west wall. Spot of low cover nearby	  
DS-26	Element IV, Door 23/24	Corner of soffit and outer (west) wall	Corrosion staining, dry	

Obs. No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
DS-27	Element IV, Door 24	Outer (West) wall	Fine cracking near south end of element	 

Obs. No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
DS-28	Element IV, Door 24	Outer (West) wall	Small spall	 
DS-29	Element IV, Door 24, near Joint IV/V	Outer (West) wall	Transverse crack with efflorescence, corrosion staining and weeping near bottom corner of wall.	

Obs. No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
DS-30	Joint IV/V	Ceiling	Efflorescence seen on near corner to soffit and wall, over top of the galvanized steel plate.	 

Obs. No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
DS-31	Element V, Door 29	Outer (West) wall	Corrosion staining, spall, exposed rebar	  

Obs. No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
DS-32	-	Soffit	Typical soffit view	
DS-33	-	Outer (East) wall	Typical outer wall view	
DS-34	-	Outer (East) wall	Typical joint	

Obs. No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
DS-35	-	Top surface base slab	Typical floor	
DS-36	-	Outer (West) wall	Similar to photo #12 of 2000 inspection (site where temporary bulkhead was removed). Minor surface corrosion of cut rebar.	
DS-37	Element V, North of Joint V/VI	Outer (West) wall	Rebar protruding from wall, possibly part of the temporary bulkheads. Horizontal cracking observed in precast tunnel concrete. Cracks were not damp, although had a darker appearance than neighboring uncracked concrete.	

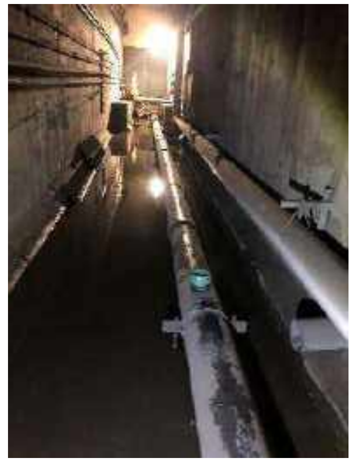
Obs. No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
DS-38	Element VI, Door 31/32	Outer (West) wall	Southern transversal construction joint at CP2 in Element VI (See Figure A-2(b) for CP numbering description). Significant corrosion product observed at construction joint. Corrosion product was damp above the drainpipe, indicating seeping water though the joint.	
DS-39	Element II, Contraction Pour 3 (See Figure A-2(b))	Intermediate (East) wall	Staining on the contraction pour, intermediate wall immediately under the soffit. The staining was not reachable during inspection. On site, the surface stain appeared to be dry (no shining). Similar observations were made at other contraction pours in the Downstream Air Duct including: › CP1, 2, 3 and 6 in Element II; › CP2 in Element III; › CP2-4 in Element IV; abd › CP3 and 6 in Element VI See Figure A-2(b) for contraction pour numbering. No such staining was observed in the Upstream Air Duct	

Table A-3: Upstream Air Duct observations

Obs No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
US-1	Joint I/II	Outer (East) wall	Efflorescence high on wall, built up over top of seismic retrofitting Steel strands supporting the utility lines below this location are corroded through.	  
US-2	Element II, ~5 m north of Door 13	Outer (East) wall	NDT on element II by WSP Rebar exposed at a past core repair location. Likely the location of 2000/2001 LPR measurement. No signs of corrosion on embedded bar.	

Obs No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
US-3	Joint II/III	Outer (East Wall)	Transverse crack in joint. Core JUE-P3-C2 collected.	 
US-4	Joint II/III	Inner (West) wall	Delamination (shown in red paint) already observed in 2018/2019 Inspection Reports. Cores JUW-P1 and JUW-C1 collected.	

Obs No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
US-5	Element III	Overview		  

Obs No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
US-6	Element III, Door 19	Overview	Standing water (max depth ~100 mm) in walkway area	
US-7	Joint III/IV	Outer (East) wall	Minor cracking with traces of efflorescence between the two steel plates, and a build-up at the centre just above the conduit Casting joint and minor honeycombing visible between precast and infill joint	 

US-8	Element IV, Door 22	Outer (East) wall	Spall and exposed rebar. Possible indication of AAR with rim around coarse aggregates and gel Core WUE-P2 collected here	 

Obs No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
				  

Obs No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
US-9	Element IV, Door 24	Outer (East) wall	Fine cracks	
US-10	Element IV, Door 24	Outer (East) wall	Weeping crack and efflorescence	 

Obs No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
US-11	Element V	Outer (East) wall	Fine cracks	
US-12	Element V	Outer (East) wall	Small spall and cracking	
US-13	Element V	Outer (East) wall	Map cracking	

Obs No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
US-14	Joint 0/I	Outer (East) Wall	Efflorescence, no delamination noted	

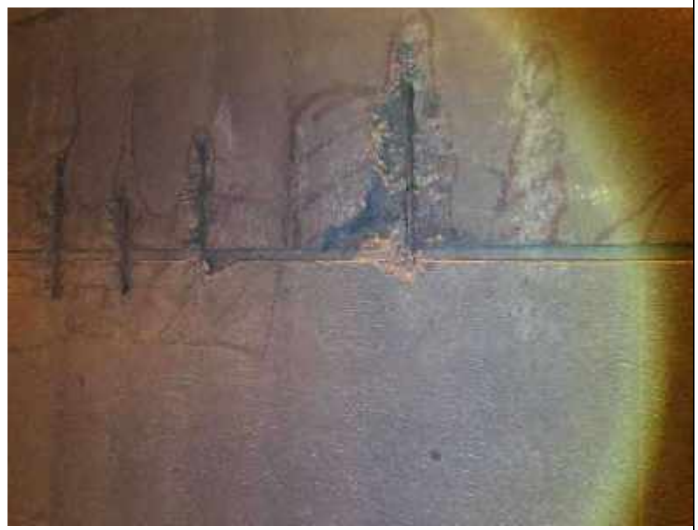
Table A-4: Southern (Deas) Approach Ramp observations

Obs. No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
SA-1	Element A1 and Portal Building	East wall	Damage to joint seal between approach ramp element (A1) and portal building above light fixture, joint pulled out locally and and broken. Leak at joint between portal building and approach ramp	   

Obs. No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
SA-2	Element A2, North of Door 37	East wall, roadway level	Efflorescence build up, sample collected (S4).	 
SA-3	Element A1 and A2, Near Door 37	East wall	Indication of AAR also approaching roadway level.	

Obs. No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
SA-4	Elements A1 and A2, North of Door 37	East wall, roadway level	Indication of AAR under T-beams Corrosion staining at joint	 

Obs. No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
SA-5	Element A10, Just south of door 38 under first T-beam	East wall, roadway level	Indication of AAR at the top of the wall, limited indications lower in wall. Core SAE-C8 collected.	

Obs. No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
SA-6	Element A8, Door 38	East wall, roadway level	Delamination and spalling	
SA-7	Elements A1 and A2	East and west walls, upper portion of wall	AAR gel and cracking at south approach retaining walls Examples of typical appearance of upper portion of retaining walls	 

SA-8	B and C type elements	Base slab joints	Ongoing repair of joints on the roadway. Asphalt was locally removed to expose concrete. Exposed rebar at base slab corner shown signs of local corrosion with limited cross section reductions. Areas were inaccessible for coring due to ongoing rehab works.	 

Obs. No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
				 

Table A-5: Northern (Lulu) Approach Ramp observations

Obs. No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
NA-1	A-type elements and pole foundation	East and west walls, top portsion	Sign of AAR and cracking on the retaining wall	  

NA-2	Element A1	West wall	Duplicate photo #32 of 2000 inspection	  

Obs. No.	Location (Area, Element/Joint ID, Wall ID)		Observation Notes	Photos
				
NA-3	Element A10	East wall	Proposed location for NAE-C7, Core taken from curb concrete adjacent to this location	 

Table A-6: Northbound Roadway Tube observations

Obs. No.	Location (Element/Joint ID, Wall ID)		Observation Notes	Photos
NB-1	Joint 0/1, Door 5	East wall	Core WNE-C6 location	  

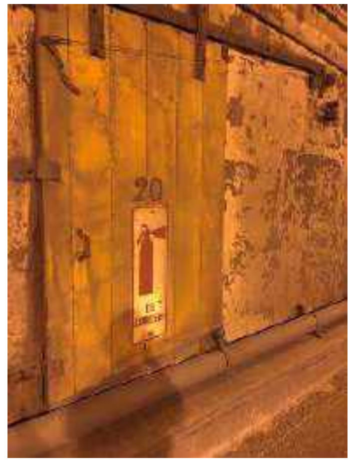
Obs. No.	Location (Element/Joint ID, Wall ID)		Observation Notes	Photos
NB-2	Element I, Door 7/8	Soffit	Spall on ceiling toward east side	 

NB-3	Element II, Door 12	East wall	Seemed to be affected by fire in the past Core WNE-P5 location.	

Obs. No.	Location (Element/Joint ID, Wall ID)		Observation Notes	Photos
				  

Obs. No.	Location (Element/Joint ID, Wall ID)		Observation Notes	Photos
NB-4	Element II, South of Door 13	West wall	Spall and exposed rebar Low cover, ~30 mm	
NB-5	Joint II/III	East wall	Sounded delaminated (Hammer) Core JNE-P6 location	 

Obs. No.	Location (Element/Joint ID, Wall ID)		Observation Notes	Photos
NB-6	Element III, North of Door 18	West wall	Delaminated area	
NB-7	Element IV, Door 20	East wall	Core WNE-C5 location	 

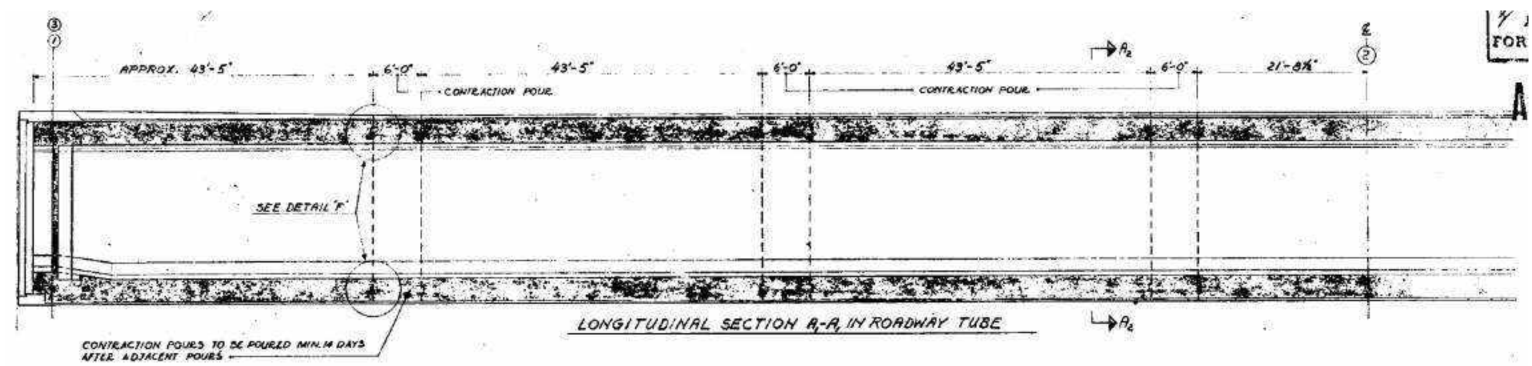
Obs. No.	Location (Element/Joint ID, Wall ID)		Observation Notes	Photos
NB-8	Element IV, Door 20	Central wall	Example of door numbering	
NB-9	Element V, ~10 m north of Door 27	Central Wall	Small areas of delaminated concrete	 

Obs. No.	Location (Element/Joint ID, Wall ID)		Observation Notes	Photos
NB-10	Element V, North of Door 28	West wall	Exposed vertical rebar	 
NB-11	Element V, North of Door 28	West wall	Core WNW-C4 location adjacent to spall/exposed vertical rebar.	

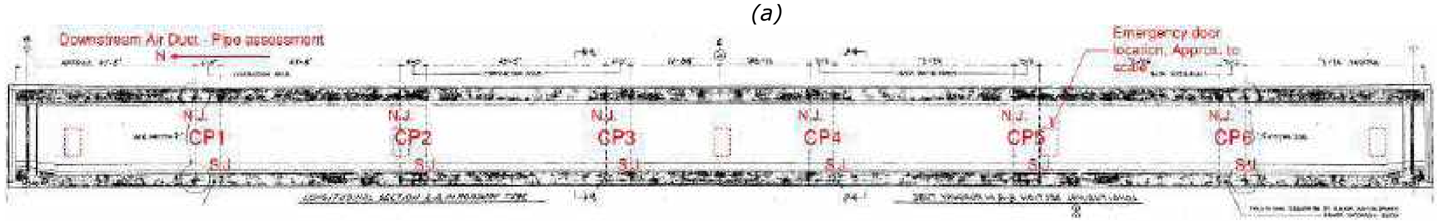
Obs. No.	Location (Element/Joint ID, Wall ID)		Observation Notes	Photos
NB-12	Joint V/VI area, Looking North	Overview, including joint in top slab	Overview showing general condition of roadway tunnel with peeling paint/coating on walls, joint with seismic retrofit plates	
NB-13	Element VI, Door 33/34	East wall	Core WNE-C3 location.	 

A.3 Observation of Drainpipes at Transverse Construction Joints

As discussed in Section 3.1.1 in the main body of the report, transverse construction joints include a continuous drain detail that is connected to drainpipes located in the outer walls of the air duct tubes. The drainpipe detail is shown in Figure 3-2(c) and the drainpipes are at each of the transverse construction joints shown in Figure A-2. Transverse construction joints exist between the 6' wide contraction pours located between the individual 43' 5" segments. Each element contains six contraction pours and 12 transverse construction joints. The entire tunnel included 72 transverse construction joints and 144 drainpipes. At the time of the inspection reported herein, at total of six drainpipes were not visible due to obstructions associated with sumps and a single pipe was missing. At the location of the missing pipe, only form cast concrete was observed with no signs of a pipe being present previously. Therefore, a total of 137 drainpipes were subject to visual inspection.



(a)



(b)

Figure A-2: Extracts from drawing 3-J-1028 including (a) Half-length longitudinal section of the immersed tunnel elements, showing the dimension of the segment and contraction pours, (b) a numbering system to identify individual transverse construction joints with approximate location of emergency doors shown.

Table A-7 documents the condition of the drainpipes as observed during the follow-up walkthrough of the air duct tubes completed on 23 September 2020. Contraction pours (CP) are numbered in the table as indicated in Figure A-2(b), with N.J. meaning north joint and S.J., south joint.

"Leak" in the table indicates that water/moisture was present in the drainpipe. Observations DS-2, DS-3, and DS-38 in Table A-2 provide examples of this condition. The "Leak" equated to slow drips in the worst case, and in most cases the pipe was damp with no dripping observed at the time of inspection. "Dry" indicates the pipe and chamfer below were dry. "Condition (Cond.) 1" indicates that the pipe was observed to be dry, however seepage through the construction joint under the pipe was seen. "Condition (Cond.) 2" again indicates the pipe was dry with seepage seen through the construction joint above the pipe. In one case, seeping through the construction joint above and below a dry pipe was observed.

These results are also discussed in Section 3.4 in the report's main body.

Table A-7: Overview of observed condition of drainpipes. Notes: ***CP*** *– Contraction Pour;* ***N.J.*** *– North Joint;* ***S.J.*** *– South Joint;* ***Cond. 1*** *– Dry pipe, joint weeping below drain;* ***Cond. 2*** *– Dry pipe, joint weeping above drain.*

	CP1		CP2		CP3		CP4		CP5		CP6	
	N.J.	**S.J.**	**N.J.**	**S.J.**	**N.J.**	**S.J.**	**N.J.**	**S.J.**	**N.J.**	**S.J.**	**N.J.**	**S.J.**
	Element I											
Downstream Air Duct	Leak	Leak	Dry	Leak	Leak	Cond. 1	**	Dry	Leak	Dry	Leak	Dry
Upstream Air Duct	Leak	Dry	Leak	Leak	Dry	Dry	Dry	Dry	Dry	Cond. 2	Dry	Leak
	Element II											
Downstream Air Duct	Dry	Leak	Leak	Leak	Dry	Dry	Dry	Dry	Dry	Leak	Dry	Leak
Upstream Air Duct	Dry	Leak	Leak	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry
	Element III											
Downstream Air Duct	Leak	Dry	Cond. 1	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Obstructed	
Upstream Air Duct	Cond. 1&2	Cond. 1	Leak	Dry	Dry	Dry	Leak	Leak	Dry	Dry	Obstructed	
	Element IV											
Downstream Air Duct	Leak	Dry	Leak	Dry	Dry	Dry	Leak	Dry	Dry	Leak	Leak	Leak
Upstream Air Duct	Obstructed		Leak	Leak	Dry	Dry	Leak	Dry	Leak	Leak	Leak	Leak
	Element V											
Downstream Air Duct	Dry	Leak	Dry	Dry	Dry†	Dry	Cond. 1	Dry	Leak	Leak	Leak	Leak
Upstream Air Duct	Leak	Leak	Dry	Dry	Cond. 1	Dry	Cond. 1	Leak	Dry	Dry	Dry	Dry
	Element VI											
Downstream Air Duct	Dry	Leak	Cond. 1	Cond. 2*	Dry	Dry	Leak‡	Leak	Cond. 1	Dry	Leak	Leak
Upstream Air Duct	Leak	Dry	Cond. 1	Dry	Dry	Cond. 1	Dry	Dry	Leak	Dry	Leak	Leak
* Obs No. DS-38 in Table A-2 ** Pipe Missing † Corrosion staining, dry ‡ Joint also seeping												

A.4 Core Data Sheets

The tables on the following pages provides compiled core data sheets from the individual cores extracted during the site inspection.

Date		27 July 2020		Inspector	BRPE
Core number		JUW-P1		Reinforcement	No
Diameter	4"	Location	Joint II/III, Inner wall	Length	9-10 cm

Photo documentation

Comments

General / Prior to coring

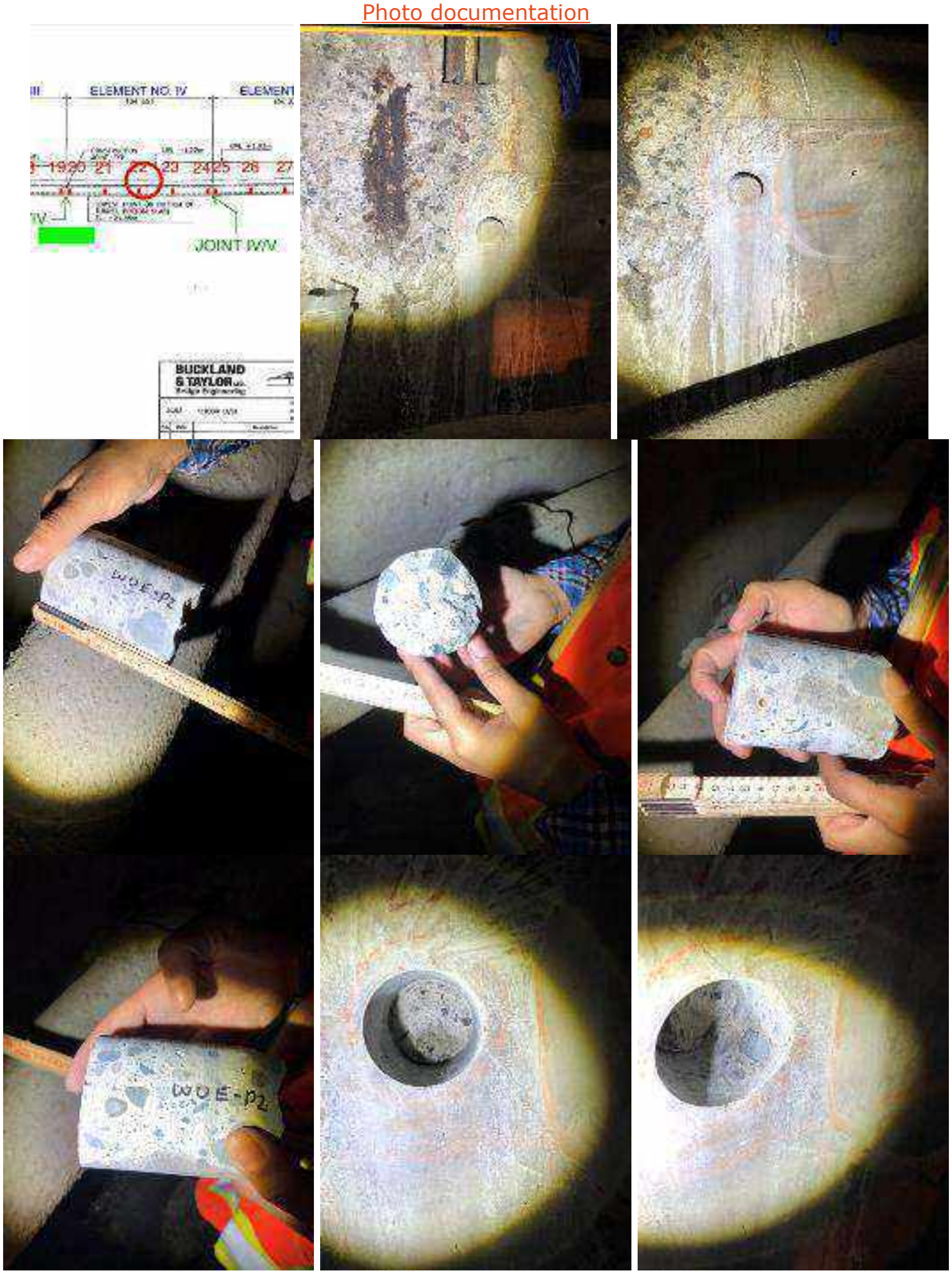
- Joint Concrete, Inner (West) Wall
- Cored adjacent to large/full height delamination in Joint II/III
- Core JUW-C1 is companion sample for chloride profile. Honeycombing observed near core location

Notes to core

- No visible cracks
- Porous concrete with 2 large voids

Date	28 July 2020			Inspector	BRPE
Core number	WUE-P2			Reinforcement	No
Diameter	3"	Location	Element IV, Door 22, Outer wall	Length	80 mm

Photo documentation



Comments

General / Prior to coring

- Core at delamination site for petrographic analysis

Notes to core

- No visible cracks observed in the core.
- Well consolidated concrete.
- Rim seen on 2 aggregate particles (photos taken)

Date		28 July 2020		Inspector	BRPE
Core number		JUE-P3-C2		Reinforcement	No
Diameter	3"	Location	Joint II/III, Outer wall	Length	~30 cm

Photo documentation








Comments

General / Prior to coring

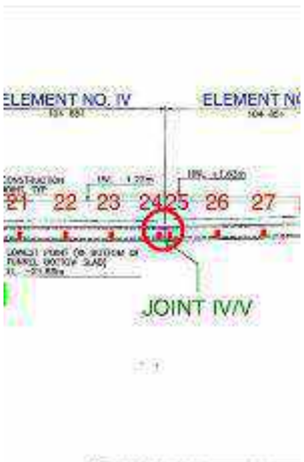
- Core at outer wall of upstream air duct, Joint II/III
- Core taken at a vertical crack in the joint

Notes to core

- 2 pieces
- Core appeared to fracture during removal, approximately ½ depth.
- Clear color change near crack in the core (likely carbonation)
- Crack appears to penetrate ~1/2 depth of core

Date	28 July 2020		Inspector	BRPE	
Core number	WUE-P4		Reinforcement	Yes	
Diameter	4" top 7 cm, 3" after	Location	Element IV, Door 24, Outer wall	Length	~30 cm (total)

Photo documentation



Comments

General / Prior to coring

- Deep core in outer (east) wall of upstream air duct.
- Check for AAR (petrographic analysis core)
- Fine, random cracks observed at concrete surface (hypothesized to potentially be caused by/early signs of AAR)

Notes to core

- 4" core started, but encountered rebar and therefore core continued with a 3" core. No signs of corrosion.
- Dense, well consolidated concrete
- No crack visible
- Some rims at aggregates, but no clear signs of AAR gel.

Date	29 July 2020	Inspector	BRPE
Core number	WNE-P5	Reinforcement	No
Diameter	4"	Location	Northbound Roadway Tube, East wall, Door 12
Length	~8-10 cm		

Photo documentation




Comments

General / Prior to coring

- Petrographic core at area of interest in northbound roadway tube.
- Core taken ~110 cm over road level (@ curb).
- A coating is on top surface of concrete. Concrete surface appears to be rough. Soffit at same location also has surface spalling. Possibly fire damage on wall and soffit.

Notes to core

- Flexible coating ~1 mm thick on surface
- Possible repair after fire damage
- No visible cracks in core.

Date	29 July 2020		Inspector	BRPE	
Core number	JNE-P6		Reinforcement	No	
Diameter	4"	Location	Northbound Roadway Tube, Joint II/III, East wall	Length	~6 cm plus additional ~3 cm of shards collected

Photo documentation

Comments

General / Prior to coring

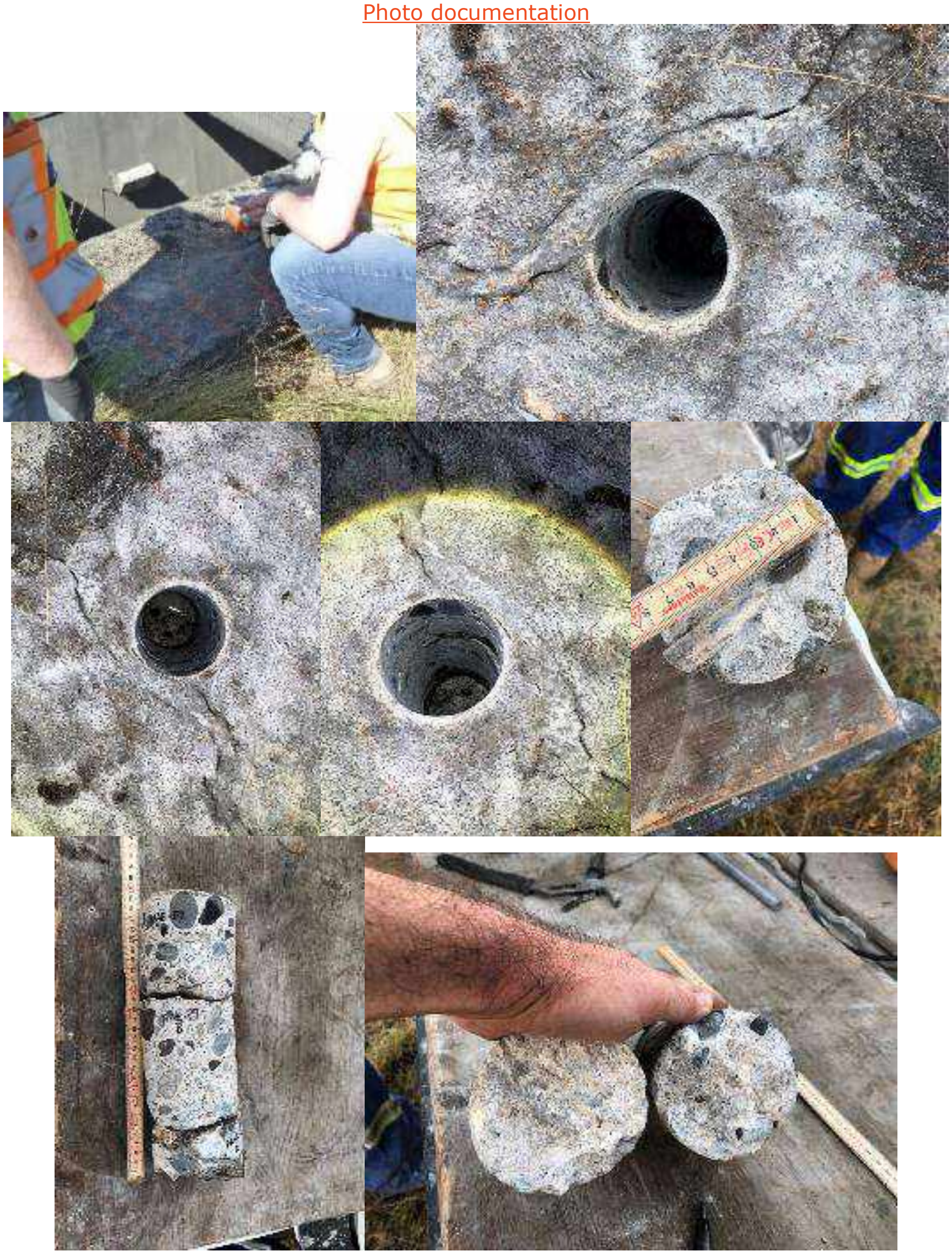
- Joint concrete in northbound roadway tube
- Audible delamination at location. Core through the delamination and 'sound' concrete behind.
- Core taken from wall at ~120-130 cm over roadway level @ curb

Notes to core

- ~30 mm depth of a 'pinkish' concrete
- Gaps are present between aggregate and paste at deepest parts of the 6 cm deep core
- No visible "vertical" cracks in the core, horizontal fracture/delamination at 6 cm depth
- ~10 mm gaps are present at core site at ~5-6 cm depth in core hole (photo taken)

Date	28 July 2020		Inspector	BRPE	
Core number	NAE-P7		Reinforcement	Yes	
Diameter	4"	Location	Northern (Lulu) Approach, Element A5E	Length	29 cm

Photo documentation



Comments

General / Prior to coring

- Core immediately next to past core (2000/2001 study).
- Core was centered on a vertical crack in the wall
- Core from top of retaining wall, northern approach east side wall A5W

Notes to core

- Core came out in 3 pieces – horizontal cracks in concrete (did not appear to be fractures from coring process)
- Signs of AAR. White deposits on horizontal cracks in the core.
- Rebar encountered, smooth bar, ~21 cm deep. Some corrosion stains.

Date	28 July 2020		Inspector	BRPE	
Core number	JUW-C1		Reinforcement	No	
Diameter	4"	Location	Upstream air duct, Inner (West) wall, Joint II/III	Length	~10 cm

Photo documentation




Comments

General / Prior to coring

- Upstream air duct, Inner (West) wall, Joint II/III
- Companion core to JUW-P1
- Near areas of delamination and honeycombing in Joint concrete

Notes to core

- No visible cracks
- Somewhat porous concrete, apparently more so than tunnel concrete
- Better consolidation in this core than JUW-P1
- Top 10-40 mm concrete has differing color (pinkish) paste.

Date	29 July 2020	Inspector	BRPE		
Core number	WNE-C3	Reinforcement	No	*	
Diameter	4"	Location	Northbound Roadway Tube, East wall, Door 34	Length	~13 cm

Photo documentation

Comments

General / Prior to coring

- Northbound roadway tunnel entry, approximately 10 north of door 34 - chloride profile
- Core from 85-90 cm over roadway level @ curb
- Companion NDT (corrosion and cover measurements) completed at this site

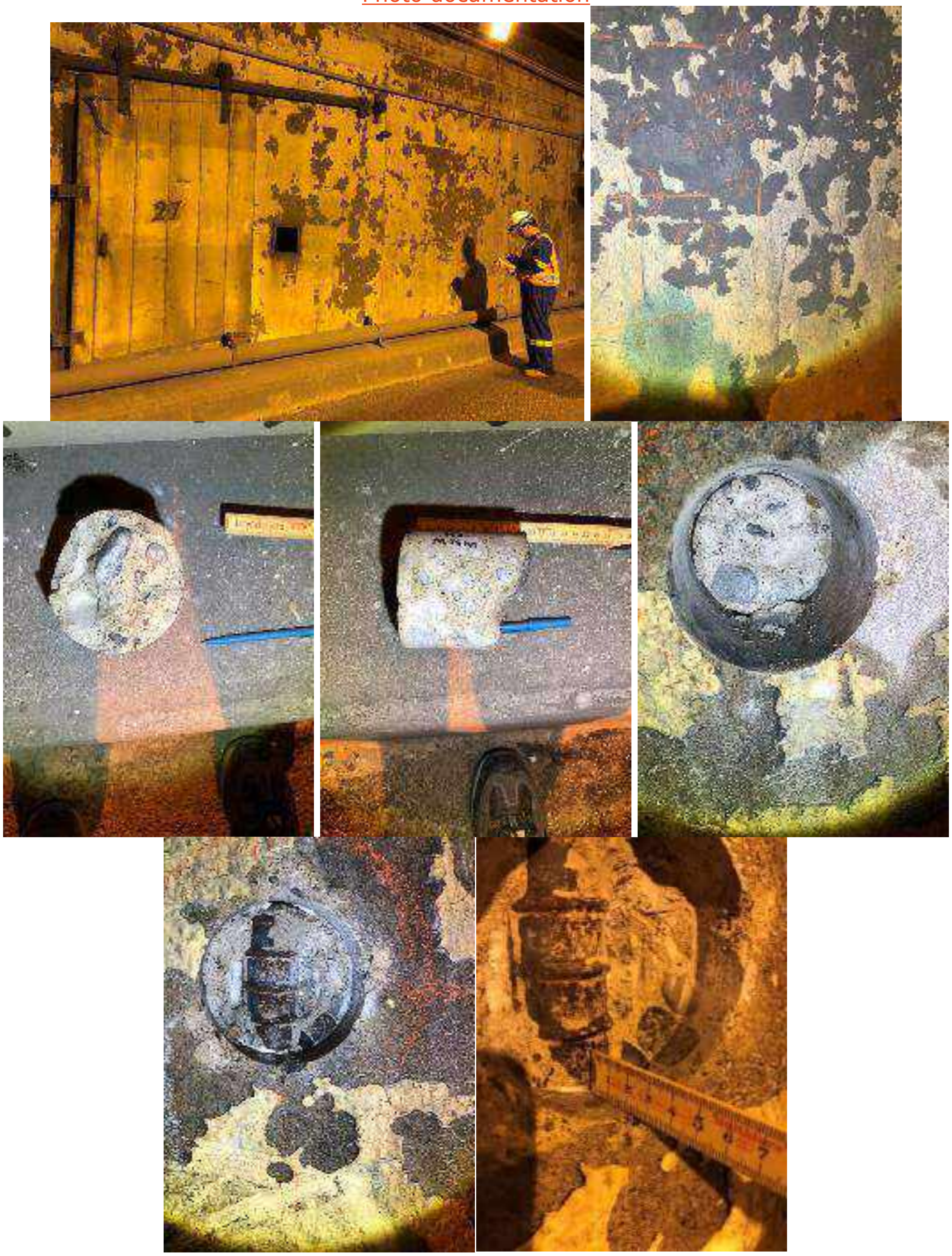
Notes to core

- No visible cracks
- Coating/paint on surface of core.

* Rebar was encountered at core used to expose rebar for NDT. Minimal signs of corrosion on exposed bar.

Date	29 July 2020	Inspector	BRPE		
Core number	WNW-C4	Reinforcement	No	*	
Diameter	4"	Location	Northbound Roadway Tube, West wall, Door 27	Length	~9 cm

Photo documentation



Comments

General / Prior to coring

- Core ¼ way into tunnel for chloride profile
- Core from ~130 cm over roadway level @ curb
- Companion NDT (corrosion and cover measurements) completed at this site

Notes to core

- No visible cracks in core.

* Rebar was encountered at core used to expose rebar for NDT. Low cover (~25 mm) and corrosion seen.

Date	29 July 2020			Inspector		BRPE	
Core number	WNE-C5 (A & B)			Reinforcement		No	
Diameter	4"	Location	Northbound Roadway Tube, East wall, Door 20	Length		7-10 cm	

Photo documentation

Comments

General / Prior to coring

- Core ½ way into tunnel for chloride profile
- Core from ~100 cm over roadway level @ curb
- Companion NDT (corrosion and cover measurements) completed at this site

Notes to core

- At least 3 materials encountered. Porous brown material at center of core under ~5-20 mm of what appears to be a SFRC. Possibly original concrete at sides of core.
- Core for accessing rebar was also kept (core B). Core B was approximately 4 cm deep. This core seems to be entirely the (possible) SFRC outer layer seen in the first core. Determined to complete chloride profile on Core B.

* Rebar was encountered at core used to expose rebar for NDT, limited signs of corrosion.

Date	29 July 2020		Inspector	BRPE	
Core number	WNE-C6		Reinforcement	No	*
Diameter	4"	Location	Northbound Roadway Tube, East wall, Door 5	Length	8-9 cm

Photo documentation

Comments

General / Prior to coring

- Point of interest chloride profile core near door 5, 'exit' of tunnel.
- Core from ~125 cm over roadway level @ curb
- Companion NDT (corrosion and cover measurements) completed at this site

Notes to core

- Coating/paint on surface of core
- No visible cracks
- Well consolidated concrete

* Rebar was encountered at core used to expose rebar for NDT. No signs of corrosion.

Date	28 July 2020		Inspector	BRPE	
Core number	NAE-C7		Reinforcement	Yes	Possibly cast iron embedment
Diameter	4"	Location	Northern (Lulu) Approach, Vertical core into central curb	Length	~9 cm

Photo documentation

Comments

General / Prior to coring

- Core taken from central curb.

Notes to core

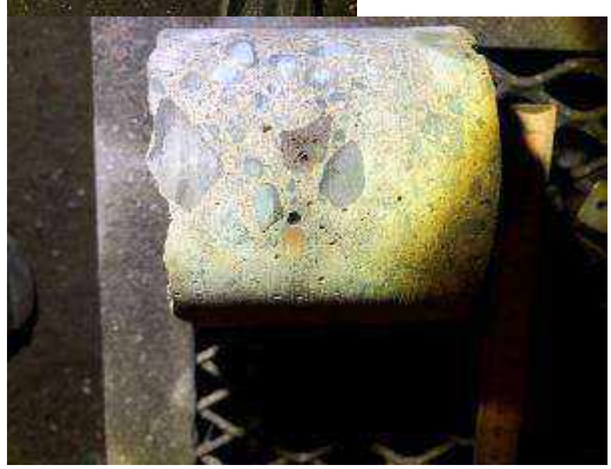
- Core intersected a steel object, likely not rebar and core stopped. Base slab was not reached.

Date	29 July 2020		Inspector	BRPE	
Core number	SAE-C8		Reinforcement	No	*
Diameter	4"	Location	Southern (Deas) Approach, Element A10, East wall	Length	~9 cm

Photo documentation





Comments

General / Prior to coring

- Southern approach wall, element A10, core from east wall. Core under second T-beam of the element
- Core from ~80 cm over roadway level @ curb

Notes to core

- No visible cracks
- Some rims and rust colored staining around aggregate and a porous aggregate seen (photo taken)

Appendix B Field Test Results, Chloride Profiles, & Carbonation Depth Measurements

Corrosion potential and Galvapulse current density measurements were completed at numerous locations on a 2m x 2m grid surrounding the exposed rebar. Cover measurements were completed at over a 1 m x 1 m grid at each location and were made using ground penetrating radar equipment that was calibrated to the physically measured cover depth (measured at site where rebar was exposed for corrosion measurements). Chloride profiles and carbonation depth were also measured from samples extracted throughout the structure. The original test reports are provided in the following pages, while Table B-1 and Table C-2 provides interpretation classifications of the half-cell potential and Galvapulse measurements, respectively, which are utilized in the main body of this report.

Table B-1: Interpretation approach for half-cell potential measurements per ASTM C876.

Potential (mV_{CSE})	Description from Appendix X1 of ASTM C876	Classification (at time of measurement)
> -200 mV_{CSE}	There is a greater than 90% probability that no reinforcing steel corrosion is occurring in that area at the time of measurement	Corrosion unlikely
-200 to -350 mV_{CSE}	Corrosion activity of the reinforcing steel in that area is uncertain	Potential for corrosion
< 350 mV_{CSE}	There is a greater than 90% probability that reinforcing steel corrosion is occurring in that area at the time of measurement	Moderate/severe corrosion potential

Table B-2 Interpretation approach for Galvapulse measured corrosion density per [9].

Galvapulse measured corrosion current density ($\mu A/cm^2$)	Interpretation of corrosion rate
< 0.5	Negligible
0.5 – 5	Slow
5 - 15	Moderate
> 15	High

Issued: 2020-08-21
Revised: 2020-09-04

COWI North America Ltd.
138 13th Street East, Suite 400
Vancouver, V7L 0E5

Attention: Mr. Brad Pease, Ph.D.

Project: George Massey Crossing
Subject: Summary Report for Core Sampling & Nondestructive Testing

INTRODUCTION

WSP attended the George Massey Tunnel site between July 27 and 30, 2020 to complete sampling of concrete cores and undertake nondestructive testing at various areas within the air ducts, the northbound roadway tube, and the north approach east wing wall. COWI personnel were also in attendance during WSP's time on site.

SCOPE

WSP's scope of services is generally summarized as follows:

- Extraction of several cores from concrete elements for:
  - Petrographic analysis (samples provided to others for the analysis);
  - Measurement of the water-soluble chloride concentration profiles; and
  - Measurement of the alkalinity of the concrete pore solution to determine depth of carbonation;
- Nondestructive testing at several locations to determine the following:
  - Corrosion current within the steel reinforcement;
    - Measurements were made using Galvapulse from Force Technology;
  - Half-cell potentials to determine the probability of corrosion per ASTM C876;
    - Potential measurements were made using a $Cu\text{-}CuSO_4$ electrode;
    - Measurements were made at numerous location over a 2m x 2m grid surrounding the rebar ground connection location;
    - Average, Minimum, and Maximum recorded corrosion potentials are presented;
  - Rebar cover measurements to determine the depth and spacing between steel reinforcing bars.
    - Cover measurements were made using Ground Penetrating RADAR;
    - Measurements were made at over a 1m x 1m grid surrounding the rebar ground connection location for half-cell potential measurements;
    - The depth of the exposed bar was measured and used to calibrate the GPR device at each location to ensure accuracy of the nondestructive test results at other locations;
- Analytical chemistry testing of samples of efflorescence / leachate material acquired by COWI representatives at three locations of leaking cracks.

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TEST RESULTS

A summary of the testing completed at each location along with the test results are presented in Tables 1 to 18. A photograph of each core is also included.

Table 1 – Test Results for West Side Air Duct, between Doors 20 and 21

Location ID	Photo
West Side Air Duct West Wall, between Doors 20 and 21	 Rebar for electrical connection
Testing	Rebar Spacing and Cover; Corrosion Current; Half-cell Potential.
Results	Rebar Spacing: Horizontal - Average 395 mm, Min 360 mm, Max 430 mm
	Rebar Spacing: Vertical - Average 130 mm, Min 100 mm, Max 180 mm
	Rebar Cover: Horizontal - Average 108 mm, Min 101 mm, Max 120 mm
	Rebar Cover: Vertical - Average 60 mm, Min 45 mm, Max 75 mm
	Corrosion Current: Average 2.6 $\mu A/cm^2$, Min < 0.5 $\mu A/cm^2$, Max 4.7 $\mu A/cm^2$
	Half-cell Potential: Average -25 mV_{CSE}, Min -33 mV_{CSE}, Max -13 mV_{CSE}

Table 2 – Test Results for West Side Air Duct West Wall, between Doors 17 and 18

Location ID	Photo
West Side Air Duct West Wall, between Doors 17 and 18	 Sample location near spall
Testing	Powder Samples for Chlorides/pH
Results	Chloride Ion Content (%wt. concrete)/pH: 0-20mm = 0.120/12.29 20-40mm = 0.102/11.64 40-60mm = 0.136/12.36 60-80mm = 0.076/12.37

Table 3 – Test Results for East Side Air Duct West Wall, between Doors 13 and 14

Location ID	Photo
East Side Air Duct East Wall, between Doors 13 and 14	No photo available
Testing	Rebar Spacing and Cover; Corrosion Current; and Half-cell Potential.
Results	Rebar Spacing: Horizontal - Average 435 mm, Min 380 mm, Max 490 mm
	Rebar Spacing: Vertical - Average 266 mm, Min 220 mm, Max 300 mm
	Rebar Cover: Horizontal - Average 70 mm, Min 55 mm, Max 79 mm
	Rebar Cover: Vertical - Average 38 mm, Min 36 mm, Max 42 mm
	Corrosion Current: Average 3.1 $\mu A/cm^2$, Min 2.4 $\mu A/cm^2$, Max 4.7 $\mu A/cm^2$
	Half-cell Potential: Average -83 mV_{CSE}, Min -126 mV_{CSE}, Max -55 mV_{CSE}

Table 4 – JUW-C1

Location ID	Photo
JUW-C1	
Testing	Core extracted. Detailed petrographic analysis to be completed by others
Results	N/A

Table 5 – WUE-P2

Location ID	Photo
WUE-P2	
Testing	Core extracted. Detailed petrographic analysis to be completed by others
Results	N/A

Table 6 – JUW-P1

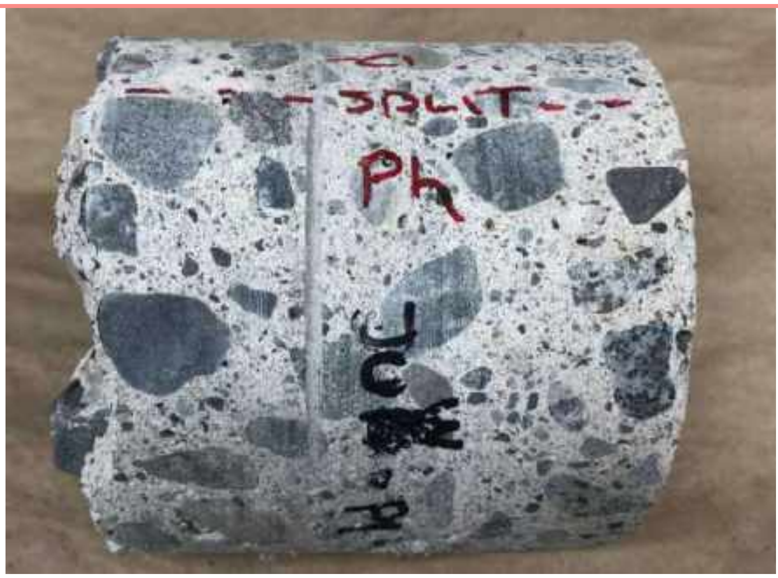
Location ID	Photo
JUW-P1	 Intact core  Core split along longitudinal axis; fractured face treated with phenolphthalein.
Testing	Chloride Profile and pH, Carbonation Depth Rebar Depth: 68 mm
Results	Chloride Ion Content (%wt. concrete)/pH: 0-15mm = < 0.010/10.78 15-30mm = < 0.010/9.88 30-45mm = < 0.010/12.15 45-60mm = 0.021/12.51 60-75mm = 0.030/12.58
	Carbonation Depth: 33 to 45 mm; Average = 38 mm

Table 7 – JUE-P3-C2

Location ID	Photo
JUE-P3-C2	 Intact core  Core split along longitudinal axis; fractured face treated with phenolphthalein.
Testing	Chloride Profile and pH for top 80mm of core, Carbonation Depth for top 80mm of core. Detailed petrographic analysis to be completed by others on remaining length of core.
Results	Chloride Ion Content (%wt. concrete)/pH: 0-15mm = 0.050/12.61 15-30mm = 0.068/12.65 30-45mm = 0.056/12.64 45-60mm = < 0.010/12.68 60-75mm = 0.023/12.61
	Carbonation Depth: 3 to 13 mm; Average = 5 mm

Table 8 – JNE-P6

Location ID	Photo
JNE-P6 - 5 m N of Door 15	
Testing	Core extracted. Detailed petrographic analysis to be completed by others.
Results	N/A

Table 9 – WUE-P4

Location ID	Photo
WUE-P4	
Testing	Core extracted. Detailed petrographic analysis to be completed by others.
Results	N/A

Table 10 – WNE-P5

Location ID	Photo
WNE-P5 - 6 m S of Door 12, 1.1 m above roadway	
Testing	Core extracted. Detailed petrographic analysis to be completed by others.
Results	N/A

Table 11 – NAE-P7

Location ID	Photo
NAE-P7	
Testing	Core extracted. Detailed petrographic analysis to be completed by others.
Results	N/A

Table 12 – WNE-C3

<table>
<tr><th>Location ID</th><th>Photo</th></tr>
<tr><td>WNE-C3 -
9 m N of Door 34,
0.9 m above
roadway</td><td>

Intact core


Core split along longitudinal axis; fractured face treated with phenolphthalein</td></tr>
<tr><td>Testing</td><td>Rebar Spacing and Cover;
Corrosion Current;
Half-cell Potential; and
Powder Samples for Chlorides/pH</td></tr>
<tr><td rowspan="8">Results</td><td>Rebar Spacing: Horizontal - Average 306 mm, Min 300 mm, Max 320 mm</td></tr>
<tr><td>Rebar Spacing: Vertical - Average 223 mm, Min 200 mm, Max 240 mm</td></tr>
<tr><td>Rebar Cover: Horizontal - Average 43 mm, Min 39 mm, Max 46 mm</td></tr>
<tr><td>Rebar Cover: Vertical - Average 61 mm, Min 59 mm, Max 63 mm</td></tr>
<tr><td>Corrosion Current: Average 3.5 $\mu A/cm^2$, Min 2.0 $\mu A/cm^2$, Max 4.9 $\mu A/cm^2$</td></tr>
<tr><td>Half-cell Potential: Average -134 mV_{CSE}, Min -245 mV_{CSE}, Max -53 mV_{CSE}</td></tr>
<tr><td>Chloride Ion Content (%wt. concrete)/pH:
0-15mm = 0.058/12.59
15-30mm = 0.027/12.61
30-45mm = < 0.010/12.61
45-60mm = < 0.010/12.65
60-75mm = < 0.010/12.68</td></tr>
<tr><td>Carbonation Depth: 0 mm</td></tr>
</table>

Table 13 – WNE-C4

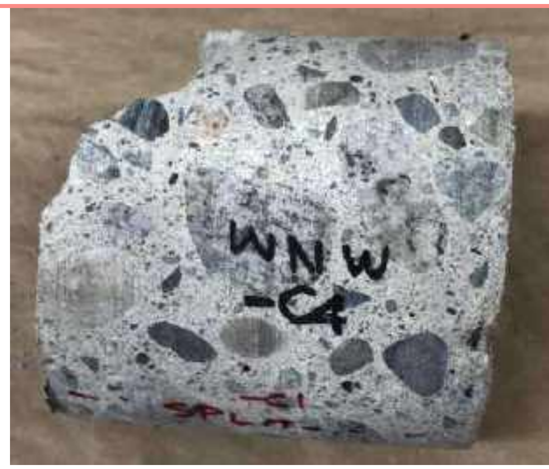
Location ID	Photo
WNE-C4 - 5 m N of Door 27, 1.3 m above roadway	 Intact core  Core split along longitudinal axis; fractured face treated with phenolphthalein.
Testing	Rebar Spacing and Cover; Corrosion Current; Half-cell Potential; and Powder Samples for Chlorides/pH
Results	Rebar Spacing: Horizontal - Average 440 mm, Min 420 mm, Max 460 mm
	Rebar Spacing: Vertical - Average 450 mm, Min 420 mm, Max 480 mm
	Rebar Cover: Horizontal - Average 52 mm, Min 50 mm, Max 53 mm
	Rebar Cover: Vertical - Average 32 mm, Min 31 mm, Max 33 mm
	Corrosion Current: Average 10.7 µA/cm², Min 10.7 µA/cm², Max 10.7 µA/cm²
	Half-cell Potential: Average -271 mV_{CSE}, Min -250 mV_{CSE}, Max -323 mV_{CSE}
	Chloride Ion Content (%wt. concrete)/pH: 0-15mm = 0.394/12.36 15-30mm = 0.214/12.55 30-45mm = 0.053/12.53 45-60mm = 0.014/12.56
	Carbonation Depth: 5 to 18 mm; Average = 8 mm

Table 14 – WNE-C5

Location ID	Photo
WNE-C5 - 4 m N of Door 20, 1.0 m above roadway	 Intact core
Testing	Rebar Spacing and Cover; Corrosion Current; and Half-cell Potential.
Results	Rebar Spacing: Horizontal - Average 325 mm, Min 310 mm, Max 340 mm
	Rebar Spacing: Vertical - Average 162 mm, Min 130 mm, Max 200 mm
	Rebar Cover: Horizontal - Average 46 mm, Min 45 mm, Max 49 mm
	Rebar Cover: Vertical - Average 54 mm, Min 50 mm, Max 57 mm
	Corrosion Current: Average < 0.5 µA/cm², Min <0.5 µA/cm², Max <0.5 µA/cm²
	Half-cell Potential: Average -320 mV_{CSE}, Min -369 mV_{CSE}, Max -250 mV_{CSE}

Table 15 – WNE-C6

Location ID	Photo
WNE-C6 - 0 m of Door 15, 1.2 m from roadway	 Intact core  Core split along longitudinal axis; fractured face treated with phenolphthalein
Testing	Rebar Spacing and Cover; Corrosion Current; Half-cell Potential; and Powder Samples for Chlorides/pH
Results	Rebar Spacing: Horizontal - Average 305 mm, Min 300 mm, Max 310 mm
	Rebar Spacing: Vertical - Average 225 mm, Min 220, Max 230 mm
	Rebar Cover: Horizontal - Average 44 mm, Min 43 mm, Max 45 mm
	Rebar Cover: Vertical - Average 49 mm, Min 46 mm, Max 54 mm
	Corrosion Current: Average 9.5 µA/cm², Min 5.6 µA/cm², Max 13.5 µA/cm²
	Half-cell Potential: Average -170 mV_{CSE}, Min -196 mV_{CSE}, Max -150 mV_{CSE}
	Chloride Ion Content (%wt. concrete)/pH: 0-15mm = 0.154/12.46 15-30mm = 0.019/12.56 30-45mm = <0.010/12.59 45-60mm = <0.010/12.59 60-75mm = <0.010/12.55
	Carbonation Depth: 6 to 16 mm; Average = 11 mm

Table 16 – SAE-C8

Location ID	Photo
SAE-C8 – 8 m N of 1st T-beam on S end, 0.8 m above roadway	 Intact core Core split along longitudinal axis; fractured face treated with phenolphthalein
Testing	Powder Samples for Chlorides/pH; and half core sent for detailed petrographic analysis by others.
Results	Chloride Ion Content (%wt. concrete)/pH: 0-15mm = 0.154/12.47 15-30mm = 0.172/12.51 30-45mm = 0.105/12.53 45-60mm = 0.065/12.62 60-75mm = 0.040/12.57

Table 17 – NAE-C7

Location ID	Photo
NAE-C7 – 13th T-beam at N end, from top of curb	 Intact core  Core split along longitudinal axis; fractured face treated with phenolphthalein.
Testing	Powder Samples for Chlorides/pH; and Carbonation Depth. Reinforcement at 56 mm
Results	Chloride Ion Content (%wt. concrete)/pH: 0-15mm = 0.078/12.29 15-30mm = 0.215/12.50 30-45mm = 0.133/12.52 45-60mm = 0.132/12.56 60-75mm = 0.084/12.58
	Carbonation Depth: 5 to 8 mm; Average = 6 mm

Three additional samples were obtained by COWI on site for chloride sampling. These samples were crushed for chloride ion content and pH. Results are summarized in Table 18.

Table 18 – NAE-C7

Location ID	Results
S2	Chloride Ion Content (%wt. concrete)/pH = 0.158/11.13
S3	Chloride Ion Content (%wt. concrete)/pH = 0.529/9.84
S4	Chloride Ion Content (%wt. concrete)/pH = 0.032/11.70

SUMMARY

Presented herein is a summary of the testing completed by WSP in relation to this project.

It is trusted that the enclosed data meets COWI's present requirements. Please contact the undersigned if the information presented in this report requires clarification or if you have any questions.

Thank you for retaining WSP.

WSP CANADA INC.

Tim Bandura, EIT
Project Engineer, Materials Discipline
Environment

Reviewed by:

Scott R. Cumming, M.E., P.Eng. (BC, ON)
National Practice Lead, Concrete & Construction Materials

TWB/src

Enclosure: Caro Analytical Services – Work Order #0081388 – Test Report of 2020-August-28

WSP ref.: 201-07676-00, Phase 04

CARO
ANALYTICAL SERVICES

CERTIFICATE OF ANALYSIS

REPORTED TO	WSP Canada Inc. - Langley Unit 100 - 20339 96 Avenue Langley, BC V1M 0E4		
ATTENTION	Lily Hu	**WORK ORDER**	0081388
PO NUMBER	201-0767-00/02	**RECEIVED / TEMP**	2020-08-10 15:58 / 27°C
PROJECT	201-07676-00/02	**REPORTED**	2020-08-28 16:45
PROJECT INFO		**COC NUMBER**	No #

Introduction:

CARO Analytical Services is a testing laboratory full of smart, engaged scientists driven to make the world a safer and healthier place. Through our clients' projects we become an essential element for a better world. We employ methods conducted in accordance with recognized professional standards using accepted testing methodologies and quality control efforts. CARO is accredited by the Canadian Association for Laboratories Accreditation (CALA) to ISO/IEC 17025:2017 for specific tests listed in the scope of accreditation approved by CALA.

Big Picture Sidekicks

You know that the sample you collected after snowshoeing to site, digging 5 meters, and racing to get it on a plane so you can submit it to the lab for time sensitive results needed to make important and expensive decisions (whew) is VERY important. We know that too.

We've Got Chemistry

It's simple. We figure the more you enjoy working with our fun and engaged team members; the more likely you are to give us continued opportunities to support you.

Ahead of the Curve

Through research, regulation knowledge, and instrumentation, we are your analytical centre for the technical knowledge you need, BEFORE you need it, so you can stay up to date and in the know.

If you have any questions or concerns, please contact me at rsundar@caro.ca

Authorized By:

Rochita Sundar
Junior Account Manager

CARO ANALYTICAL SERVICES

TEST RESULTS

REPORTED TO	WSP Canada Inc. - Langley	**WORK ORDER**	0081388
PROJECT	201-07676-00/02	**REPORTED**	2020-08-28 16:45

Analyte	Result	RL	Units	Analyzed	Qualifier
WNE-C3 0-15mm (0081388-01) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	**0.058**	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.59**	0.10	pH units	2020-08-28	
WNE-C3 15-30mm (0081388-02) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	**0.027**	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.61**	0.10	pH units	2020-08-28	
WNE-C3 30-45mm (0081388-03) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.61**	0.10	pH units	2020-08-28	
WNE-C3 45-60mm (0081388-04) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.65**	0.10	pH units	2020-08-28	
WNE-C3 60-75mm (0081388-05) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.68**	0.10	pH units	2020-08-28	
WNE-C6 0-15mm (0081388-06) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	**0.154**	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.46**	0.10	pH units	2020-08-28	
WNE-C6 15-30mm (0081388-07) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	**0.019**	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.56**	0.10	pH units	2020-08-28	
WNE-C6 30-45mm (0081388-08) | Matrix: Solid | Sampled: 2020-08-13					

Analyte	Result	RL	Units	Analyzed	Qualifier
WNE-C6 30-45mm (0081388-08) | Matrix: Solid | Sampled: 2020-08-13, Continued					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.59**	0.10	pH units	2020-08-28	
WNE-C6 45-60mm (0081388-09) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.59**	0.10	pH units	2020-08-28	
WNE-C6 60-75mm (0081388-10) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.55**	0.10	pH units	2020-08-28	
WNE-C4 0-15mm (0081388-11) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	**0.394**	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.36**	0.10	pH units	2020-08-28	
WNE-C4 15-30mm (0081388-12) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	**0.214**	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.55**	0.10	pH units	2020-08-28	
WNE-C4 30-45mm (0081388-13) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	**0.053**	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.53**	0.10	pH units	2020-08-28	
WNE-C4 45-60mm (0081388-14) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	**0.014**	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.56**	0.10	pH units	2020-08-28	
JUW-P1 0-15mm (0081388-15) | Matrix: Solid | Sampled: 2020-08-13					

Analyte	Result	RL	Units	Analyzed	Qualifier
JUW-P1 0-15mm (0081388-15) | Matrix: Solid | Sampled: 2020-08-13, Continued					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**10.78**	0.10	pH units	2020-08-28	
JUW-P1 15-30mm (0081388-16) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**9.88**	0.10	pH units	2020-08-28	
JUW-P1 30-45mm (0081388-17) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.15**	0.10	pH units	2020-08-28	
JUW-P1 45-60mm (0081388-18) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	**0.021**	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.51**	0.10	pH units	2020-08-28	
JUW-P1 60-75mm (0081388-19) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	**0.030**	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.58**	0.10	pH units	2020-08-28	
NAE-C7 0-15mm (0081388-20) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	**0.078**	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.29**	0.10	pH units	2020-08-28	
NAE-C7 15-30mm (0081388-21) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	**0.215**	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.50**	0.10	pH units	2020-08-28	
NAE-C7 30-45mm (0081388-22) | Matrix: Solid | Sampled: 2020-08-13					

Analyte	Result	RL	Units	Analyzed	Qualifier
NAE-C7 30-45mm (0081388-22) | Matrix: Solid | Sampled: 2020-08-13, Continued					
General Parameters					
Chloride, Water-Soluble	**0.133**	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.52**	0.10	pH units	2020-08-28	
NAE-C7 45-60mm (0081388-23) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	**0.132**	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.56**	0.10	pH units	2020-08-28	
NAE-C7 60-75mm (0081388-24) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	**0.084**	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.58**	0.10	pH units	2020-08-28	
JUE-P3-C2 0-15mm (0081388-25) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	**0.050**	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.61**	0.10	pH units	2020-08-28	
JUE-P3-C2 15-30mm (0081388-26) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	**0.068**	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.65**	0.10	pH units	2020-08-28	
JUE-P3-C2 30-45mm (0081388-27) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	**0.056**	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.64**	0.10	pH units	2020-08-28	
JUE-P3-C2 45-60mm (0081388-28) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.68**	0.10	pH units	2020-08-28	
JUE-P3-C2 60-75mm (0081388-29) | Matrix: Solid | Sampled: 2020-08-13					

TEST RESULTS

REPORTED TO	WSP Canada Inc. - Langley	WORK ORDER	0081388
PROJECT	201-07676-00/02	REPORTED	2020-08-28 16:45

Analyte	Result	RL	Units	Analyzed	Qualifier
JUE-P3-C2 60-75mm (0081388-29) | Matrix: Solid | Sampled: 2020-08-13, Continued					
General Parameters					
Chloride, Water-Soluble	**0.023**	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.61**	0.10	pH units	2020-08-28	
SAE-C8 0-15mm (0081388-30) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	**0.154**	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.47**	0.10	pH units	2020-08-28	
SAE-C8 15-30mm (0081388-31) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	**0.172**	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.51**	0.10	pH units	2020-08-28	
SAE-C8 30-45mm (0081388-32) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	**0.105**	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.53**	0.10	pH units	2020-08-28	
SAE-C8 45-60mm (0081388-33) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	**0.065**	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.62**	0.10	pH units	2020-08-28	
SAE-C8 60-75mm (0081388-34) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	**0.040**	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.57**	0.10	pH units	2020-08-28	
D17-D18 0-20mm (0081388-35) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	**0.120**	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.29**	0.10	pH units	2020-08-28	PH1
D17-D18 20-40mm (0081388-36) | Matrix: Solid | Sampled: 2020-08-13					

TEST RESULTS

REPORTED TO	WSP Canada Inc. - Langley	**WORK ORDER**	0081388
PROJECT	201-07676-00/02	**REPORTED**	2020-08-28 16:45

Analyte	Result	RL	Units	Analyzed	Qualifier
D17-D18 20-40mm (0081388-36) | Matrix: Solid | Sampled: 2020-08-13, Continued					
General Parameters					
Chloride, Water-Soluble	**0.102**	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**11.64**	0.10	pH units	2020-08-28	PH1
D17-D18 40-60mm (0081388-37) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	**0.136**	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.36**	0.10	pH units	2020-08-28	PH1
D17-D18 60-85mm (0081388-38) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	**0.076**	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**12.37**	0.10	pH units	2020-08-28	PH1
S2 (0081388-39) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	**0.158**	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**11.13**	0.10	pH units	2020-08-28	
S3 (0081388-40) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	**0.529**	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**9.84**	0.10	pH units	2020-08-28	
S4 (0081388-41) | Matrix: Solid | Sampled: 2020-08-13					
General Parameters					
Chloride, Water-Soluble	**0.032**	0.010	% dry	2020-08-23	
pH (1:2 H2O Solution)	**11.70**	0.10	pH units	2020-08-28	

Sample Qualifiers:

PH1 Due to limited sample volume or matrix, the ratio of water to soil was greater than 2:1

CARO ANALYTICAL SERVICES

APPENDIX 1: SUPPORTING INFORMATION

REPORTED TO	WSP Canada Inc. - Langley	**WORK ORDER**	0081388
PROJECT	201-07676-00/02	**REPORTED**	2020-08-28 16:45

Analysis Description	Method Ref.	Technique	Accredited	Location
Chloride, Water-Soluble in Solid	CSA A23.2-4B	Hot Water Extraction / Potentiometric Titration		Richmond
pH in Solid	Carter 16.2 / SM 4500-H+ B (2017)	1:2 Soil/Water Slurry / Electrometry	✓	Richmond

Glossary of Terms:

RL	Reporting Limit (default)
% dry	Percent (dry weight basis)
<	Less than the specified Reporting Limit (RL) - the actual RL may be higher than the default RL due to various factors
pH units	pH < 7 = acidic, ph > 7 = basic
CSA	Canadian Standards Association Chemical Test Methods
SM	Standard Methods for the Examination of Water and Wastewater, American Public Health Association

General Comments:

The results in this report apply to the samples analyzed in accordance with the Chain of Custody document. This analytical report must be reproduced in its entirety. CARO is not responsible for any loss or damage resulting directly or indirectly from error or omission in the conduct of testing. Liability is limited to the cost of analysis. Samples will be disposed of 30 days after the test report has been issued unless otherwise agreed to in writing.

Please note any regulatory guidelines applied to this report are added as a convenience to the client, at their request, to help provide some initial context to analytical results obtained. Although CARO makes every effort to ensure accuracy of the associated regulatory guideline(s) applied, the guidelines applied cannot be assumed to be correct due to a variety of factors and as such CARO Analytical Services assumes no liability or responsibility for the use of those guidelines to make any decisions. The original source of the regulation should be verified and a review of the guideline(s) should be validated as correct in order to make any decisions arising from the comparison of the analytical data obtained to the relevant regulatory guideline for one's particular circumstances. Further, CARO Analytical Services assumes no liability or responsibility for any loss attributed from the use of these guidelines in any way.

APPENDIX 2: QUALITY CONTROL RESULTS

REPORTED TO	WSP Canada Inc. - Langley	**WORK ORDER**	0081388
PROJECT	201-07676-00/02	**REPORTED**	2020-08-28 16:45

The following section displays the quality control (QC) data that is associated with your sample data. Groups of samples are prepared in "batches" and analyzed in conjunction with QC samples that ensure your data is of the highest quality. Common QC types include:

- **Method Blank (Blk)**: A blank sample that undergoes sample processing identical to that carried out for the test samples. Method blank results are used to assess contamination from the laboratory environment and reagents.
- **Duplicate (Dup)**: An additional or second portion of a randomly selected sample in the analytical run carried through the entire analytical process. Duplicates provide a measure of the analytical method's precision (reproducibility).
- **Blank Spike (BS)**: A sample of known concentration which undergoes processing identical to that carried out for test samples, also referred to as a laboratory control sample (LCS). Blank spikes provide a measure of the analytical method's accuracy.
- **Matrix Spike (MS)**: A second aliquot of sample is fortified with with a known concentration of target analytes and carried through the entire analytical process. Matrix spikes evaluate potential matrix effects that may affect the analyte recovery.
- **Reference Material (SRM)**: A homogenous material of similar matrix to the samples, certified for the parameter(s) listed. Reference Materials ensure that the analytical process is adequate to achieve acceptable recoveries of the parameter(s) tested.

Each QC type is analyzed at a 5-10% frequency, i.e. one blank/duplicate/spike for every 10-20 samples. For all types of QC, the specified recovery (% Rec) and relative percent difference (RPD) limits are derived from long-term method performance averages and/or prescribed by the reference method.

Analyte	Result	RL	Units	Spike Level	Source Result	% REC	REC Limit	% RPD	RPD Limit	Qualifier
General Parameters, Batch B0H1351										
Blank (B0H1351-BLK1)				Prepared: 2020-08-22, Analyzed: 2020-08-23						
Chloride, Water-Soluble	< 0.010	0.010	% dry							
Blank (B0H1351-BLK2)				Prepared: 2020-08-22, Analyzed: 2020-08-23						
Chloride, Water-Soluble	< 0.010	0.010	% dry							
Blank (B0H1351-BLK3)				Prepared: 2020-08-22, Analyzed: 2020-08-23						
Chloride, Water-Soluble	< 0.010	0.010	% dry							
Duplicate (B0H1351-DUP1)	**Source: 0081388-01**			Prepared: 2020-08-22, Analyzed: 2020-08-23						
Chloride, Water-Soluble	0.063	0.010	% dry		0.058			9	25	
Duplicate (B0H1351-DUP2)	**Source: 0081388-20**			Prepared: 2020-08-22, Analyzed: 2020-08-23						
Chloride, Water-Soluble	0.066	0.010	% dry		0.078			16	25	
Duplicate (B0H1351-DUP3)	**Source: 0081388-40**			Prepared: 2020-08-22, Analyzed: 2020-08-23						
Chloride, Water-Soluble	0.531	0.010	% dry		0.529			< 1	25	
General Parameters, Batch B0H2443										
Duplicate (B0H2443-DUP1)	**Source: 0081388-01**			Prepared: 2020-08-28, Analyzed: 2020-08-28						
pH (1:2 H2O Solution)	12.58	0.10	pH units		12.59			< 1	4	

Issued: 2020-10-13

COWI North America Ltd.
138 13th Street East, Suite 400
Vancouver, V7L 0E5

Attention: Mr. Brad Pease, Ph.D.

Project: George Massey Crossing
Subject: Summary Report for Additional Analytical Chemistry Testing

INTRODUCTION

WSP attended the George Massey Tunnel site on September 23, 2020 to complete sampling of additional concrete powder samples at various areas within the air ducts. COWI personnel were also in attendance during WSP's time on site.

SCOPE

WSP's scope of services is generally summarized as follows:

- Extraction of powder samples from concrete elements at locations designated by COWI representatives for:
  - Measurement of the water-soluble chloride concentration profiles in accordance with CSA A23.2-4B; and
  - Measurement of the alkalinity of the concrete pore solution to determine depth of carbonation in accordance with Carter 16.2/SM 4500-H+ B (2017).

TEST RESULTS

A summary of the testing completed at each location along with the analytical chemistry test results are presented in Table 1. Contraction pour numbers were labelled sequentially in numerical order from north to south for the Downstream Air Duct and south to north for the Upstream Air Duct on site. Photographs of typical sample locations are shown in Photos 1 and 2. Rebar cover depth adjacent to the sample locations was measured using Ground Penetrating RADAR (GPR). It is understood that sample locations were selected to represent benchmark conditions within the tunnel concrete.

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Table 1 – Analytical Chemistry Test Results

Location	Test ID	Rebar Depth (mm)	Test Depth (mm)	Chloride Ion Content (%wt. concrete)	pH
Downstream Air Duct Element I 2nd Contraction Pour Outer Wall	CL1	42	0 – 15	0.016	11.14
			15 – 30	0.022	-[1]
			30 – 45	0.017	12.14
			45 – 60	0.021	-[1]
			60 – 75	0.024	-[1]
Downstream Air Duct Element III Between 2nd and 3rd Contraction Pour on Outer Wall	CL2	49	0 – 15	0.021	12.14
			15 – 30	0.016	-[1]
			30 – 45	0.014	12.25
			45 – 60	<0.010	12.28
			60 – 75	<0.010	-[1]
Downstream Air Duct Element V Near 1st Contraction Pour on Outer Wall	CL3	38	0 – 15	0.013	12.27
			15 – 30	<0.010	11.85
			30 – 45	<0.010	12.31
			45 – 60	<0.010	12.25
			60 – 75	<0.010	11.63
Downstream Air Duct Element V/VI At Joint on Outer Wall	CL4	51	0 – 15	0.012	11.08
			15 – 30	0.020	11.74
			30 – 45	0.052	12.19
			45 – 60	0.037	12.24
			60 – 75	0.028	12.33
Downstream Air Duct Element VI/0 At Joint on Intermediate Wall	CL5	71	0 – 15	0.022	11.39
			15 – 30	0.010	11.73
			30 – 45	0.016	12.15
			45 – 60	0.019	12.28
			60 – 75	<0.010	12.24

Notes:

1 – Insufficient powder sample remained for pH testing after processing by the lab for chloride ion content.

Table 1 – Analytical Chemistry Test Results (Continued)

Location	Test ID	Rebar Depth (mm)	Test Depth (mm)	Chloride Ion Content (%wt. concrete)	pH
Upstream Air Duct Element VI Between 3rd and 4th Contraction Pour on Outer Wall	CL6	48	0 – 15	0.013	12.02
			15 – 30	<0.010	12.12
			30 – 45	<0.010	12.32
			45 – 60	<0.010	12.16
			60 – 75	<0.010	12.28
Upstream Air Duct Element VI/V At Joint on Outer Wall	CL7	45	0 – 15	0.012	11.88
			15 – 30	0.010	12.21
			30 – 45	<0.010	11.96
			45 – 60	<0.010	12.12
			60 – 75	<0.010	12.21
Upstream Air Duct Element VI/V At Joint on Intermediate Wall	CL8	78	0 – 15	0.029	11.29
			15 – 30	<0.010	11.89
			30 – 45	0.013	12.36
			45 – 60	<0.010	12.39
			60 – 75	<0.010	12.41
Upstream Air Duct Element IV Between 1st and 2nd Contraction Pour on Outer Wall	CL9	45	0 – 15	0.012	12.05
			15 – 30	0.012	11.92
			30 – 45	0.011	12.33
			45 – 60	<0.010	12.25
			60 – 75	<0.010	11.98
Downstream Air Duct Element II Near 1st Contraction Pour on Outer Wall	CL10	35	0 – 15	0.015	12.22
			15 – 30	0.015	12.39
			30 – 45	<0.010	11.80
			45 – 60	<0.010	12.33
			60 – 75	<0.010	12.16

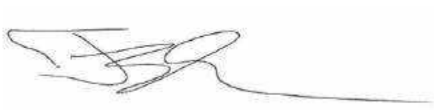
SUMMARY

Presented herein is a summary of the additional chloride and pH testing of the concrete pore solution completed by WSP at locations designated by COWI in relation to this project.

It is trusted that the enclosed data meets COWI's present requirements. Please contact the undersigned if the information presented in this report requires clarification or if you have any questions.

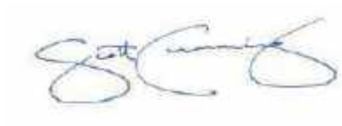
Thank you for retaining WSP.

WSP CANADA INC.



Tim Bandura, EIT
Project Engineer, Materials Discipline

Reviewed by:



Scott R. Cumming, M.E., P.Eng. (BC, ON)
National Practice Lead, Concrete & Construction Materials

TWB/src

Enclosure: Caro Analytical Services – Work Order #0092792 – Test Report of 2020-October-08

WSP ref.: 201-07676-00, Phase 04

Photo 1 – Typical view of sample location at Downstream Air Duct at Element V/VI at Joint on Outer Wall.

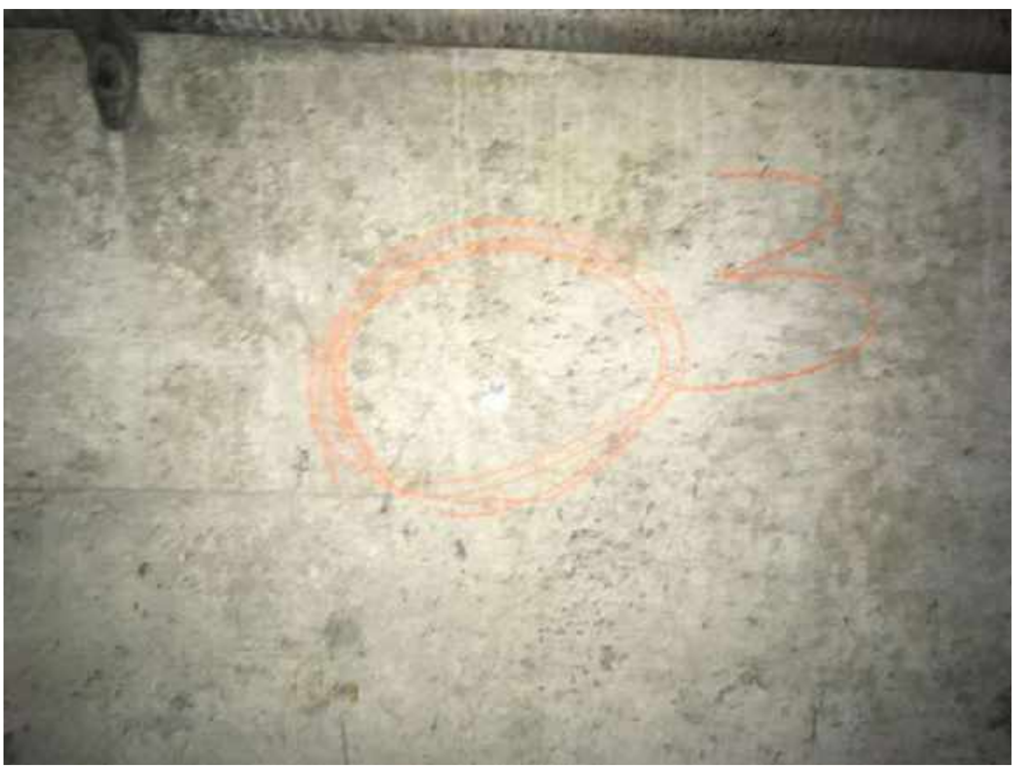


Photo 2 – Typical view of sample location at Downstream Air Duct Element V near 1st Contraction Pour on Outer Wall.

CERTIFICATE OF ANALYSIS

REPORTED TO	WSP Canada Inc. - Vancouver #1000-840 Howe Street Vancouver, BC V6Z 2M1		
ATTENTION	Sam Wallace	**WORK ORDER**	0092792
PO NUMBER		**RECEIVED / TEMP**	2020-09-24 14:55 / NA
PROJECT	201-07676-00	**REPORTED**	2020-10-08 17:46
PROJECT INFO			

Introduction:

CARO Analytical Services is a testing laboratory full of smart, engaged scientists driven to make the world a safer and healthier place. Through our clients' projects we become an essential element for a better world. We employ methods conducted in accordance with recognized professional standards using accepted testing methodologies and quality control efforts. CARO is accredited by the Canadian Association for Laboratories Accreditation (CALA) to ISO/IEC 17025:2017 for specific tests listed in the scope of accreditation approved by CALA.

Big Picture Sidekicks

You know that the sample you collected after snowshoeing to site, digging 5 meters, and racing to get it on a plane so you can submit it to the lab for time sensitive results needed to make important and expensive decisions (whew) is VERY important. We know that too.

We've Got Chemistry

It's simple. We figure the more you enjoy working with our fun and engaged team members; the more likely you are to give us continued opportunities to support you.

Ahead of the Curve

Through research, regulation knowledge, and instrumentation, we are your analytical centre for the technical knowledge you need, BEFORE you need it, so you can stay up to date and in the know.

If you have any questions or concerns, please contact me at rsundar@caro.ca

Authorized By:

Rochita Sundar
Junior Account Manager

CARO ANALYTICAL SERVICES

TEST RESULTS

REPORTED TO	WSP Canada Inc. - Vancouver	**WORK ORDER**	0092792
PROJECT	201-07676-00	**REPORTED**	2020-10-08 17:46

Analyte	Result	RL	Units	Analyzed	Qualifier
C1 0-15 (0092792-01) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	**0.016**	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**11.14**	0.10	pH units	2020-10-08	PH1
C1 15-30 (0092792-02) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	**0.022**	0.010	% dry	2020-10-05	
C1 30-45 (0092792-03) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	**0.017**	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**12.14**	0.10	pH units	2020-10-08	PH1
C1 45-60 (0092792-04) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	**0.021**	0.010	% dry	2020-10-05	
C1 60-75 (0092792-05) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	**0.024**	0.010	% dry	2020-10-05	
C2 0-15 (0092792-06) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	**0.021**	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**12.14**	0.10	pH units	2020-10-08	PH1
C2 15-30 (0092792-07) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	**0.016**	0.010	% dry	2020-10-05	
C2 30-45 (0092792-08) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	**0.014**	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**12.25**	0.10	pH units	2020-10-08	PH1

Analyte	Result	RL	Units	Analyzed	Qualifier
C2 45-60 (0092792-09) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**12.28**	0.10	pH units	2020-10-08	PH1
C2 60-75 (0092792-10) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-10-05	
C3 0-15 (0092792-11) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	**0.013**	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**12.27**	0.10	pH units	2020-10-08	PH1
C3 15-30 (0092792-12) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**11.85**	0.10	pH units	2020-10-08	PH1
C3 30-45 (0092792-13) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**12.31**	0.10	pH units	2020-10-08	PH1
C3 45-60 (0092792-14) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**12.25**	0.10	pH units	2020-10-08	PH1
C3 60-75 (0092792-15) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**11.63**	0.10	pH units	2020-10-08	PH1
C4 0-15 (0092792-16) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					

Analyte	Result	RL	Units	Analyzed	Qualifier
C4 0-15 (0092792-16) | Matrix: Solid | Sampled: 2020-09-23, Continued					
General Parameters, Continued					
Chloride, Water-Soluble	**0.012**	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**11.08**	0.10	pH units	2020-10-08	PH1
C4 15-30 (0092792-17) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	**0.020**	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**11.74**	0.10	pH units	2020-10-08	PH1
C4 30-45 (0092792-18) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	**0.052**	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**12.19**	0.10	pH units	2020-10-08	PH1
C4 45-60 (0092792-19) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	**0.037**	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**12.24**	0.10	pH units	2020-10-08	PH1
C4 60-75 (0092792-20) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	**0.028**	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**12.33**	0.10	pH units	2020-10-08	PH1
C5 0-15 (0092792-21) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	**0.022**	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**11.39**	0.10	pH units	2020-10-08	PH1
C5 15-30 (0092792-22) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	**0.010**	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**11.73**	0.10	pH units	2020-10-08	PH1
C5 30-45 (0092792-23) | Matrix: Solid | Sampled: 2020-09-23					

TEST RESULTS

REPORTED TO	WSP Canada Inc. - Vancouver	**WORK ORDER**	0092792
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Analyte	Result	RL	Units	Analyzed	Qualifier
C5 30-45 (0092792-23) | Matrix: Solid | Sampled: 2020-09-23, Continued					
General Parameters					
Chloride, Water-Soluble	**0.016**	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**12.15**	0.10	pH units	2020-10-08	PH1
C5 45-60 (0092792-24) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	**0.019**	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**12.28**	0.10	pH units	2020-10-08	PH1
C5 60-75 (0092792-25) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**12.24**	0.10	pH units	2020-10-08	PH1
C6 0-15 (0092792-26) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	**0.013**	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**12.02**	0.10	pH units	2020-10-08	PH1
C6 15-30 (0092792-27) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**12.12**	0.10	pH units	2020-10-08	PH1
C6 30-45 (0092792-28) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**12.32**	0.10	pH units	2020-10-08	PH1
C6 45-60 (0092792-29) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**12.16**	0.10	pH units	2020-10-08	PH1
C6 60-75 (0092792-30) | Matrix: Solid | Sampled: 2020-09-23					

Analyte	Result	RL	Units	Analyzed	Qualifier
C6 60-75 (0092792-30) | Matrix: Solid | Sampled: 2020-09-23, Continued					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**12.28**	0.10	pH units	2020-10-08	PH1
C7 0-15 (0092792-31) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	**0.012**	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**11.88**	0.10	pH units	2020-10-08	PH1
C7 15-30 (0092792-32) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	**0.010**	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**12.21**	0.10	pH units	2020-10-08	PH1
C7 30-45 (0092792-33) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**11.96**	0.10	pH units	2020-10-08	PH1
C7 45-60 (0092792-34) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**12.12**	0.10	pH units	2020-10-08	PH1
C7 60-75 (0092792-35) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**12.21**	0.10	pH units	2020-10-08	PH1
C8 0-15 (0092792-36) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	**0.029**	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**11.29**	0.10	pH units	2020-10-08	PH1
C8 15-30 (0092792-37) | Matrix: Solid | Sampled: 2020-09-23					

CARO ANALYTICAL SERVICES

TEST RESULTS

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Analyte	Result	RL	Units	Analyzed	Qualifier
C8 15-30 (0092792-37) | Matrix: Solid | Sampled: 2020-09-23, Continued					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**11.89**	0.10	pH units	2020-10-08	PH1
C8 30-45 (0092792-38) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	**0.013**	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**12.36**	0.10	pH units	2020-10-08	PH1
C8 45-60 (0092792-39) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**12.39**	0.10	pH units	2020-10-08	PH1
C8 60-75 (0092792-40) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**12.41**	0.10	pH units	2020-10-08	PH1
C9 0-15 (0092792-41) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	**0.012**	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**12.05**	0.10	pH units	2020-10-08	PH1
C9 15-30 (0092792-42) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	**0.012**	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**11.92**	0.10	pH units	2020-10-08	PH1
C9 30-45 (0092792-43) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	**0.011**	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**12.33**	0.10	pH units	2020-10-08	PH1
C9 45-60 (0092792-44) | Matrix: Solid | Sampled: 2020-09-23					

Analyte	Result	RL	Units	Analyzed	Qualifier
C9 45-60 (0092792-44) | Matrix: Solid | Sampled: 2020-09-23, Continued					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**12.25**	0.10	pH units	2020-10-08	PH1
C9 60-75 (0092792-45) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**11.98**	0.10	pH units	2020-10-08	PH1
C10 0-15 (0092792-46) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	**0.015**	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**12.22**	0.10	pH units	2020-10-08	PH1
C10 15-30 (0092792-47) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	**0.015**	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**12.39**	0.10	pH units	2020-10-08	PH1
C10 30-45 (0092792-48) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**11.80**	0.10	pH units	2020-10-08	PH1
C10 45-60 (0092792-49) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**12.33**	0.10	pH units	2020-10-08	PH1
C10 60-75 (0092792-50) | Matrix: Solid | Sampled: 2020-09-23					
General Parameters					
Chloride, Water-Soluble	< 0.010	0.010	% dry	2020-10-05	
pH (1:2 H2O Solution)	**12.16**	0.10	pH units	2020-10-08	PH1

Sample Qualifiers:

PH1 Due to limited sample volume or matrix, the ratio of water to soil was greater than 2:1

CARO ANALYTICAL SERVICES

APPENDIX 1: SUPPORTING INFORMATION

REPORTED TO	WSP Canada Inc. - Vancouver	**WORK ORDER**	0092792
PROJECT	201-07676-00	**REPORTED**	2020-10-08 17:46

Analysis Description	Method Ref.	Technique	Accredited	Location
Chloride, Water-Soluble in Solid	CSA A23.2-4B	Hot Water Extraction / Potentiometric Titration		Richmond
pH in Solid	Carter 16.2 / SM 4500-H+ B (2017)	1:2 Soil/Water Slurry / Electrometry	✓	Richmond

Glossary of Terms:

RL	Reporting Limit (default)
% dry	Percent (dry weight basis)
<	Less than the specified Reporting Limit (RL) - the actual RL may be higher than the default RL due to various factors
pH units	pH < 7 = acidic, ph > 7 = basic
CSA	Canadian Standards Association Chemical Test Methods
SM	Standard Methods for the Examination of Water and Wastewater, American Public Health Association

General Comments:

The results in this report apply to the samples analyzed in accordance with the Chain of Custody document. This analytical report must be reproduced in its entirety. CARO is not responsible for any loss or damage resulting directly or indirectly from error or omission in the conduct of testing. Liability is limited to the cost of analysis. Samples will be disposed of 30 days after the test report has been issued unless otherwise agreed to in writing.

Please note any regulatory guidelines applied to this report are added as a convenience to the client, at their request, to help provide some initial context to analytical results obtained. Although CARO makes every effort to ensure accuracy of the associated regulatory guideline(s) applied, the guidelines applied cannot be assumed to be correct due to a variety of factors and as such CARO Analytical Services assumes no liability or responsibility for the use of those guidelines to make any decisions. The original source of the regulation should be verified and a review of the guideline(s) should be validated as correct in order to make any decisions arising from the comparison of the analytical data obtained to the relevant regulatory guideline for one's particular circumstances. Further, CARO Analytical Services assumes no liability or responsibility for any loss attributed from the use of these guidelines in any way.

APPENDIX 2: QUALITY CONTROL RESULTS

REPORTED TO	WSP Canada Inc. - Vancouver	**WORK ORDER**	0092792
PROJECT	201-07676-00	**REPORTED**	2020-10-08 17:46

The following section displays the quality control (QC) data that is associated with your sample data. Groups of samples are prepared in “batches” and analyzed in conjunction with QC samples that ensure your data is of the highest quality. Common QC types include:

- **Method Blank (Blk)**: A blank sample that undergoes sample processing identical to that carried out for the test samples. Method blank results are used to assess contamination from the laboratory environment and reagents.
- **Duplicate (Dup)**: An additional or second portion of a randomly selected sample in the analytical run carried through the entire analytical process. Duplicates provide a measure of the analytical method's precision (reproducibility).
- **Blank Spike (BS)**: A sample of known concentration which undergoes processing identical to that carried out for test samples, also referred to as a laboratory control sample (LCS). Blank spikes provide a measure of the analytical method's accuracy.
- **Matrix Spike (MS)**: A second aliquot of sample is fortified with with a known concentration of target analytes and carried through the entire analytical process. Matrix spikes evaluate potential matrix effects that may affect the analyte recovery.
- **Reference Material (SRM)**: A homogenous material of similar matrix to the samples, certified for the parameter(s) listed. Reference Materials ensure that the analytical process is adequate to achieve acceptable recoveries of the parameter(s) tested.

Each QC type is analyzed at a 5-10% frequency, i.e. one blank/duplicate/spike for every 10-20 samples. For all types of QC, the specified recovery (% Rec) and relative percent difference (RPD) limits are derived from long-term method performance averages and/or prescribed by the reference method.

Analyte	Result	RL	Units	Spike Level	Source Result	% REC	REC Limit	% RPD	RPD Limit	Qualifier
General Parameters, Batch B0I2557										
Blank (B0I2557-BLK1)				Prepared: 2020-09-28, Analyzed: 2020-10-05						
Chloride, Water-Soluble	< 0.010	0.010	% dry							
Blank (B0I2557-BLK2)				Prepared: 2020-09-28, Analyzed: 2020-10-05						
Chloride, Water-Soluble	< 0.010	0.010	% dry							
Blank (B0I2557-BLK3)				Prepared: 2020-09-28, Analyzed: 2020-10-05						
Chloride, Water-Soluble	< 0.010	0.010	% dry							
Duplicate (B0I2557-DUP1)		**Source: 0092792-02**		Prepared: 2020-09-28, Analyzed: 2020-10-05						
Chloride, Water-Soluble	0.028	0.010	% dry		0.022				25	
Duplicate (B0I2557-DUP2)		**Source: 0092792-25**		Prepared: 2020-09-28, Analyzed: 2020-10-05						
Chloride, Water-Soluble	< 0.010	0.010	% dry		0.010				25	
Duplicate (B0I2557-DUP3)		**Source: 0092792-50**		Prepared: 2020-09-28, Analyzed: 2020-10-05						
Chloride, Water-Soluble	< 0.010	0.010	% dry		< 0.010				25	

Appendix C Damage Ratings and Petrographic Reports

C.1 Damage Rating Index – Description and Reports

The damage rating index (DRI) approach is intended as a semi-quantitative point-count method to quantify ASR-related features in a prepared concrete specimen when viewed under ~15-16x magnification with a stereomicroscope as describe in the literature (see e.g., [12], [14], [16]-[18]). Specimens here were cut and polished cores obtained from the structure. The following features, typically considered as evidence of ASR, are counted when observed:

- Crack in aggregate;
- Crack in aggregate with ASR gel;
- Debonded aggregate;
- Reaction rim;
- ASR gel in air void;
- Crack in matrix;
- Crack in matrix with ASR gel; and
- Corroded aggregate

Counts of individual features are weighted by a multiplier, intended to weight the significance of the feature. Features clearly related to ASR like "Crack in matrix with ASR gel" are weighted more heavily than features that are either potentially not deleterious (e.g., reaction rim) or not definitively associated with ASR (e.g., crack in aggregate without ASR gel). The resulting DRI is effectively a numerical representation of the ASR-related damage observed in the concrete sample.

According to [14], the damage rating results can be "fairly subjective" and may vary based on the experience and judgment of the petrographer. A standardized test procedure is not yet established for DRI and fundamental details like the features to be counted and multipliers assigned to these features vary in the literature. Nevertheless, the DRI method provides useful information on the relative severity of ASR on a set of cores extracted from the same structure, when carried out by the same petrographer [14]. The petrography and DRI reports for this investigation were completed by a single petrographer (the same petrographer that completed 2000 analyses reported in [1]). DRIs in this appendix are intended to provide a starting point for possible future use to monitor progression of ASR in the structure.

Table C-8 provides an overview of computed DRIs, which can be used as a baseline for comparison with future assessments and to assess the relative severity of ASR in the structure. DRIs using various multiplier sets are presented and data from core A5 (tested in 2000 [1]), are included. Corroded aggregate was not a feature included in the 2000 DRI reports, therefore DRIs presented below both include (A) and exclude (B) the contribution of corroded aggregate.

Table C-8: Overview of damage rating indices from cores subjected to petrographic assessments. Damage rating indices are computed using various multiplier sets from references indicated. Minimum and maximum DRI values for each concrete type/year observed are indicated in bold.

Core ID	Concrete type	Year observed	Damage Rating Index				
			Multipliers from [1] & [14]	Multipliers from [12]		Multipliers from [17] & [18]	
			A	A	B	A	B
WUE-P2	Precast tunnel concrete	2020	54	62	81	**18**	31
WUE-P4			143	135	**148**	104	113
WNE-P5			127	112	121	104	109
JUE-P3-C2	Infill joint concrete – Outer wall		488	493	**693**	**332**	466
JUW-C1	Infill joint concrete – Intermediate wall		266	256	**287**	187	208
JNE-P6			182	157	232	**114**	164
NAE-P7	Approach Ramp Concrete		217	295	**300**	**133**	137
SAE-C8			201	189	232	171	199
A5 – 100 mm depth**	Approach Ramp Concrete (2000)	2000	288	**269**	-	238	-
A5 – 150 mm depth**			211	200	-	**167**	-

* A – Excluding contribution of corroded aggregate, B – Including contribution of corroded aggregate.
** Results from sample A5, extracted in 2000 and reported in [1]. Core NAE-P7 extracted ~1' away from core A5.

Results in Table C-8 are separated by concrete type and year of testing and, as all results are from the same petrographer, comparison of 2000 and 2020 results are considered to provide an indication of the propagation of ASR-induced damages from the represented location.

As discussed above, computed DRIs provide an indication of the relative severity of ASR in the cores. Due to the semi-quantitative nature of DRIs, in the analysis below differences in DRIs of less than 50-100 are considered to have limited significance whereas larger differences are considered more significant. Review of the above DRIs therefore indicates:

- The precast tunnel concrete has the amongst the lowest DRIs observed, with values ranging from 18 to 148, depending on calculation approach.
- The infill joint concrete at the outer wall has reported DRIs from 1.7 to 3.4 times higher than current DRIs from the approach ramp concrete with notable macroscopic visual signs of ASR-induced damage (although freeze/thaw may be a contributing factor to damage observed in the approach ramp, as discussed in Section 4.3.3 and below).
- The infill joint concrete at the intermediate walls and the approach ramp concrete have similar DRI ranges of 114 to 287 and 133 to 300, respectively.
- The DRI reported from the top of element A5 in the Northern (Lulu) Approach ramp is essentially unchanged between 2000 and 2020.
- Comparison of computed DRIs from the top of the approach ramps versus at the roadway level indicates that, at the microscopic level, as similar severity of ASR-induced damage is observed. Core SAE-C8, from near the roadway level, has DRIs ranging from 171-232, while Core NAE-P7 from the top of the retaining wall has DRIs ranging from 133-300. The outward appearance of the concrete at the two core locations differs significantly and as, discussed in Section 4.3.3, it is likely that ASR and freeze/thaw attack are occurring in the top ~2-3 m of the retaining walls constructed from non-air-entrained concrete.

The original Damage Rating Index reports are provided on the following pages.

DAMAGE RATING INDEX FOR DETERMINATION OF ASR IN HARDENED CONCRETE

COWI
138 13th Street East
North Vancouver, BC
V7L 0E5

Project number: 20149024
September 17, 2020

Attention: Brad Pease, Ph.D.

PROJECT:	**Massey Tunnel**		
Sample:	C1	**Location:**	Joint concrete

FEATURE	COUNT	MULTIPLIER	CONTRIBUTION TO DRI
Crack in aggregate	10	0.25	2.5
Crack in aggregate with ASR gel	13	2	26
Debonded aggregate	15	3	45
Reaction rim	118	0.5	59
ASR gel in air void	5	2	10
Crack in matrix	11	0.5	5.5
Crack in matrix with ASR gel	18	4	72
Corroded aggregate *(not used in 2000)*	9	3	(27)
DRI, Gross			220.0
Area viewed (cm^2)			86
DRI, Corrected			**256**
ASR SEVERITY RATING			**Moderate**

Petrographer: ____________________
F. Shrimer, P. Geo.

PROFESSIONAL GEOSCIENTIST — PROVINCE OF BRITISH COLUMBIA — F. H. SHRIMER

DATE: September 17, 2020

GOLDER

DAMAGE RATING INDEX FOR DETERMINATION OF ASR IN HARDENED CONCRETE

COWI
138 13th Street East
North Vancouver, BC
V7L 0E5

Project number: 20149024
September 8, 2020

Attention: Brad Pease, Ph.D.

PROJECT:	Massey Tunnel		
Sample:	P2	Location:	

FEATURE	COUNT	MULTIPLIER	CONTRIBUTION TO DRI
Crack in aggregate	21	0.25	5.25
Crack in aggregate with ASR gel	9	2	18
Debonded aggregate	0	3	0
Reaction rim	83	0.5	41.5
ASR gel in air void	7	2	14
Crack in matrix	0	0.5	0
Crack in matrix with ASR gel	0	4	0
Corroded aggregate	8	3	(24)
DRI, Gross			78.75
Area viewed (cm^2)			127
DRI, Corrected			**62**
ASR SEVERITY RATING			**Minor**

Petrographer: ______________________
F. Shrimer, P. Geo.

PROFESSIONAL GEOSCIENTIST, PROVINCE OF BRITISH COLUMBIA, F. H. SHRIMER

DATE: September 8, 2020

GOLDER

DAMAGE RATING INDEX FOR DETERMINATION OF ASR IN HARDENED CONCRETE

COWI
138 13th Street East
North Vancouver, BC
V7L 0E5

Project number: 20149024
September 8, 2020

Attention: Brad Pease, Ph.D.

PROJECT:	**Massey Tunnel**		
Sample:	P3	**Location:**	

FEATURE	COUNT	MULTIPLIER	CONTRIBUTION TO DRI
Crack in aggregate	38	0.25	9.5
Crack in aggregate with ASR gel	50	2	100
Debonded aggregate	22	3	66
Reaction rim	226	0.5	113
ASR gel in air void	9	2	18
Crack in matrix	6	0.5	3
Crack in matrix with ASR gel	50	4	200
Corroded aggregate *(not used in 2000)*	69	3	(207)
DRI, Gross			509.5
Area viewed (cm^2)			103.4
DRI, Corrected			**493**
ASR SEVERITY RATING			**Significant**

Petrographer: ____________________
F. Shrimer, P. Geo.

PROFESSIONAL GEOSCIENTIST PROVINCE OF BRITISH COLUMBIA F. H. SHRIMER

DATE: September 8, 2020

GOLDER

DAMAGE RATING INDEX FOR DETERMINATION OF ASR IN HARDENED CONCRETE

COWI
138 13th Street East
North Vancouver, BC
V7L 0E5

Project number: 20149024
August 30, 2020

Attention: Brad Pease, Ph.D.

PROJECT:	**Massey Tunnel**		
Sample:	P4	**Location:**	

FEATURE	COUNT	MULTIPLIER	CONTRIBUTION TO DRI
Crack in aggregate	25	0.25	6.25
Crack in aggregate with ASR gel	15	2	30
Debonded aggregate	7	3	21
Reaction rim	70	0.5	35
ASR gel in air void	1	2	2
Crack in matrix	6	0.5	3
Crack in matrix with ASR gel	6	4	24
Corroded aggregate *(not used in 2000)*	4	3	(12)
DRI, Gross			121.25
Area viewed (cm^2)			90
DRI, Corrected			**135**
ASR SEVERITY RATING			**Minor-Moderate**

Petrographer: F. Shrimer, P. Geo.

PROFESSIONAL PROVINCE OF F. H. SHRIMER BRITISH COLUMBIA GEOSCIENTIST

DATE: August 30, 2020

DAMAGE RATING INDEX FOR DETERMINATION OF ASR IN HARDENED CONCRETE

COWI
138 13th Street East
North Vancouver, BC
V7L 0E5

Project number: 20149024
September 16, 2020

Attention: Brad Pease, Ph.D.

PROJECT:	Massey Tunnel		
Sample:	P5	Location:	

FEATURE	COUNT	MULTIPLIER	CONTRIBUTION TO DRI
Crack in aggregate	13	0.25	3.25
Crack in aggregate with ASR gel	12	2	24
Debonded aggregate	8	3	24
Reaction rim	46	0.5	23
ASR gel in air void	0	2	0
Crack in matrix	7	0.5	3.5
Crack in matrix with ASR gel	1	4	4
Corroded aggregate *(not used in 2000)*	2	3	(6)
DRI, Gross			81.75
Area viewed (cm^2)			72.7
DRI, Corrected			**112**
ASR SEVERITY RATING			**Minor**

Petrographer: F. Shrimer, P. Geo.

PROFESSIONAL PROVINCE OF F. H. SHRIMER BRITISH COLUMBIA GEOSCIENTIST

DATE: September 16, 2020

DAMAGE RATING INDEX FOR DETERMINATION OF ASR IN HARDENED CONCRETE

COWI
138 13th Street East
North Vancouver, BC
V7L 0E5

Project number: 20149024
September 10, 2020

Attention: Brad Pease, Ph.D.

PROJECT:	**Massey Tunnel**		
Sample:	P6	**Location:**	

FEATURE	COUNT	MULTIPLIER	CONTRIBUTION TO DRI
Crack in aggregate	6	0.25	1.5
Crack in aggregate with ASR gel	5	2	10
Debonded aggregate	8	3	24
Reaction rim	102	0.5	51
ASR gel in air void	1	2	2
Crack in matrix	11	0.5	505
Crack in matrix with ASR gel	0	4	0
Corroded aggregate *(not used in 2000)*	15	3	(45)
DRI, Gross			94
Area viewed (cm^2)			60
DRI, Corrected			**157**
ASR SEVERITY RATING			**Moderate**

Petrographer: ______________________
F. Shrimer, P. Geo.

PROFESSIONAL GEOSCIENTIST — PROVINCE OF BRITISH COLUMBIA — F. H. SHRIMER

DATE: September 10, 2020

DAMAGE RATING INDEX FOR DETERMINATION OF ASR IN HARDENED CONCRETE

GOLDER

COWI
138 13th Street East
North Vancouver, BC
V7L 0E5

Project number: 20149024
September 8, 2020

Attention: Brad Pease, Ph.D.

PROJECT:	Massey Tunnel		
Sample:	P7	Location:	

FEATURE	COUNT	MULTIPLIER	CONTRIBUTION TO DRI
Crack in aggregate	11	0.25	2.75
Crack in aggregate with ASR gel	6	2	12
Debonded aggregate	5	3	15
Reaction rim	100	0.5	50
ASR gel in air void	73	2	146
Crack in matrix	17	0.5	8.5
Crack in matrix with ASR gel	21	4	84
Corroded aggregate	2	3	(6)
DRI, Gross			318.25
Area viewed (cm^2)			108
DRI, Corrected			**295**
ASR SEVERITY RATING			**Moderate**

Petrographer: ____________________
F. Shrimer, P. Geo.

PROFESSIONAL GEOSCIENTIST — PROVINCE OF BRITISH COLUMBIA — F. H. SHRIMER

DATE: September 8, 2020

DAMAGE RATING INDEX FOR DETERMINATION OF ASR IN HARDENED CONCRETE

COWI
138 13th Street East
North Vancouver, BC
V7L 0E5

Project number: 20149024
September 8, 2020

Attention: Brad Pease, Ph.D.

PROJECT:	Massey Tunnel		
Sample:	C8	Location:	

FEATURE	COUNT	MULTIPLIER	CONTRIBUTION TO DRI
Crack in aggregate	4	0.25	1
Crack in aggregate with ASR gel	14	2	28
Debonded aggregate	15	3	45
Reaction rim	42	0.5	21
ASR gel in air void	3	2	6
Crack in matrix	9	0.5	4.5
Crack in matrix with ASR gel	10	4	40
Corroded aggregate *(not used in 2000)*	11	3	(33)
DRI, Gross			145.5
Area viewed (cm^2)			76.8
DRI, Corrected			**189**
ASR SEVERITY RATING			**Moderate**

Petrographer: ______________________
F. Shrimer, P. Geo.

DATE: September 8, 2020

C.2 Petrographic Reports

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE

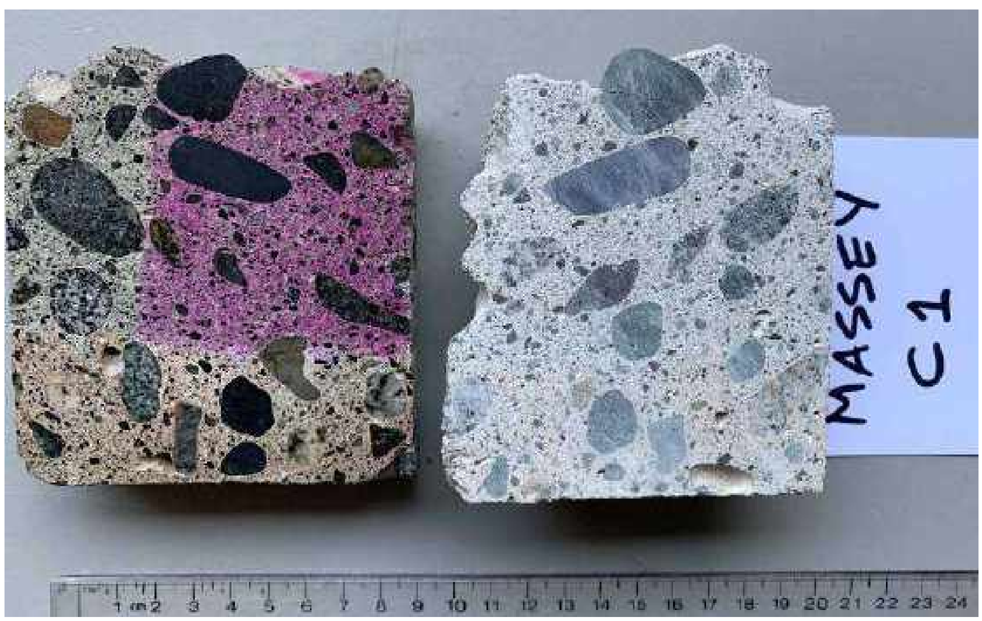
ASTM C856-18

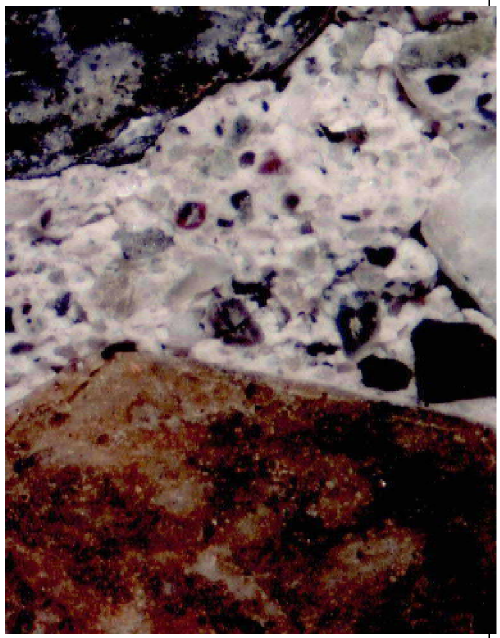
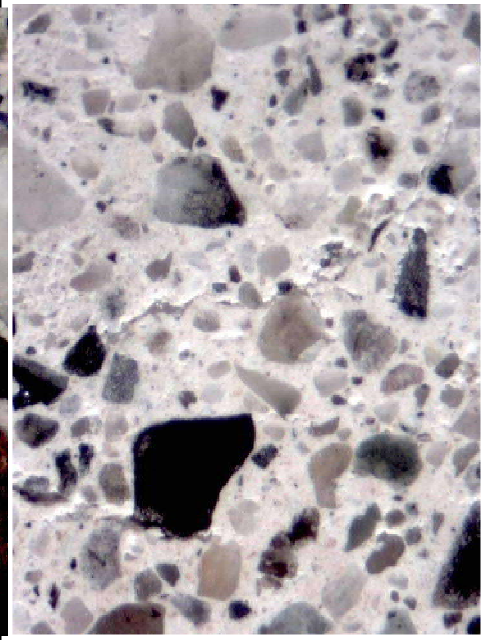
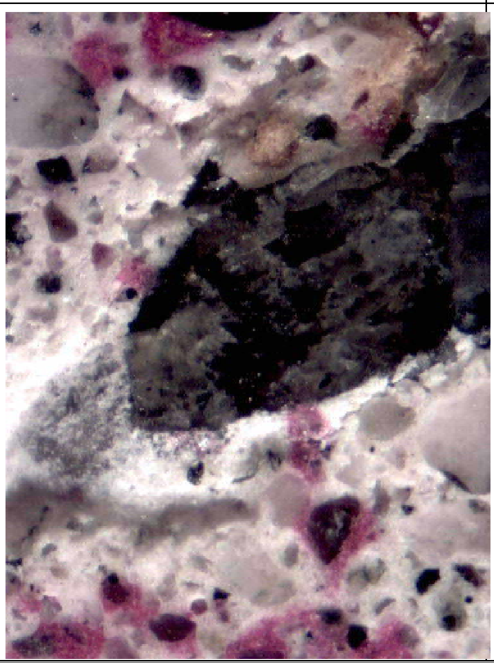
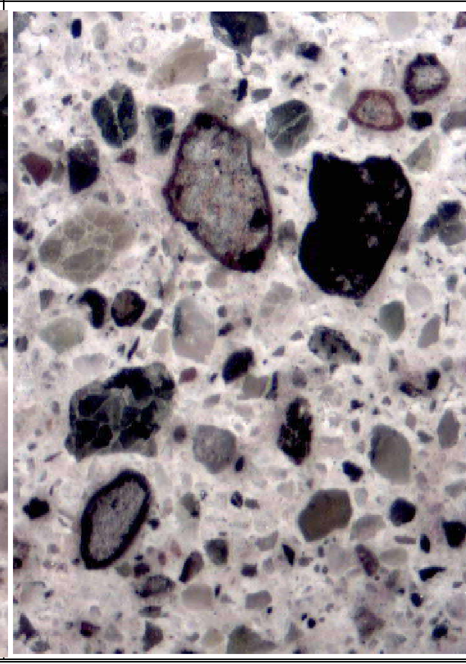
COWI
138 13th Street East
North Vancouver, BC V7L 0E5
Attention: Mr. Brad Pease, Ph.D.

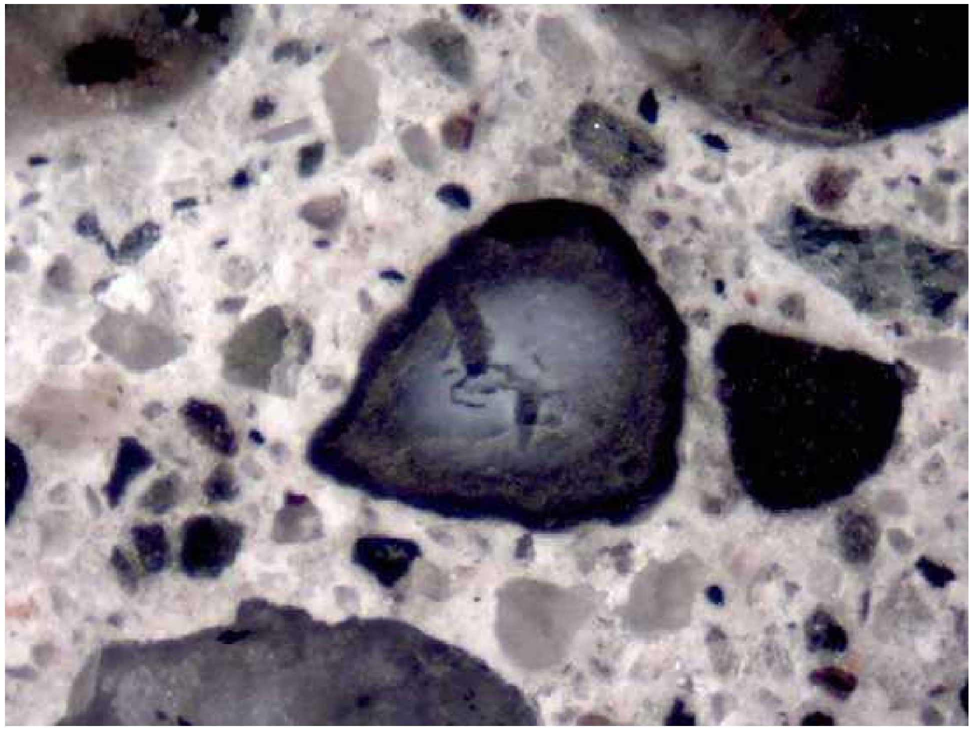
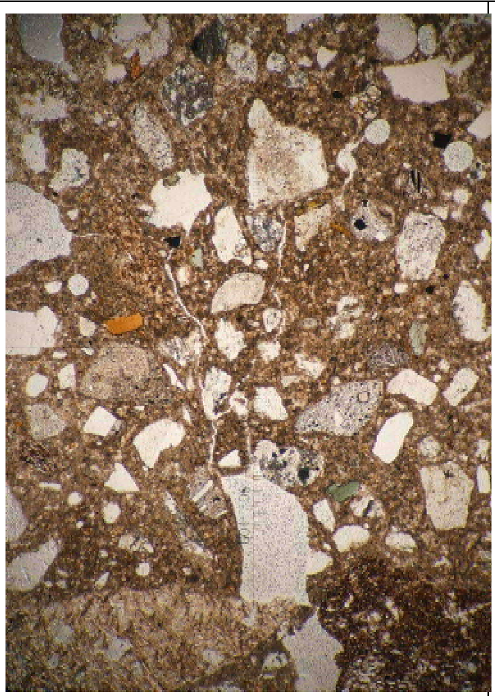
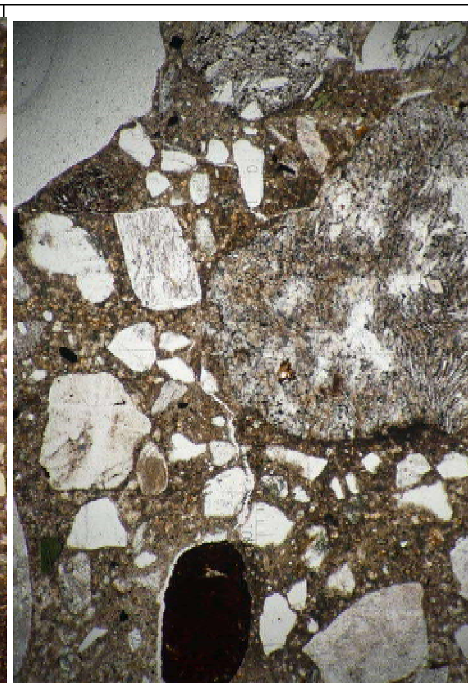
Project number: 20149024
September 17, 2020

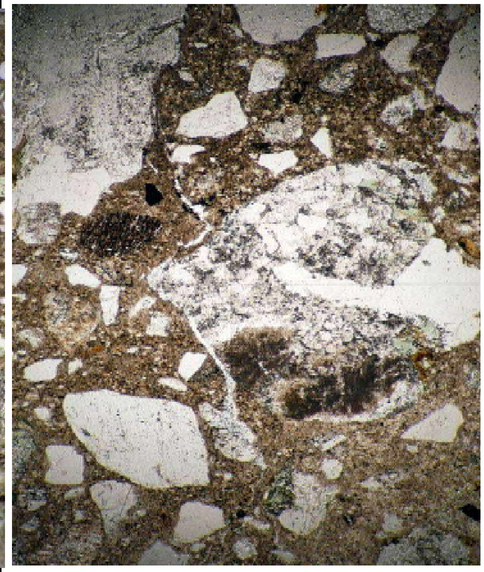
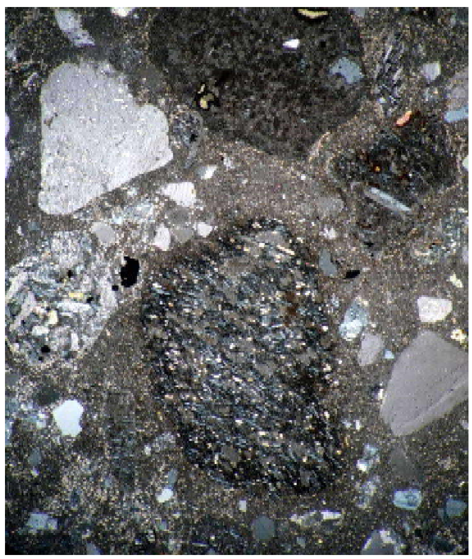
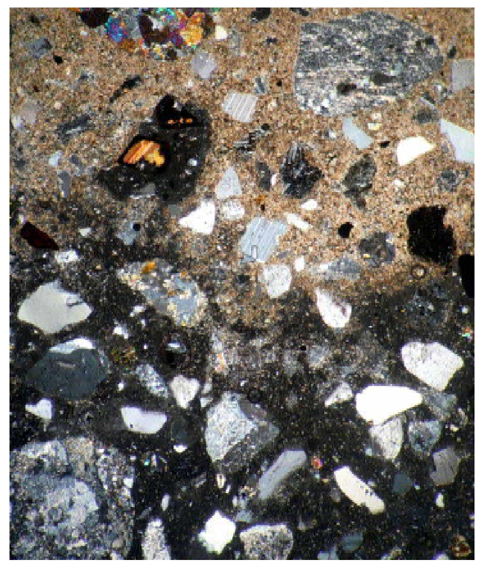
PROJECT:	**Massey Tunnel**		
Sample:	Core C1	**Location:**	Joint concrete

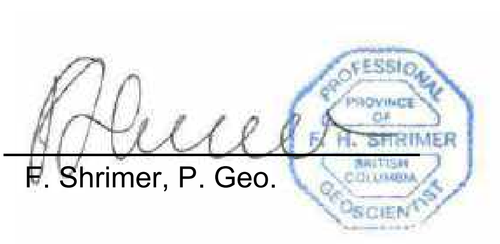
SAMPLE TYPE – GENERAL	94 mm diameter drilled core. Approximately 100 mm length. Outer surface is formed, inner surface is an uneven fracture. No reinforcing steel.
Aggregate maximum size	22 mm
Aggregate grading	Well-graded coarse and fine aggregates
Concrete consolidation	Concrete is generally dense and well-consolidated. Non air-entrained.
Cement paste	Paste is generally firm, and is carbonated to light buff pinkish in the outer 30 mm. Cement is well-hydrated. Non-air-entrained.
Coarse Aggregate	Multi-lithic natural gravel, with particle shapes that range from subangular/irregular to well-rounded. Rock lithologies are dominated by granitic and volcanic rocks with lesser amounts of gneiss, quartzite, schist, quartz sandstone, chert and siltstone. Most igneous rocks exhibit variable alteration.
Fine Aggregate	Fine aggregate is a natural sand composed of lithic fragments of granite, a variety of volcanic rocks, quartzite, sandstone, chert, gneiss, quartz, feldspar, epidote, iron oxides and mica.
Description	The concrete is well consolidated and generally exhibits good contact between paste and aggregate. Outer concrete is carbonated to a depth of 30 mm. There are zones of softer/porous paste characterized by debonding from aggregates within the sample; these are typically localized but often host cracks and disturbed paste. Reaction rims, particularly on volcanic rocks, are common throughout, also occurring on granitic rocks, quartzite/sandstone and chert. A few examples contain fresh Alkali-Silica Reaction (ASR) gel that has developed post-preparation of the sample. Other features related to ASR include cracks within aggregates and paste that contain ASR gel, and debonding sites where ASR gel is present.
Defects	Paste at the outer 30 mm is carbonated. Some patches of softer and more porous paste, often disturbed or hosting partial debonding of aggregates, were observed. Alkali-silica reaction (ASR) is indicated; features associated with ASR include reaction rims on fine and coarse aggregates, cracks extending through paste and through aggregate particles. Some cracks are devoid of ASR gel. Some debonded aggregates are observed.

Photos	
1.View of the sample prior to sawcutting.	
2. Cut and polished sample – outer surface is at bottom of view. Carbonation depth indicated by stain.	

3a. Three aggregate particles debonded from paste, magnification 10x, field of view 8.7 mm long. 3b. Crack in soft/weak paste. Mag. 20x, fov = 5 mm.		
4a. Debonded aggregate with ASR gel. Magnification of 20x, with a field of view of 5 mm long. 4b. Group of volcanic rocks with reaction rims, mag. 10x, fov 8.7 mm.		

5. Chert particle with reaction rim, and accumulation of fresh ASR gel on prepared polished surface. Mag. 15x, fov = 6.2 mm long.	
6a. Plane-polarized light view showing a series of cracks in the paste, some associated with ASR gel. Mag. 50x, fov 3 mm. 6b. Plane-polarized light view showing crack that passes along two aggregates and passes through paste. Magnification is 50x, fov 3 mm.	 

7a. Crack passes along a volcanic aggregate particle. Mag. 50x, fov 3 mm. 7b. ASR gel-lined crack in paste Mag. 50x, field of view is 3 mm in length.		
8a. Volcanic aggregate with reaction rim, seen in cross-polarized light, magnification 50x, fov 3 mm. 8b. Carbonation front about 30 mm below the top surface is seen as brown paste, uncarbonated paste appears black/dark grey-brown. Mag. 50x, fov is 3 mm length.		
SUMMARY	The outer ~30 mm of the paste is carbonated. Localized patches of softer, more absorbent paste are observed in the sample. Frequent indications of Alkali-Silica Reaction (ASR) observed in the concrete. Cracking in paste is moderate. Reaction rims on some aggregate types are common.	

Petrographer: 

F. Shrimer, P. Geo.

DATE: September 17, 2020

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE

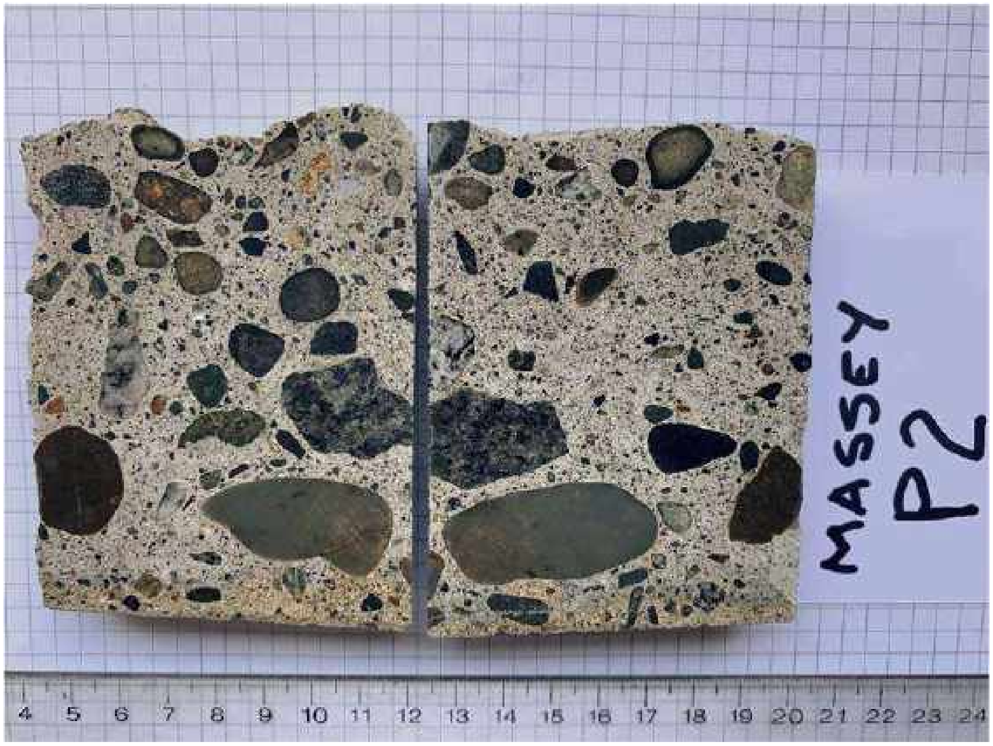
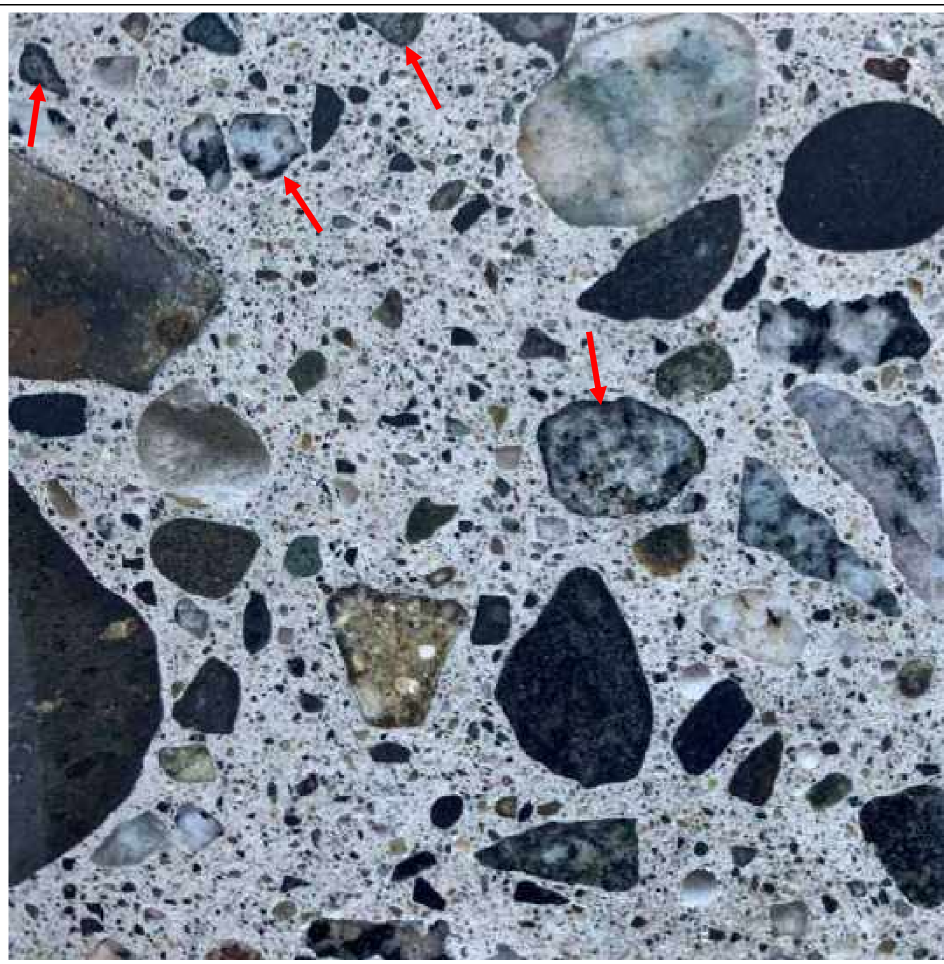
ASTM C856-18

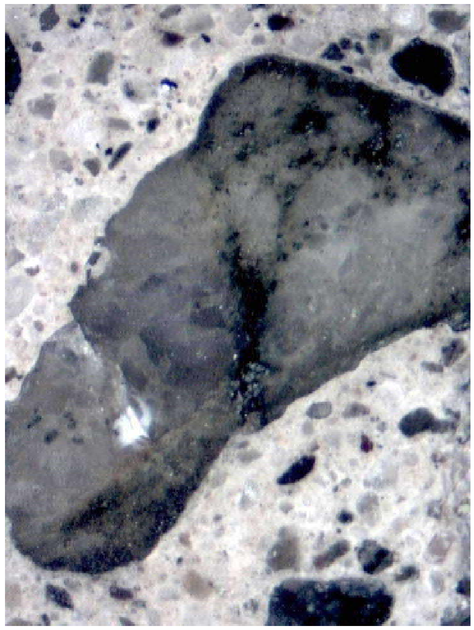
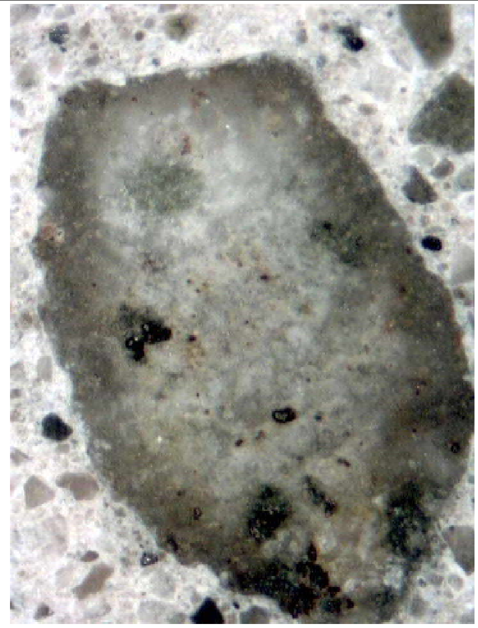
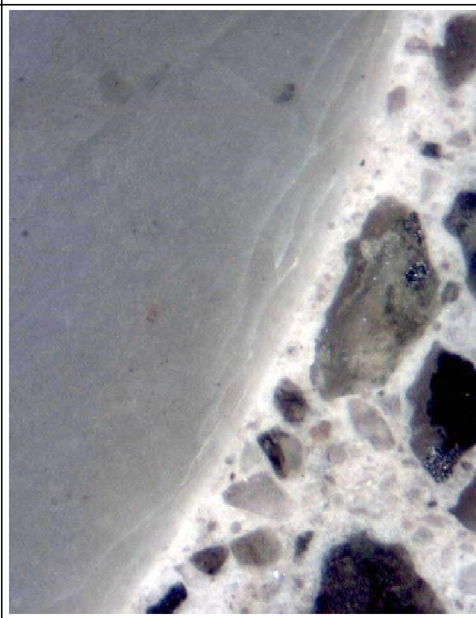
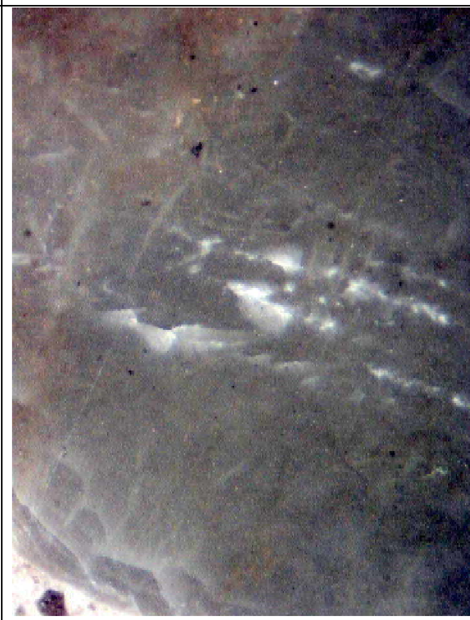
COWI
138 13th Street East
North Vancouver, BC V7L 0E5
Attention: Mr. Brad Pease, Ph.D.

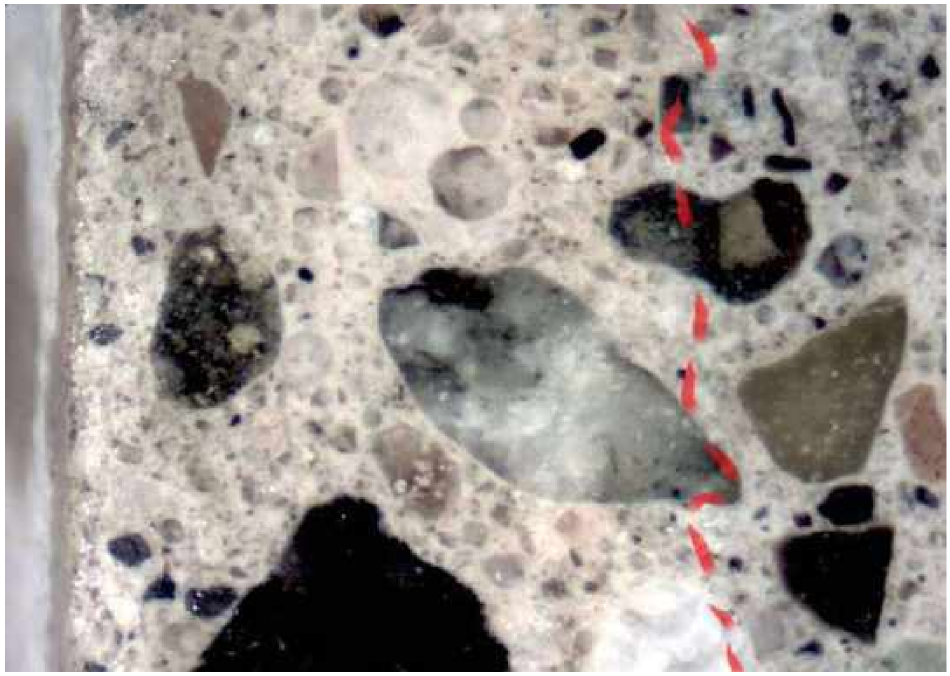
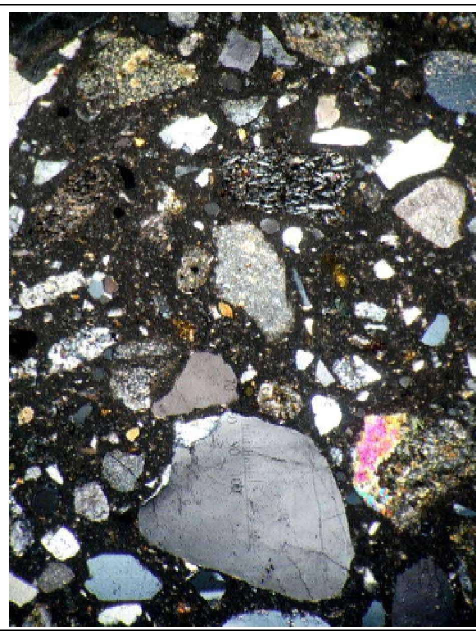
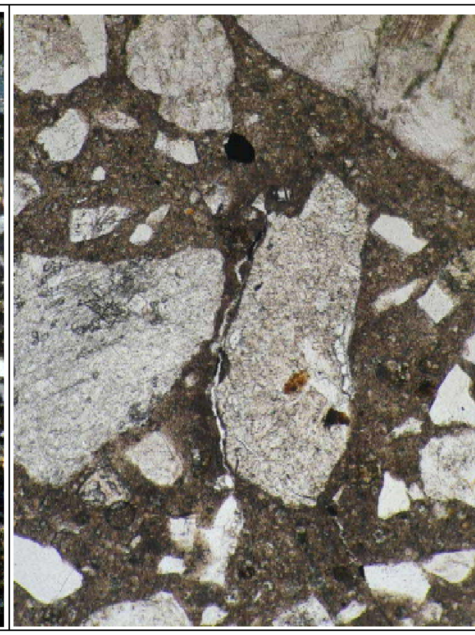
Project number: 20149024
September 16, 2020

PROJECT:	**Massey Tunnel**		
Sample:	Core P2	**Location:**	

SAMPLE TYPE – GENERAL	73 mm diameter drilled core. No reinforcing steel observed in sample.
Aggregate maximum size	19 mm
Aggregate grading	Well-graded coarse and fine aggregates
Concrete consolidation	Concrete is generally dense and well-consolidated. Non air-entrained.
Cement paste	Paste is generally firm to hard, and is light grey/cream in colour. Cement is well-hydrated. Non-air-entrained. Outer ~7-12 mm is carbonated.
Coarse Aggregate	Multi-lithic natural gravel, with particle shapes that range from subangular/irregular to well-rounded. Rock lithologies are dominated by volcanic and granitic rocks, with lesser amounts of diorite, gneiss, quartzite, quartz sandstone, chert and siltstone. Many of the igneous rocks exhibit variable degrees of alteration.
Fine Aggregate	Fine aggregate is a natural sand composed of lithic fragments of quartzite, sandstone, chert, granite, limestone, gneiss, quartz and feldspar.
Description	The concrete is well consolidated and generally exhibits good contact between paste and aggregate. A small amount of ASR-related features are observed in the sample, including reaction rims, minor ASR gel located within cracks in the paste, voids in the paste and cracks in aggregates. In thin-section, dense paste encloses fine and coarse aggregate; cracking is restricted to microcracking and is generally rarely observed.
Defects	Carbonation of paste was observed in the outer 7 -12 mm of the core. Indications of ASR in the concrete are observed throughout the core but are minor in frequency and relative effect on the concrete.

Photos	
1. View of the sample after sawcutting and polishing.	
2. Detail of polished sample – reaction rims are visible on some of the granitic aggregates (arrows).	

3a. Reaction rim has formed on the granite particle, and ASR gel (white) is observed on the interior of the aggregate, along a crack. 3b. Reaction rim on a metagranite particle. Both images are at a magnification of 10x, field of view 8.7 mm long.		
4. Two views of the same siliceous aggregate particle, showing ASR-shattered edge and resulting crack development in the left view, and ASR-filled cracks within the particle's interior. Magnification 10x, FOV is 8.8 mm length.		

5. A profile is observed in the paste with depth below the surface in this view, where the cement transitions from the pinkish-beige carbonated outer zone to the light grey zone at about 6 mm depth, indicated with red line. Magnification 10x, FOV is about 8.7 mm long.	
6. Cross-polarized light view of thin-section illustrating overall dense paste enclosing aggregates. 50x magn., field of view 3 mm length. 7. Plane-polarized light view showing an example of a two microcracks in the paste containing ASR gel. Mag. 100x, FOV 1.5 mm.	 
SUMMARY	The concrete is well-consolidated with adequate proportioning and distribution of aggregates in the paste matrix. Paste interface is good. The outer 7-12 mm of the concrete is carbonated. ASR-related effects are minimal: minor microcracking, reaction rims, ASR gel in a few locations.

Petrographer: ______________________ DATE: September 16, 2020

F. Shrimer, P. Geo.

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE
ASTM C856-18

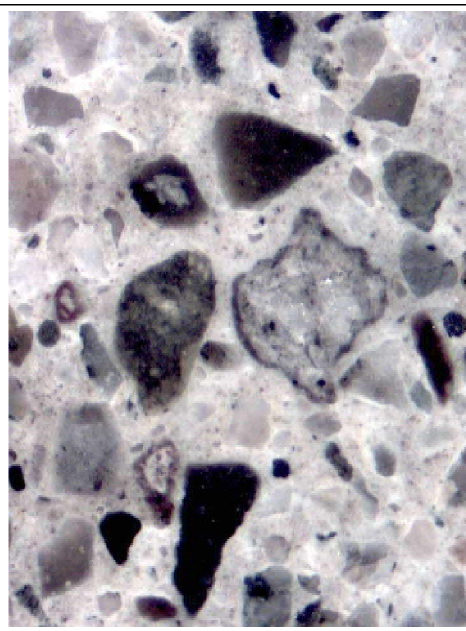
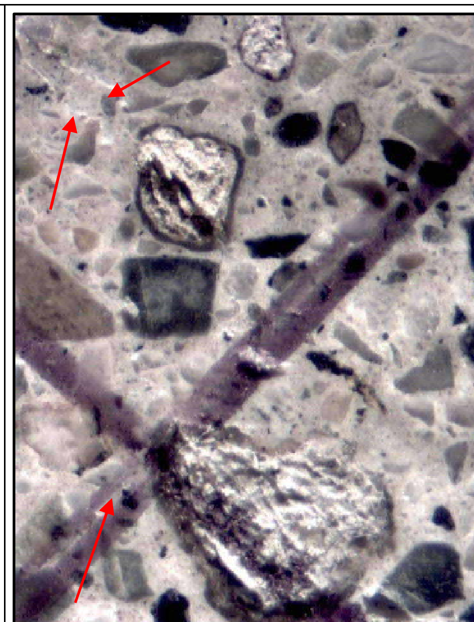
GOLDER

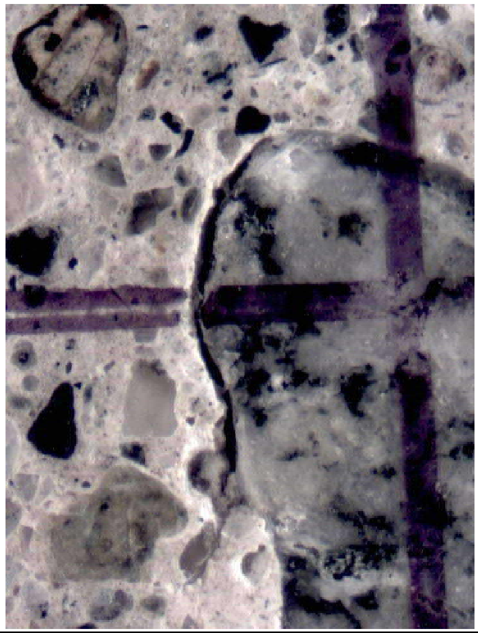
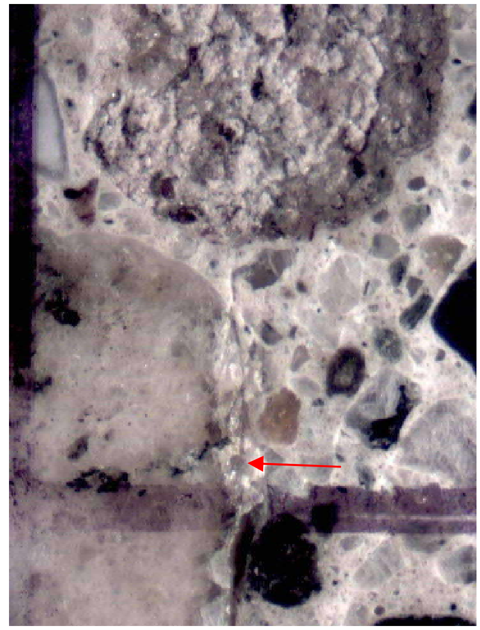
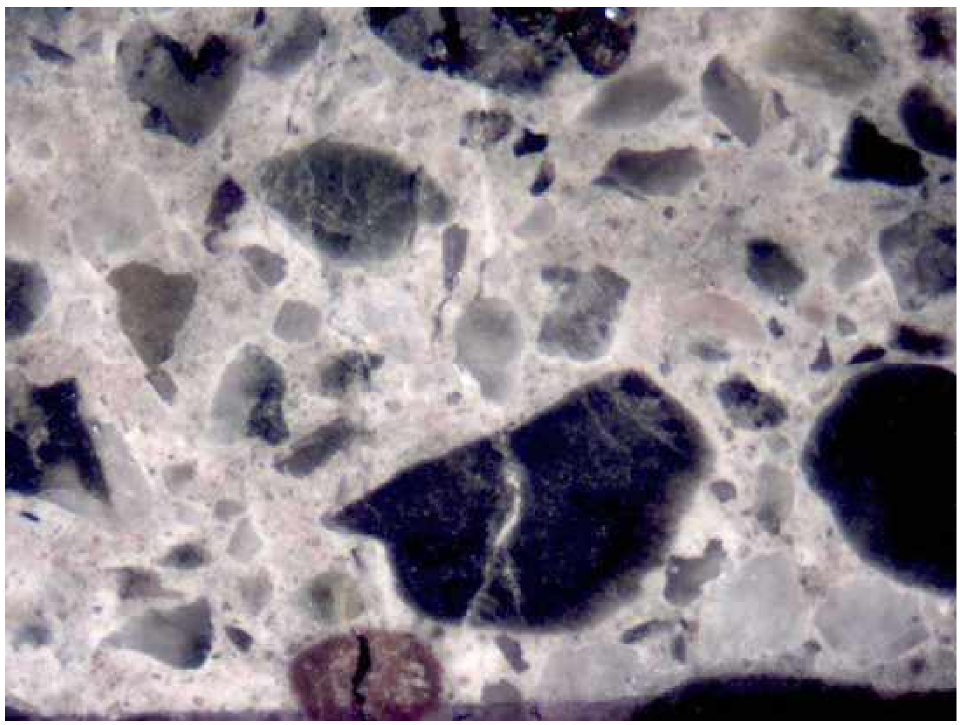
COWI
138 13th Street East
North Vancouver, BC V7L 0E5
Attention: Mr. Brad Pease, Ph.D.

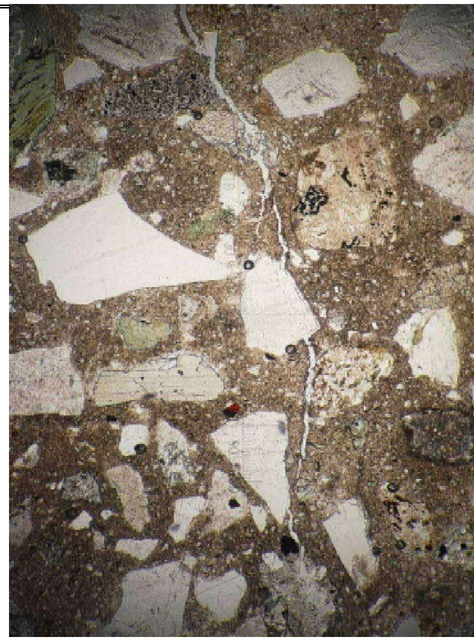
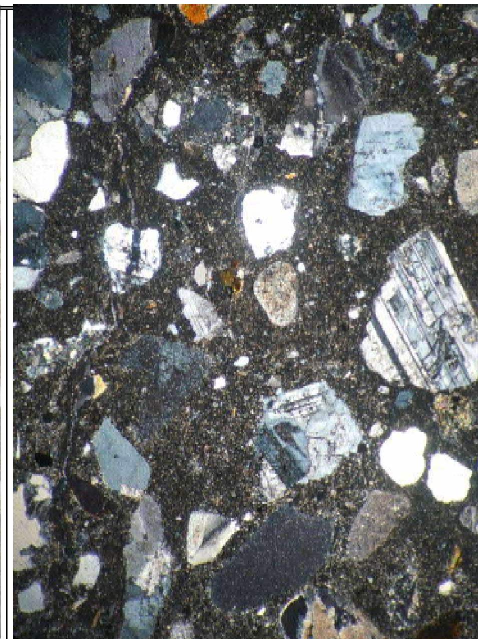
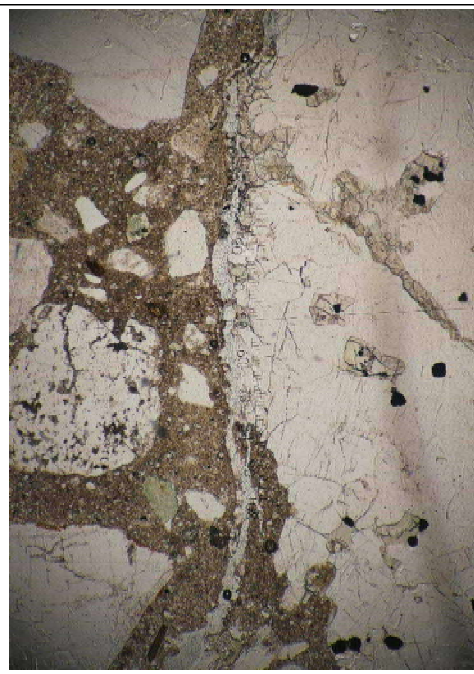
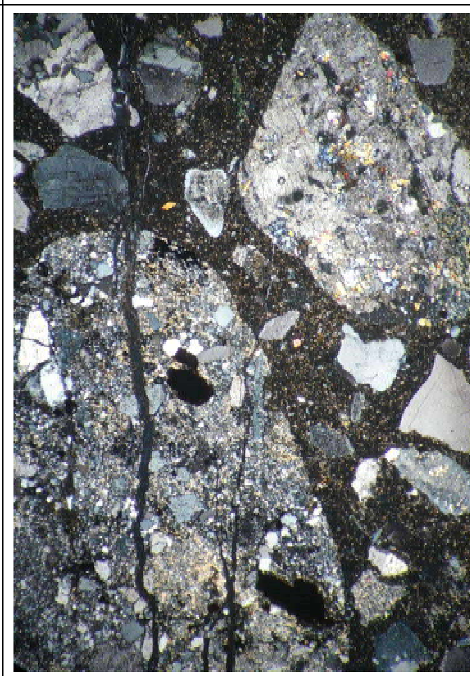
Project number: 20149024
September 14, 2020

PROJECT:	**Massey Tunnel**		
Sample:	Core P3	**Location:**	

SAMPLE TYPE – GENERAL	94 mm diameter drilled core. Approximately 230 mm length. Upper end sawcut, opposite end is a fracture surface. Core broken on (clean) fracture at about 50/80 mm from upper end (uneven break). No reinforcing steel.
Aggregate maximum size	14 mm
Aggregate grading	Well-graded coarse and fine aggregates
Concrete consolidation	Concrete is generally dense and well-consolidated. Non air-entrained.
Cement paste	Paste is generally firm. Cement is well-hydrated. Application of phenolphthalein indicated no to little carbonation in the cement. Non-air-entrained.
Coarse Aggregate	Multi-lithic natural gravel, with particle shapes that range from subangular/irregular to well-rounded. Rock lithologies are dominated by volcanic and granitic rocks with lesser amounts of gneiss, quartzite, schist, quartz sandstone, chert and siltstone.
Fine Aggregate	Fine aggregate is a natural sand composed of lithic fragments of granite, a variety of volcanic rocks, quartzite, sandstone, chert, gneiss, quartz, feldspar, epidote, iron oxides and mica.
Description	The concrete is well consolidated and generally exhibits good contact between paste and aggregate. There are some instances of debonded aggregate-paste, although these are typically partial only. Abundant features associated with ASR were observed: (1) reaction rims on siliceous, granitic and volcanic aggregates; (2) cracks through paste and aggregates, containing ASR gel; (3) debonding of aggregates from paste, with associated ASR gel-lined aggregate surrounds; (4) ASR gel located within air voids. While the reaction rims and some of the ASR deposits in the paste are visible to the unaided eye, many of the ASR gel-lined cracks are not visible unless using a microscope. Cracks that do not contain secondary deposits were observed in aggregates and in paste: these may be the result of mechanical disturbance such as loading or freeze-thaw activity or another cause, or former ASR gel has been removed. In thin-section, fine cracks are observed passing through paste and through/from some aggregate particles. ASR gel is observed in some cracks, and surrounding aggregate, or in cracks within aggregate. Volcanic rocks and siliceous rocks are the most commonly observed rock types associated with ASR features.
Defects	Alkali-silica reaction (ASR) features include reaction rims on fine and some coarse aggregates (often, volcanic), occasional cracks extending from and through aggregate particles, and rarely through paste. Some cracks are devoid of ASR gel. Debonded aggregates are observed, typically in areas of softer paste, but on occasion associated with ASR gel-lined sockets.
Photos	

1.View of the sample after sawcutting and polishing.	
2. Two views showing corroded volcanic aggregate particles with reaction rims. In the right view, cracks extend from/to the aggregate particles and are lined with ASR gel (arrows). 10x magnification, field of view 8.7 mm length.	 

3. Debonded aggregates shown in both views with associated ASR gel-filled cracks. Mag. 10x, fov 8.7 mm.	 
4. Microcracks extend through paste and through aggregate particles, filled with ASR gel. Open crack in purplish aggregate at bottom of view. Magnification 10x, field of view is 8.7 mm across.	

5a,b. Views in plane- (left) and cross-(right) polarized light illustrating hairlines cracks that pass through both paste and aggregates. Both view magnification 50x, fov 3 mm.		
6. Two views illustrating cracks that pass through the pass and aggregates. Both views at a magnification of 10x, fov = 8.7 mm. Left image in plane-polarized light, right is in cross-polarized light.		
SUMMARY	The concrete contains a variety of open (larger) and fine cracks, some lined/filled with ASR gel, some devoid of fillings. Reaction rims are common on aggregate particles. ASR gel fills some voids. Partially debonded aggregates observed. Corroded aggregates are observed, mostly in the fine aggregate size range.	

Petrographer: ____________________ DATE: September 14, 2020

F. Shrimer, P. Geo.

PROFESSIONAL GEOSCIENTIST PROVINCE OF BRITISH COLUMBIA F. H. SHRIMER

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE
ASTM C856-18

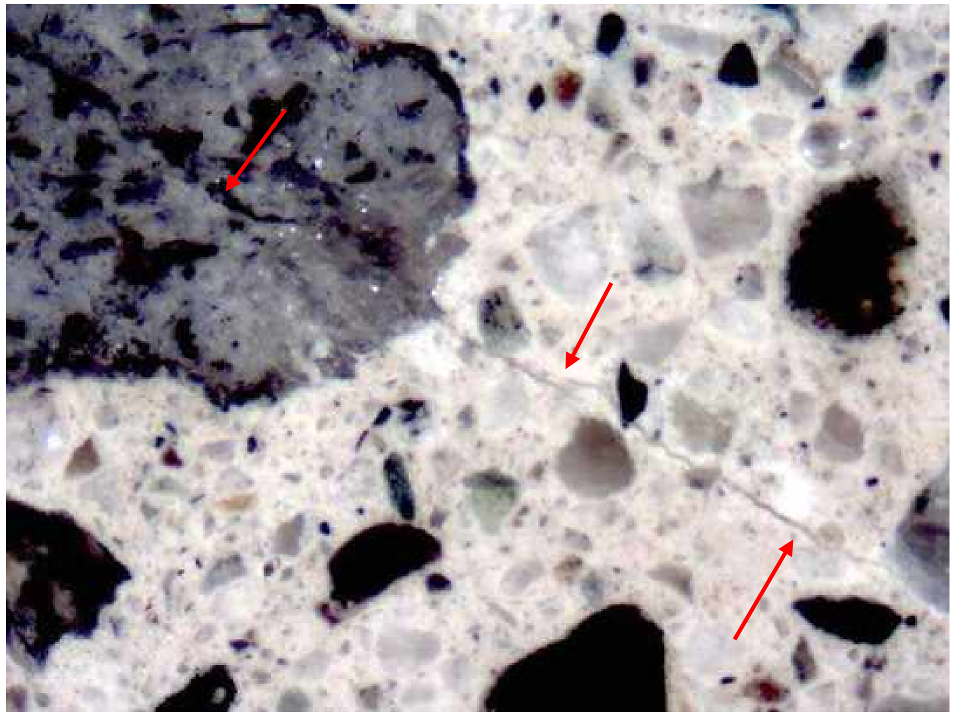
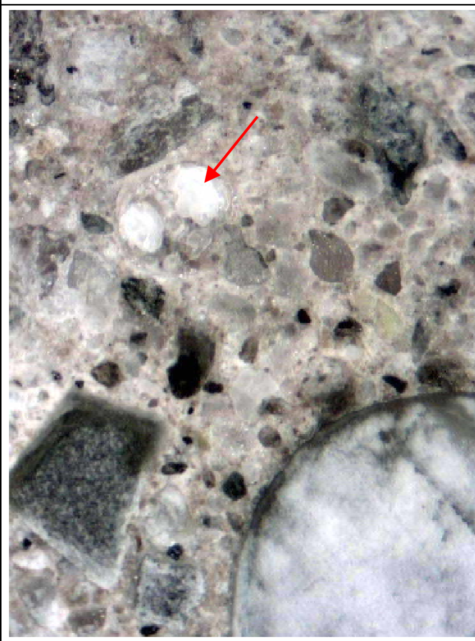
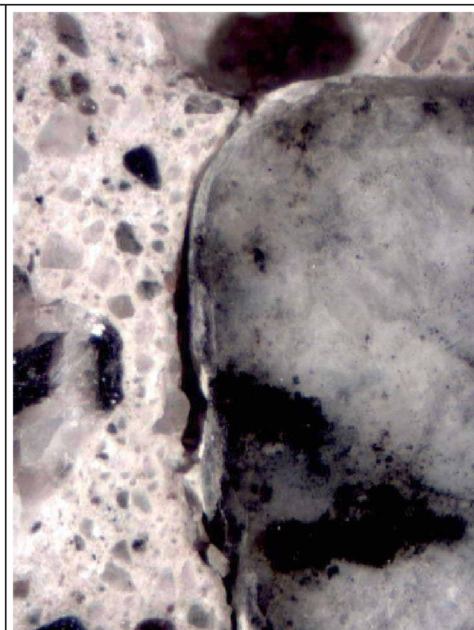
COWI
138 13th Street East
North Vancouver, BC V7L 0E5
Attention: Mr. Brad Pease, Ph.D.

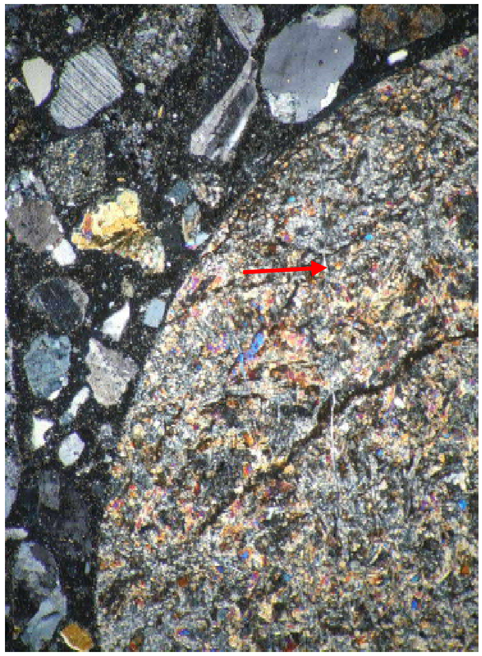
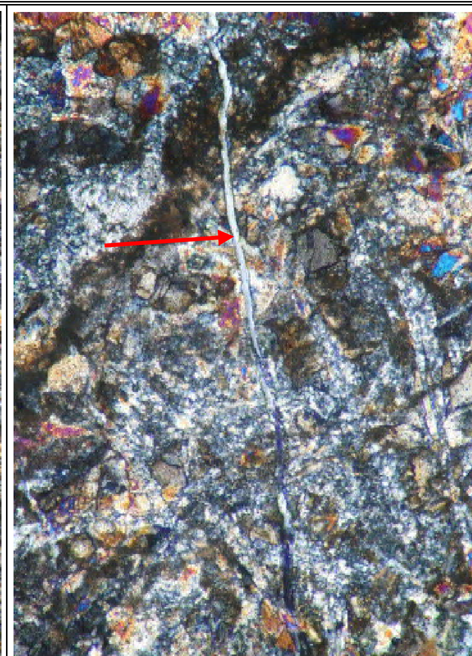
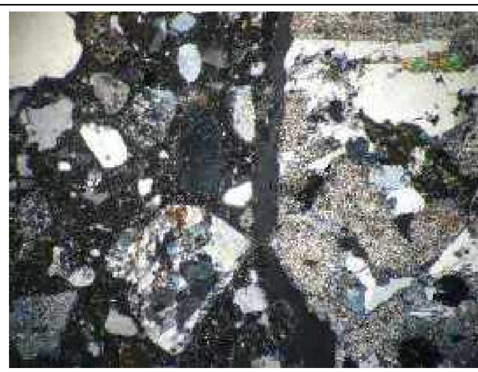
Project number: 20149024
September 15, 2020

PROJECT:	**Massey Tunnel**		
Sample:	Core P4	**Location:**	

SAMPLE TYPE – GENERAL	Two-part core: outer 63 mm is 94 mm-diameter; thereafter 70 mm diameter drilled core of about 242 mm, for total of 305 mm length. Outer surface formed, lower surface is an uneven fracture. No reinforcing steel.
Aggregate maximum size	20 mm
Aggregate grading	Well-graded coarse and fine aggregates
Concrete consolidation	Concrete is generally dense and well-consolidated. Non air-entrained.
Cement paste	Paste is generally firm with a few localized patches of softer cement, and is lightly carbonated in the outer 2 mm. Cement is well-hydrated. Non-air-entrained.
Coarse Aggregate	Multi-lithic natural gravel, with particle shapes that range from subangular/irregular to well-rounded. Rock lithologies are dominated by granitic and volcanic rocks with lesser amounts of gneiss, quartzite, schist, quartz sandstone, chert and siltstone.
Fine Aggregate	Fine aggregate is a natural sand composed of lithic fragments of granite, a variety of volcanic rocks, quartzite, sandstone, chert, gneiss, quartz, feldspar, epidote, iron oxides and mica.
Description	The concrete is well consolidated and generally exhibits good contact between paste and aggregate. A few cracks are observed that are oriented approximately parallel to the core's surface. Some features associated with Alkali-Silica Reaction (ASR) are evident throughout the sample, including reaction rims on certain aggregates (primarily volcanic rocks, some granitic rocks, chert, and quartzite), cracks that appear within the paste and aggregates, lined wit ASR gel, and debonded aggregates that are surrounded by ASR gel. In thin-section, both open and finer microcracks are observed passing through paste and through/from some aggregate particles. ASR gel is observed in some cracks, and surrounding aggregate, or in cracks within aggregate. Volcanic rocks and siliceous rocks are the most commonly observed rock types associated with ASR features.
Defects	Minor carbonation has affected the outer ~2 mm of the concrete. Alkali-silica reaction (ASR) is indicated as having occurred in the concrete. Features that were observed and are associated with ASR include reaction rims on fine and some coarse aggregates (often, volcanic), occasional cracks extending from and through aggregate particles, and rarely through paste. Most cracks are devoid of ASR gel. Debonded aggregates are observed, typically in areas of softer paste, but on occasion associated with ASR gel-lined sockets.

Photos	
1.Views of the sample prior to and after sawcutting and polishing.	

2. Detail of the polished sample, illustrating an ASR-affected volcanic aggregate particle with a reaction rim, an ASR gel-filled crack extending from the aggregate into the adjacent paste (arrows). Magnification 10x, field of view is 8.7 mm across.		
3a. Reaction rim on siliceous aggregate particle at lower right of view; ASR gel-filled void indicated with arrow. Mag. 10x, FOV of view is 8.7 mm. 3b. Debonded aggregate particle, with some ASR gel in resulting crack. Mag. 10x, fov = 8.7 mm.		

4. Two thin-section views, both seen in cross-polarized light, showing an altered volcanic coarse aggregate particle with an internal crack that passes into the surrounding paste. Left view is at a magnification of 50x, with a lengthwise fov of 3 mm; the right view is at a magnification of 200x and a fov of 0.8 mm.		
5a. A reaction rim is seen surrounding an altered granitic aggregate (dominating the right portion of the view). 5b. A volcanic aggregate particle dominates the lower left of this thin-section view, set in dense paste that contains quartz grains, other volcanics and a granite at upper right. Both view cross-polarized light, mag. 50x, fov = 3 mm long.		
SUMMARY	The concrete contains a moderate amount of open and fine cracks, some lined/filled with ASR gel, some devoid of fillings. Reaction rims are common on some particles. ASR gel fills some voids. Minor carbonation in outer 2 mm of concrete.	

Petrographer: ______________________ DATE: September 15, 2020

F. Shrimer, P. Geo.

PROFESSIONAL PROVINCE OF F. H. SHRIMER BRITISH COLUMBIA GEOSCIENTIST

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE -- SUPPLEMENTAL

ASTM C856-18

COWI
138 13th Street East
North Vancouver, BC V7L 0E5
Attention: Mr. Brad Pease, Ph.D.

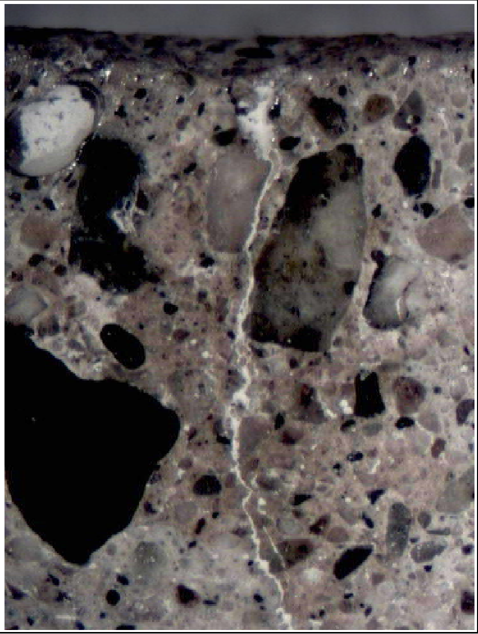
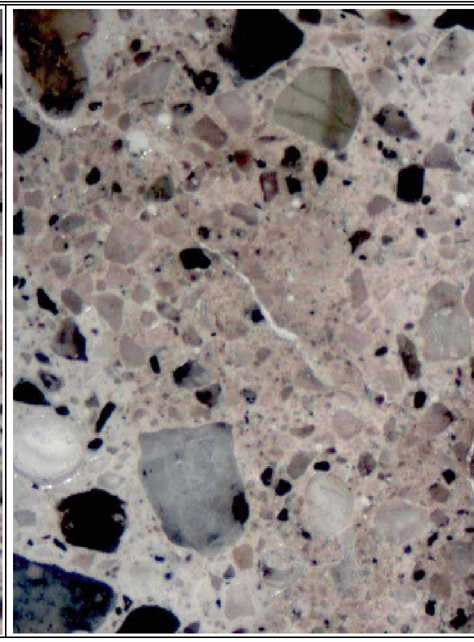
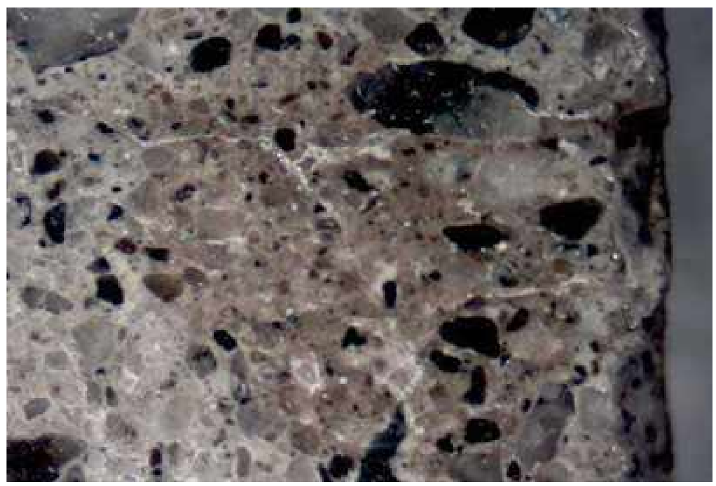
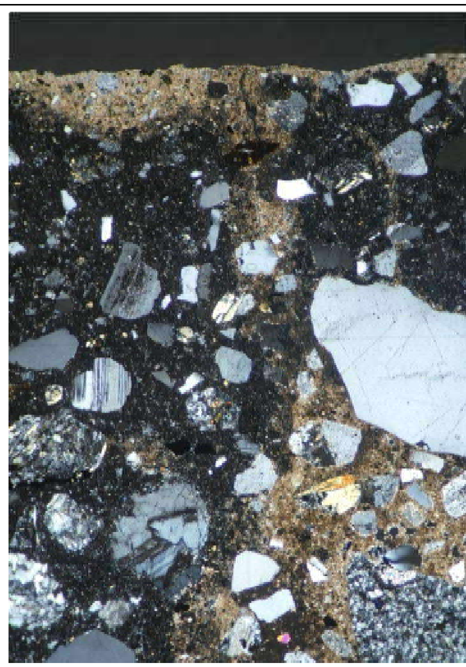
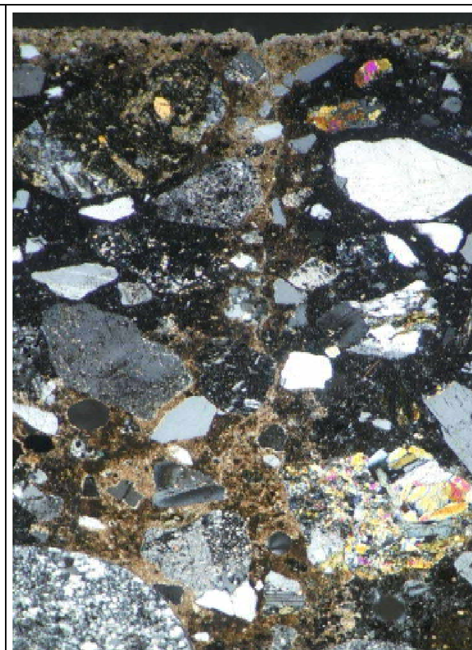
Project number: 20149024
November 12, 2020

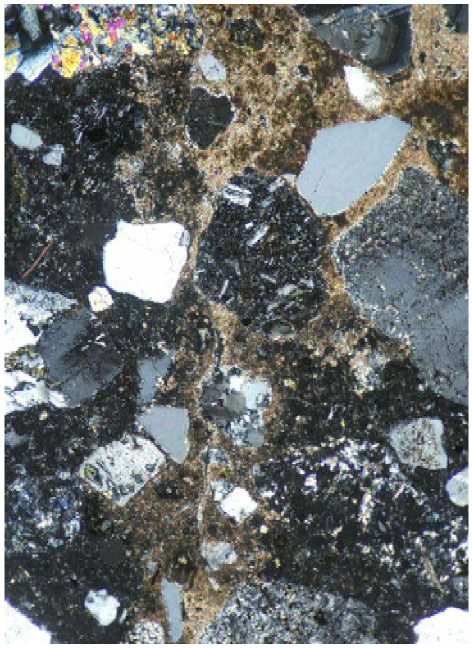
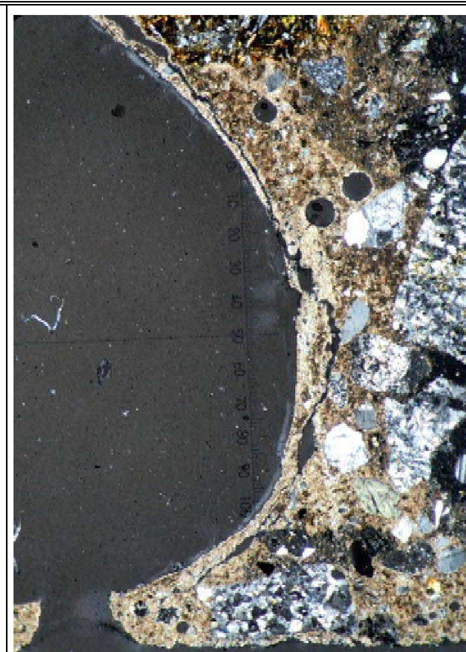
PROJECT:	**Massey Tunnel**		
Sample:	Core P4	**Location:**	**WUE - precast**

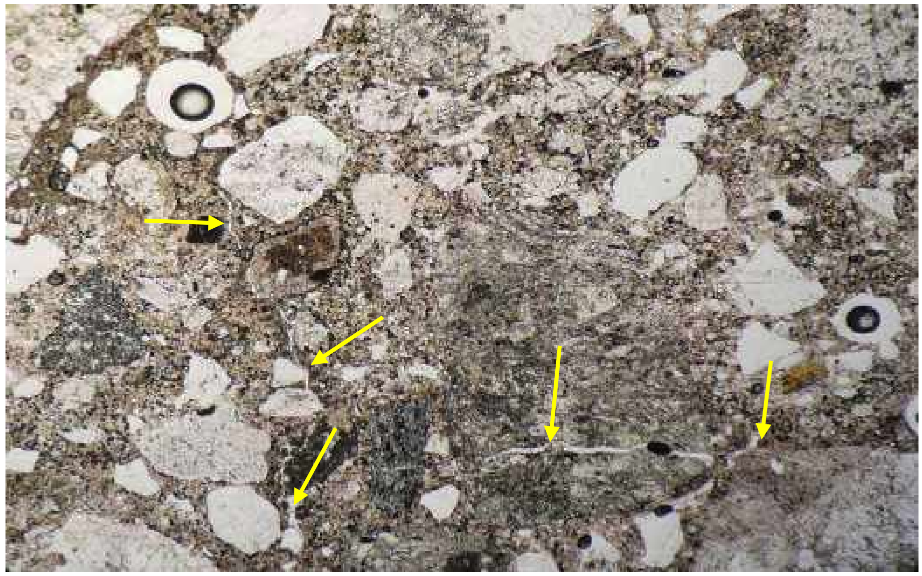
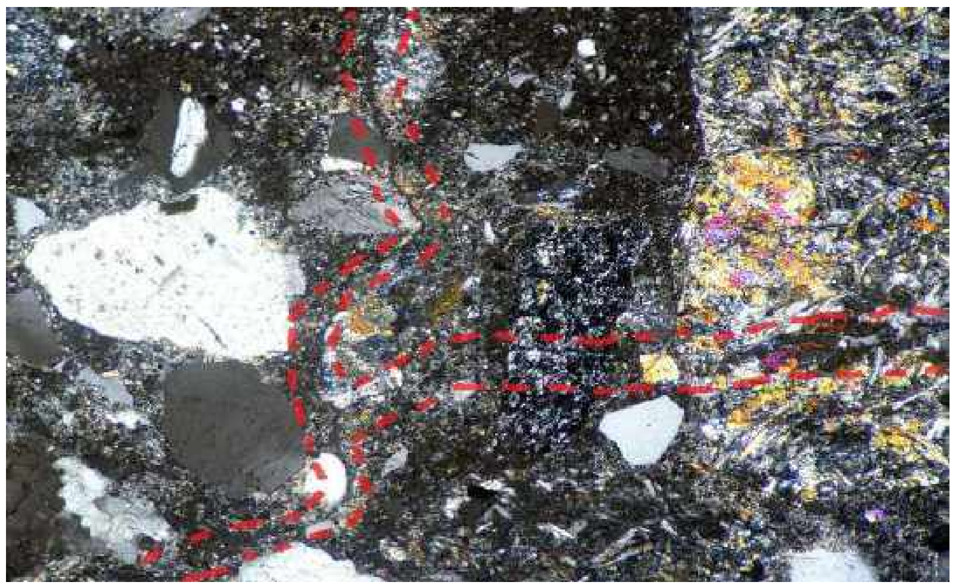
SAMPLE TYPE – GENERAL	Outer portion of core is 63 mm in length and has a diameter of 94 mm. The lower/inner portion is 242 mm in length and has a diameter of 70 mm.
Cracking Description in stereomicroscope	The concrete is well consolidated and generally exhibits good contact between paste and aggregate. Some cracks are observed that are oriented approximately parallel to the core's surface. These are considered likely to be the result of freeze-thaw activity in most cases. The small sample surface area is inadequate to evaluate whether this is universally the case. Crack expressions at the core surface are accompanied by a brown colour, which is consistent with ASR. Some of the cracks originating at the surface of the core extend in random orientations further into the concrete: some are roughly parallel to the core axis and others curl into subparallel and irregular orientations: this may be due to the presence of aggregate particles that cause cracks to divert around them. This cannot be determined since the sample that is examined is essentially two-dimensional; the absent concrete above the plane of view cannot be viewed and the concrete below the plane of view, similarly, cannot be viewed. Cracks that present with brown traces on the surface of the concrete are observed to persist with that same brown colouration within the paste that surrounds the crack to variable depths, generally to about 10 or 15 mm, although in a few examples the brownish-coloured paste extends to about 25 mm depth. In many instances, cracks are lined in part or wholly by Alkali-Silica Reaction gel; some cracks appear devoid of deposits.
Cracking Description in thin-section samples	In thin-section, both open and finer microcracks are observed passing through paste and through/from some aggregate particles. ASR gel is observed in some cracks, surrounding aggregate particles, and in cracks within aggregate particles. Volcanic rocks and siliceous rocks are the rock types most commonly observed to be associated with ASR cracks and the presence of ASR gel. Some cracks pass through paste and extend alongside aggregates, while others extend from paste through aggregate particles. In many cases, the latter types are lined with ASR gel. Some cracks are devoid of visible deposits. It is unclear as to whether these represent freeze-thaw or other mechanical processes, or if preparation of the slide resulted in removal of the ASR gel product.

Photos	 
1.Views of the sample prior to and after sawcutting and polishing.	

2. Polished samples cut from core P4 to facilitate examination.	
3. Thin-sections and offcuts from core P4.	

4a. Views of cracks in P4 concrete. Left view is from surface, and right view is about 15 mm below surface. Both cracks are ASR-gel-lined. Both are at a magnification of 10x, with a lengthwise fov of 8.7 mm.		
5. A multi-trace crack extends through brownish-coloured paste. Magnification 10x, fov = 8.7 mm long.		
6a, b. Thin-section views in cross-polarized light at surface of core P4 showing cracks that extend through paste and expressed at surface. Discoloured paste follows the trace of the cracks beneath to surface to at least 8 mm (FOV = 8.7 mm). Magnification 50x.		

7a, b. Thin-section views, both in cross-polarized light, showing cracks that pass through the paste at depths of about 35 mm (left) and 45 mm (right). Left view is at 100x magnification, right view at 50x . FOVs are 1.8 mm and 3.5 mm respectively.		
8. A crack is seen passing alongside an aggregate particle in this thin-section view, in plane polarized light. Magnification 50x, field of view is 3.3 mm.		

9. View in thin-section, plane polarized light, showing a crack that extends through a reactive volcanic particle and passes through the surrounding paste. Magnification 50x, fov 3.3 mm.	
10. Detail of previous view, seen in cross-polarized light, and viewed at a magnification of 100x (field of view 1.75 mm). Cracks are outlined with dashes.	
SUMMARY	Cracks are observed throughout the core, most/many lined wholly or in part with ASR gel. Some cracks are absent of ASR gel – those that appear in the outer portion of the core (e.g., outer 50 mm) and are oriented parallel to the core surface may be considered as resulting from freeze-thaw activity. Other cracks that are absent of ASR gel may not conclusively be considerd to have resulted from other mechanisms. Some cracks contain ASR gel in only a portion of the visible crack expression; ASR gel, having dried and hardened may have been removed in sample preparation. An overview estimate suggests that at least 80% of the cracks may contain some amount of ASR gel

Petrographer: ______________________ DATE: November 12, 2020

F. Shrimer, P. Geo.

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE
ASTM C856-18

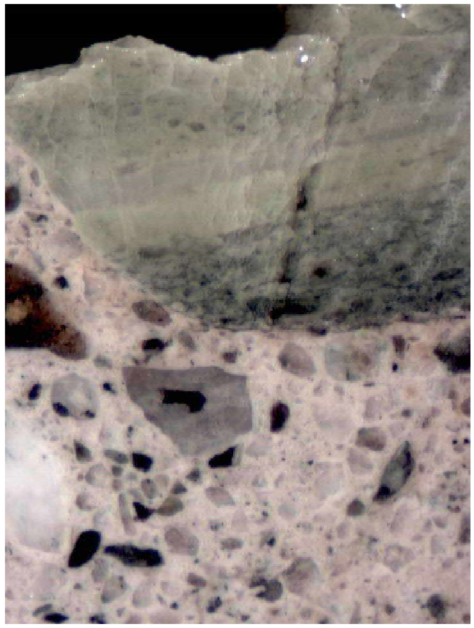
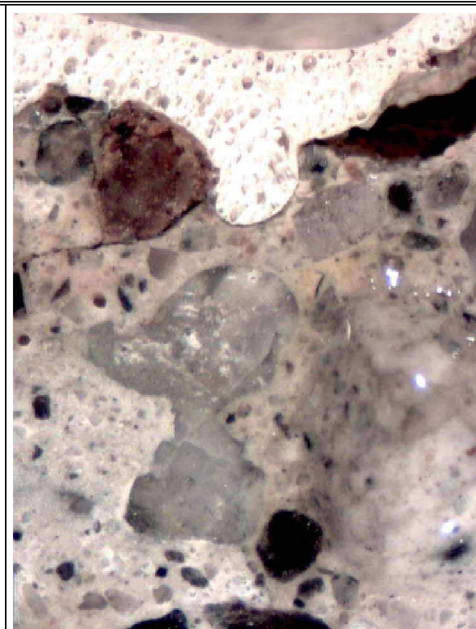
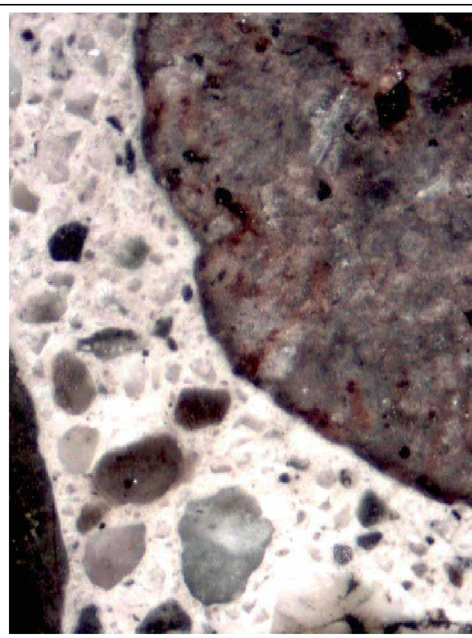
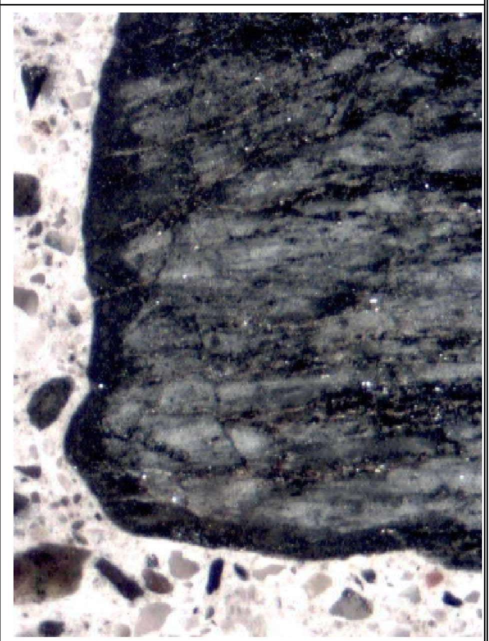
COWI
138 13th Street East
North Vancouver, BC V7L 0E5
Attention: Mr. Brad Pease, Ph.D.

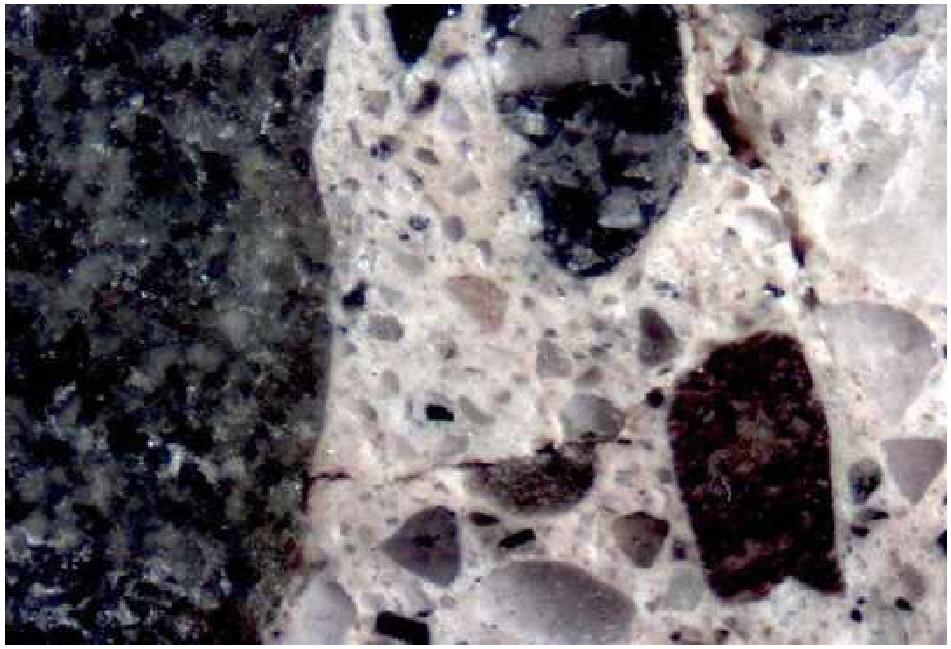
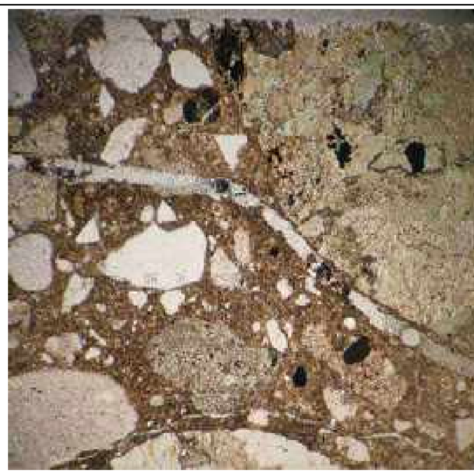
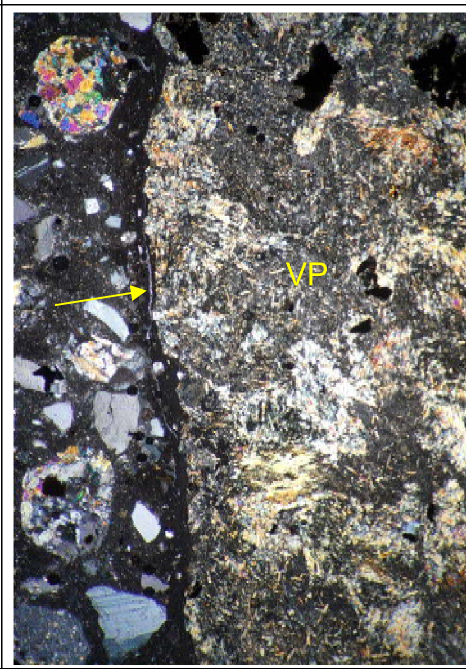
Project number: 20149024
September 17, 2020

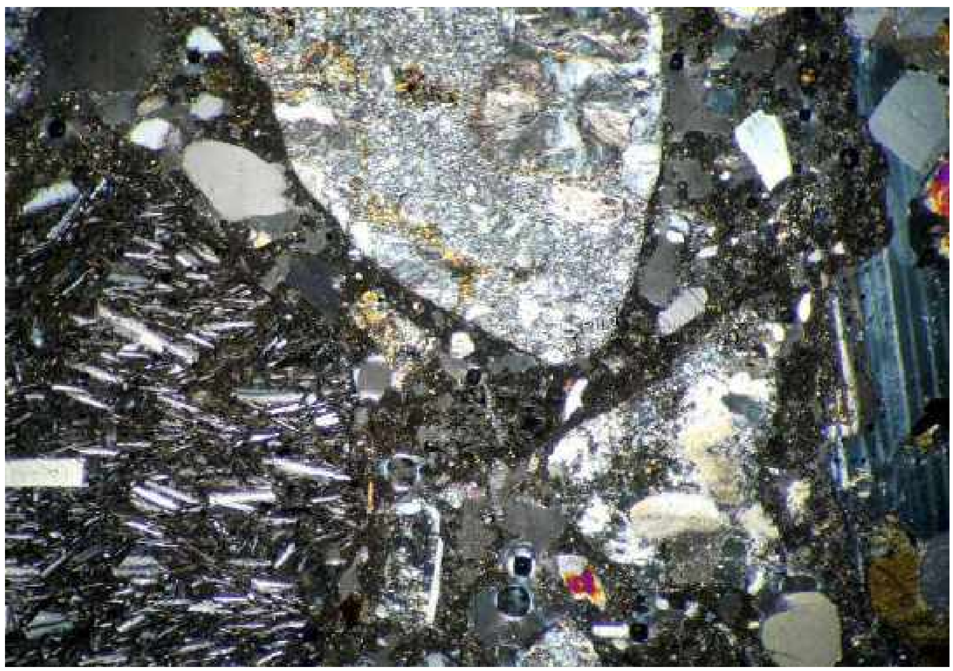
PROJECT:	**Massey Tunnel**		
Sample:	Core P5	**Location:**	

SAMPLE TYPE – GENERAL	94 mm diameter drilled core. Approximately 70 – 100 mm length. Outer surface uneven and coated with a ~1 mm-thick flexible coating which has partially separated from the concrete. Inner surface is uneven fracture. No reinforcing steel.
Aggregate maximum size	25 mm
Aggregate grading	Well-graded coarse and fine aggregates
Concrete consolidation	Concrete is generally dense and well-consolidated. Non air-entrained.
Cement paste	Paste is generally firm, and is discoloured to light buff pinkish in the outer ~ 5-10 mm. Cement is well-hydrated. Non-air-entrained.
Coarse Aggregate	Multi-lithic natural gravel, with particle shapes that range from subangular/irregular to well-rounded. Rock lithologies are dominated by granitic and volcanic rocks with lesser amounts of gneiss, quartzite, schist, quartz sandstone, chert and siltstone. Most igneous rocks exhibit variable alteration.
Fine Aggregate	Fine aggregate is a natural sand composed of lithic fragments of granite, a variety of volcanic rocks, quartzite, sandstone, chert, gneiss, quartz, feldspar, epidote, iron oxides and mica.
Description	The concrete is well consolidated and generally exhibits good contact between paste and aggregate. Outer concrete is discoloured to a light buff/beige to pinkish tone to a depth of 5 – 15 mm. Some aggregates within this zone have multiple fine 'shatter cracks' but otherwise do not exhibit attendant distress in the paste. Some open cracks in the surface extend into the concrete up to 15 mm. Reaction rims, some cracking but only very rare instances of Alkali-Silica Reaction (ASR) gel were observed, indicating a mild ASR in the concrete. Cracking is not pervasive. In this section, only very rarely are fine cracks observed passing through paste and through/from some aggregate particles. ASR gel is observed in some cracks, and surrounding aggregate, or in cracks within aggregate. Volcanic rocks and siliceous rocks are the most commonly observed rock types associated with ASR features.
Defects	Paste at the outer 5-15 mm is discoloured, ostensibly due to a reported fire. Paste and aggregate characteristics in this zone are consistent with a fire. A mild alkali-silica reaction (ASR) is indicated; features associated with ASR include reaction rims on fine and some coarse aggregates (often, volcanic), occasional cracks extending through paste and (rarely) through aggregate particles. Most cracks are devoid of ASR gel. A few debonded aggregates are observed.

Photos	
1.View of the sample.	
2. Cut and polished sample.	

3a. Shattered aggregate at surface of core; it does not appear to have exerted significant force on the surrounding paste. Mag. 10x, field of view is 8.7 mm long. 3b. White porous coating at top of core is seen adjacent a crack expression at the surface. Mag. 10x, fov = 8.7 mm.		
4. Reaction rims on a volcanic aggregate particle (left view) and gneissic particle (right view). Both views are at a magnification of 10x, with a field of view of 8.7 mm long.		

5. Crack in the paste extends from coarse aggregate particle through paste and exits at the surface. Mag. 50x, fov = 3 mm long.	
6. Plane-polarized light view showing large crack in the paste; surface of core is at top. Mag. 50x, fov 3 mm. 7. Cross-polarized light view showing discoloured (probably carbonated) patch of paste, with fine cracks. Surface of core is at top. Magnification is 50x, fov 3 mm.	 
8a. Crack in carbonated zone alongside a granitic aggregate particle. 8b. Very fine crack (arrow) alongside a volcanic porphyry (VP) aggregate. Both images are at magnifications of 50s, with a field of view that is 3 mm in length.	 

9a, b. Views seen in plane-polarized light (upper) and cross-polarized light (lower) showing dense paste that encloses aggregate particles. Both views mag. 50x, fov = 3 mm.	
	
SUMMARY	The outer ~15 mm of the core has characteristics that are consistent with fire damage. Additionally, and possibly due to the presence of cracks in the fire-damaged zone, carbonation follows a few cracks into the concrete. ASR-related indications are present but are minimal in nature, persistence and level of damage.

Petrographer: ______________________________ DATE: September 17, 2020

F. Shrimer, P. Geo.

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE

ASTM C856-18

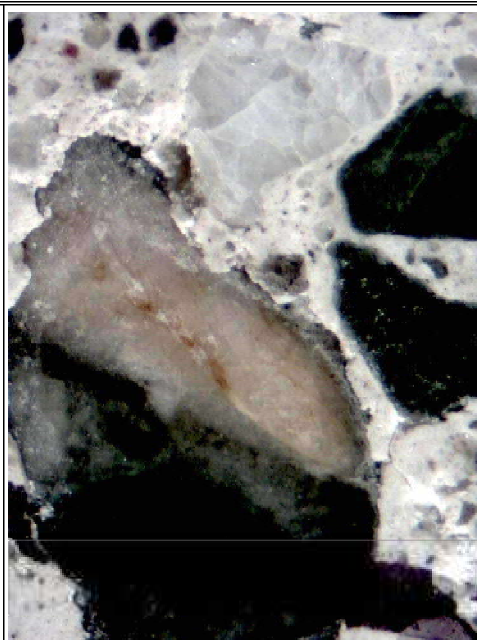
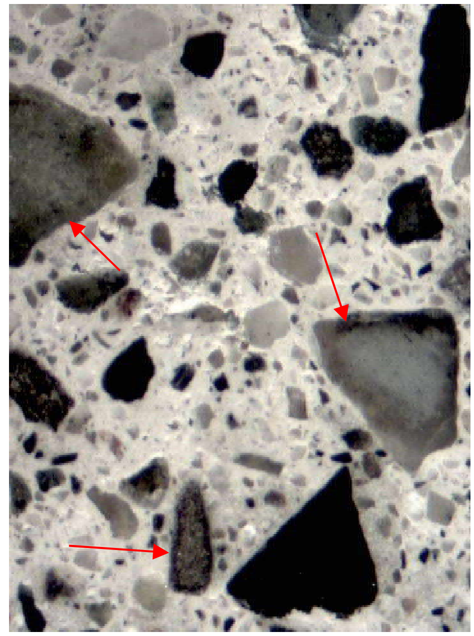
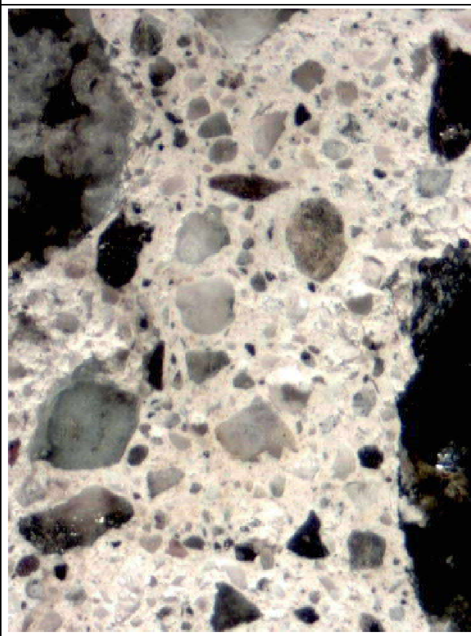
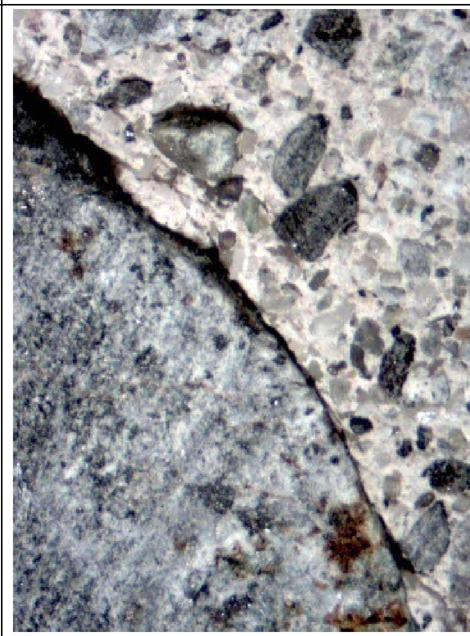
COWI
138 13th Street East
North Vancouver, BC V7L 0E5
Attention: Mr. Brad Pease, Ph.D.

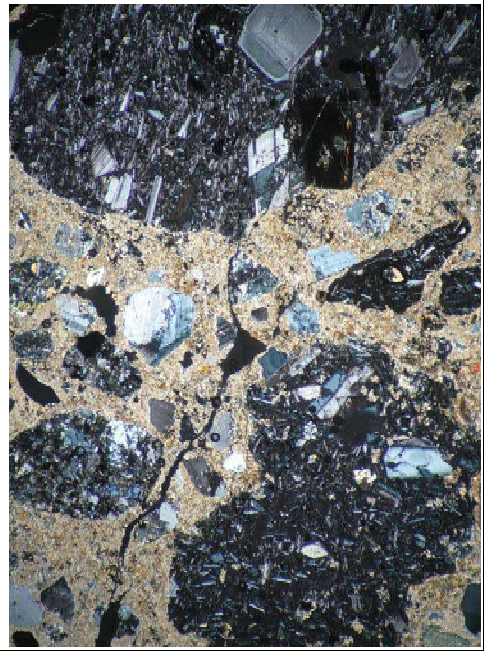
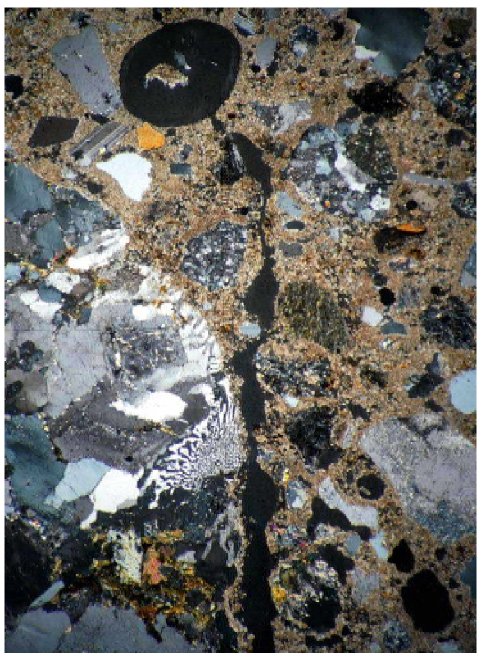
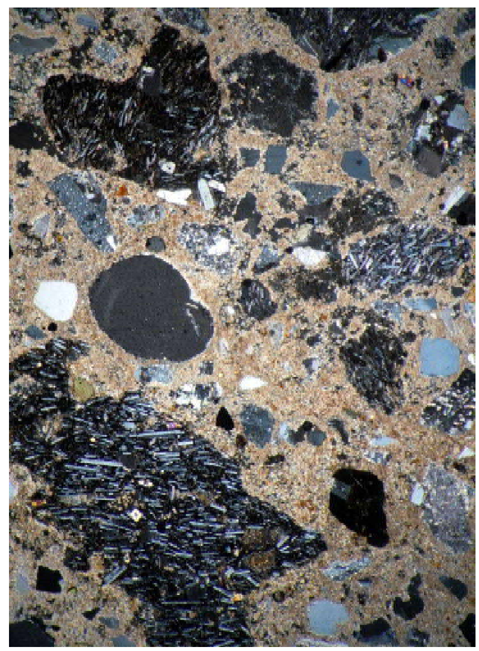
Project number: 20149024
September 16, 2020

PROJECT:	**Massey Tunnel**		
Sample:	Core P6	**Location:**	

SAMPLE TYPE – GENERAL	94 mm diameter drilled core. Approximately 42 – 60 mm length. Outer surface finished, lower surface uneven fracture. No reinforcing steel.
Aggregate maximum size	19 mm
Aggregate grading	Well-graded coarse and fine aggregates
Concrete consolidation	Concrete is generally dense and well-consolidated. Non air-entrained.
Cement paste	Paste is generally firm with localized patches of softer cement, and is carbonated and light buff in outer ~20-25 mm and light grey in colour below the carbonated zone. Cement is well-hydrated. Non-air-entrained.
Coarse Aggregate	Multi-lithic natural gravel, with particle shapes that range from subangular/irregular to well-rounded. Rock lithologies are dominated by granitic and volcanic rocks with lesser amounts of gneiss, quartzite, schist, quartz sandstone, chert and siltstone. Most igneous rocks exhibit variable alteration.
Fine Aggregate	Fine aggregate is a natural sand composed of lithic fragments of granite, a variety of volcanic rocks, quartzite, sandstone, chert, gneiss, quartz, feldspar, epidote, iron oxides and mica.
Description	The concrete is well consolidated and generally exhibits good contact between paste and aggregate. However, some localized patches of softer paste are evident; these correspond to some instances of debonding of paste from aggregate. In thin-section, fine cracks are observed passing through paste and through/from some aggregate particles. ASR gel is observed in some cracks, and surrounding aggregate, or in cracks within aggregate. Volcanic rocks and siliceous rocks are the most commonly observed rock types associated with ASR features.
Defects	Carbonated paste is found in the outer 20-25 mm of the core. Alkali-silica reaction (ASR) features include reaction rims on fine and some coarse aggregates (often, volcanic), occasional cracks extending from and through aggregate particles, and rarely through paste. Some cracks are devoid of ASR gel. A few debonded aggregates are observed, in some cases associated with ASR gel-lined sockets.

Photos	
1.Views of the sample prior to sawcutting and polishing.	 
2. Cut and polished sample.	

3a. Debonded aggregate particle: soft and porous paste encloses the aggregate. Magn. 10x, field of view is 8.7 mm. 3b. Reaction rims on siliceous aggregates (chert, volcanic rock). Mag. 10x, fov = 8.7 mm.		
4a. Soft paste in the sample is absorbent and easily disturbed. Mag. 10x, fov = 8.7 mm. 4b. Debonded aggregate particle. Mag. 10x, fov – 8.7 mm.		

5a. Carbonated paste appears as brown in this cross-polarized light view of a thin-section. Mag. 50x, fov = 3 mm long. 5b. A crack extends through a volcanic aggregate particle (upper portion of view) and passes through paste in the lower portion of the view. Mag. 50x, fov = 3 mm long.		
6. Cross-polarized light view showing large ASR gel-filled crack in the paste. Mag. 50x, fov 3 mm. 7. Cross-polarized light view showing several volcanic aggregate particles enclosed by paste. Magnification is 50x, fov 3 mm.		
SUMMARY	The concrete contains a moderate number of open and fine cracks, some lined/filled with ASR gel, some devoid of fillings. Reaction rims are common on some volcanic particles. ASR gel fills some voids and cracks. Outer ~20-25 mm of paste is carbonated.	

Petrographer: ______________________________ DATE: September 16, 2020

F. Shrimer, P. Geo.

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE

ASTM C856-18

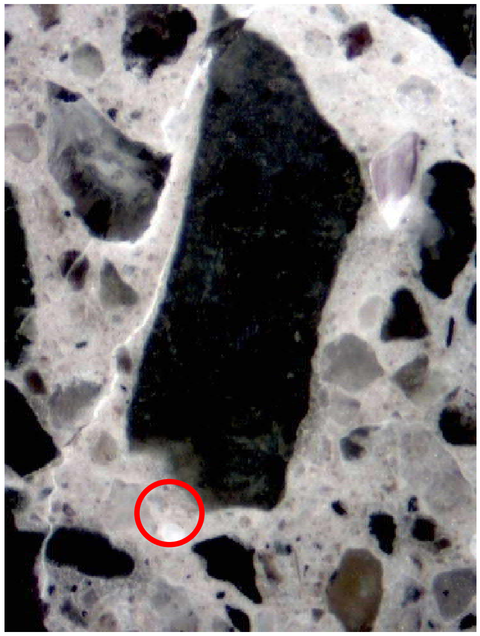
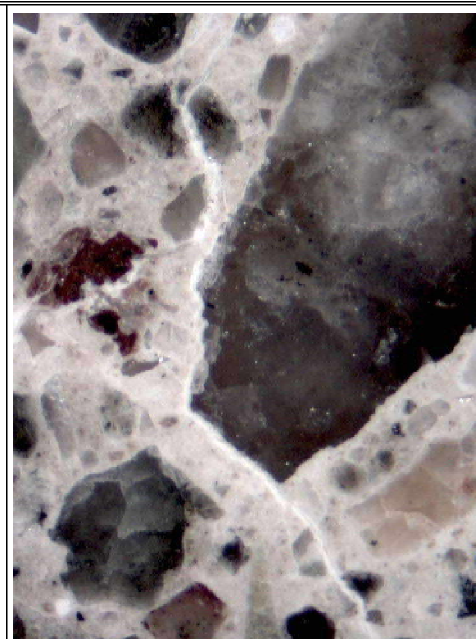
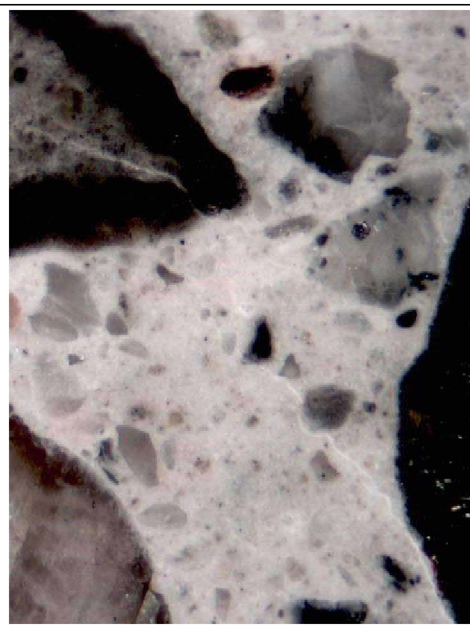
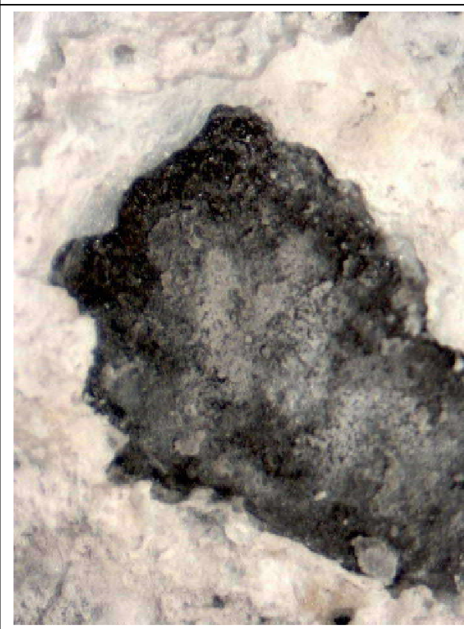
COWI
138 13th Street East
North Vancouver, BC V7L 0E5
Attention: Mr. Brad Pease, Ph.D.

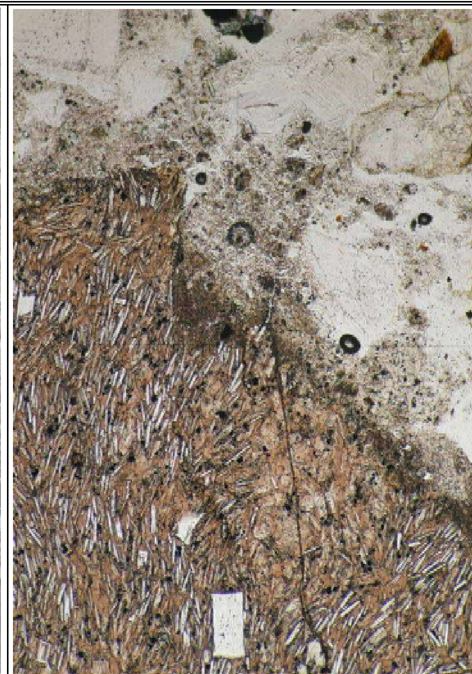
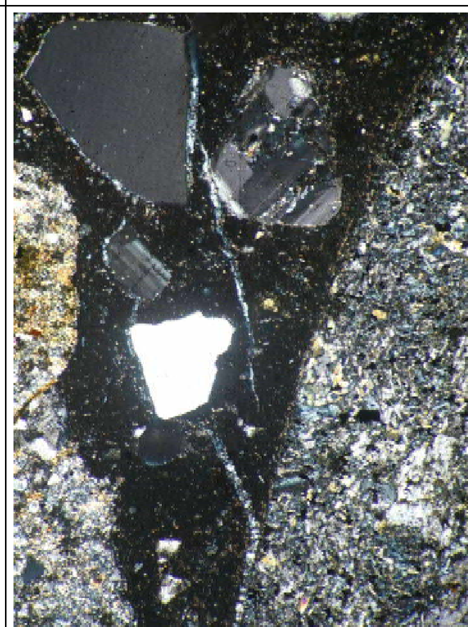
Project number: 20149024
September 10, 2020

PROJECT:	**Massey Tunnel**		
Sample:	Core P7	**Location:**	

SAMPLE TYPE – GENERAL	94 mm diameter drilled core. Approximately 30 cm length. Reinforcing steel at 21 cm depth in sample, 13 mm diameter, uncorroded.
Aggregate maximum size	32 mm
Aggregate grading	Well-graded coarse and fine aggregates
Concrete consolidation	Concrete is generally dense and well-consolidated. Non air-entrained. Open cracks extend subparallel to axis.
Cement paste	Paste is generally firm to hard, and is light grey/cream in colour. Cement is well-hydrated. Non-air-entrained.
Coarse Aggregate	Multi-lithic natural gravel, with particle shapes that range from subangular/irregular to well-rounded. Rock lithologies are dominated by granitic and volcanic rocks with lesser amounts of gneiss, quartzite, schist, quartz sandstone, chert and siltstone.
Fine Aggregate	Fine aggregate is a natural sand composed of lithic fragments of granite, a variety of volcanic rocks, quartzite, sandstone, chert, gneiss, quartz and feldspar. Occasional mollusc shell fragments were observed in the fine aggregate fraction.
Description	The concrete is well consolidated and generally exhibits good contact between paste and aggregate. In thin-section, fine cracks are observed passing through paste and through/from some aggregate particles. ASR gel is observed in some cracks, and surrounding aggregate, or in cracks within aggregate. Volcanic rocks and siliceous rocks are the most commonly observed rock types associated with ASR features.
Defects	Fine hairline and larger open cracks extend through the concrete in a variety of orientations - the largest open crack is oriented roughly subvertical. Features associated with alkali-silica reaction (ASR) include reaction rims on fine and some coarse aggregates (often, volcanic), cracks extending from and through aggregate particles, through paste. Some cracks are devoid of ASR gel while others are partially or fully filled with ASR gel. Debonded aggregates are observed, some with ASR-gel-filled surrounds. ASR gel is observed in some voids, as partial or full fillings.

Photos	
1. View of the sample prior to sawcutting and polishing.	
2. Cut and polished sample. The DRI was conducted on the portion with the 1 cm grid.	

3a. ASR-gel-filled cracks passing through paste along a volcanic aggregate particle. Note also a gel-filled air void located below the large aggregate particle. Magn. 10x, field of view is 8.7 mm. 3b. ASR-gel-filled cracks pass through paste in various orientations. Mag. 15x, fov = 6.3 mm.		
4a. Reaction rim on volcanic particle with ASR-gel-filled internal crack that extends out into paste (arrows). Mag. 20x, fov = 5.0 mm. 4b. Reaction rim and crusty dried ASR gel on reacted volcanic aggregate particle. Mag. 10x, fov – 8.7 mm.		

5a. A relatively large (≤1 mm wide) open crack is seen in thin-section, lined with old ASR gel and carbonate material, passing through a volcanic aggregate particle, through which a number of finer cracks pass, these filled with ASR gel. Mag. 50x, fov = 3 mm long. 5b. A volcanic aggregate particle dominates the lower left of this thin-section view, with a darkened reaction rim, and an internal crack that extends into the paste. Mag. 100x, fov = 1.5 mm long.		
6. Views of ASR-gel-filled cracks seen in thin-section (left is plane polarized light, right in cross-polarized light). Left mag. 50x, fov 3 mm; right is 100x, fov 1.5 mm.		
SUMMARY	The concrete contains a variety of open and fine cracks, some lined/filled with ASR gel, some devoid of fillings. Reaction rims are common on some particles. ASR gel fills some voids.	

Petrographer: ____________________________ DATE: September 10, 2020

F. Shrimer, P. Geo.

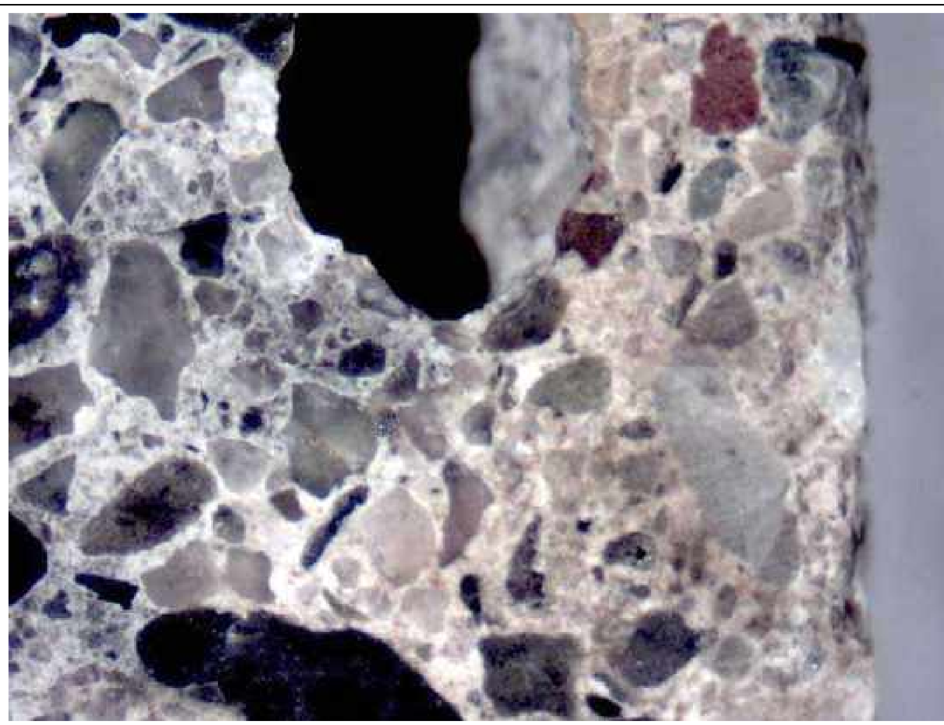
PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE
ASTM C856-18

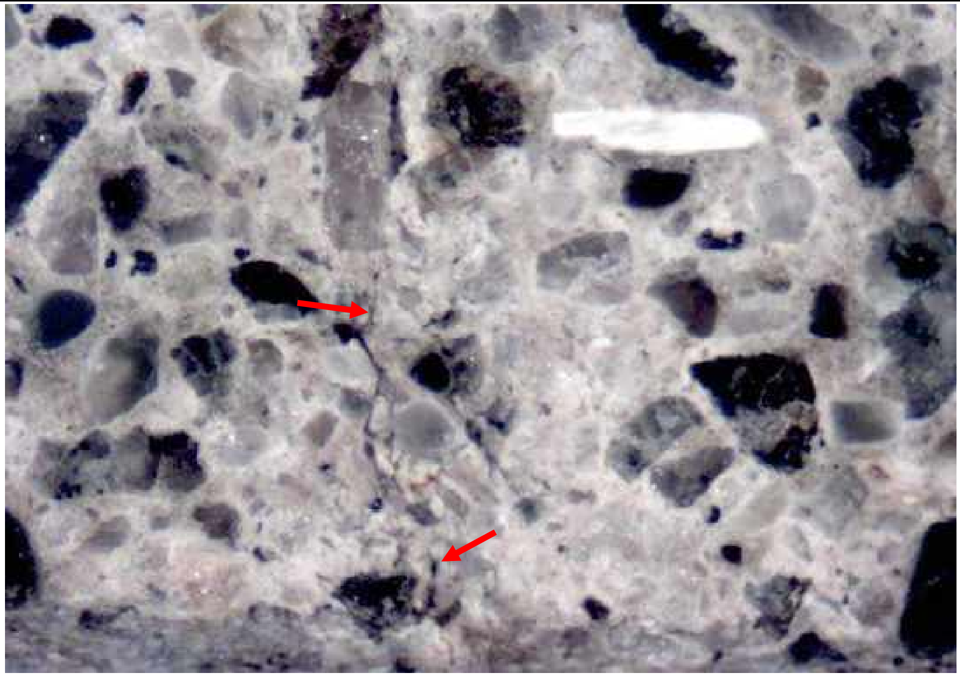
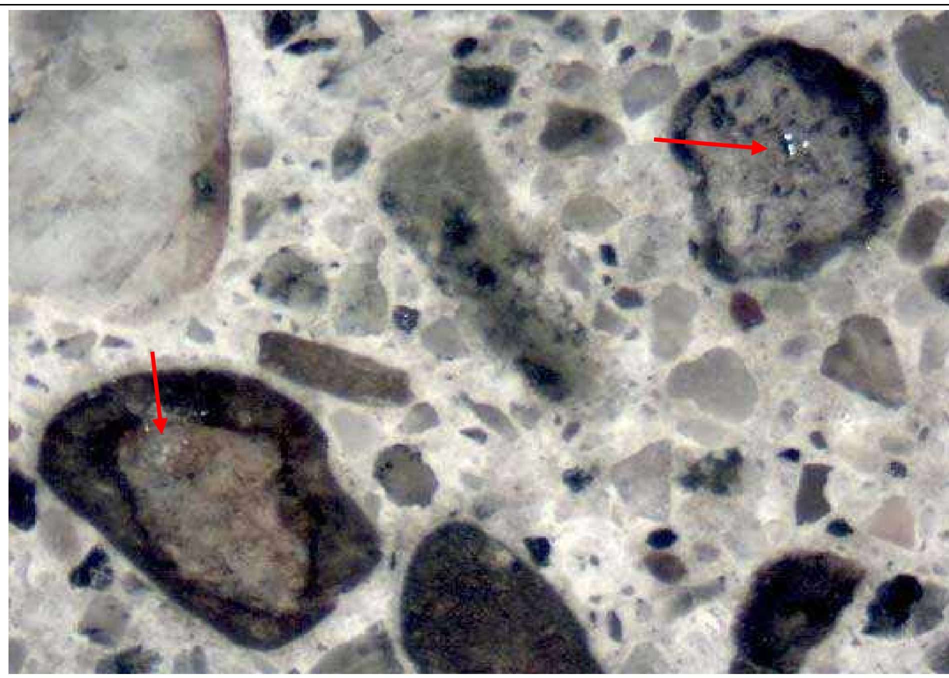
COWI
138 13th Street East
North Vancouver, BC V7L 0E5
Attention: Mr. Brad Pease, Ph.D.

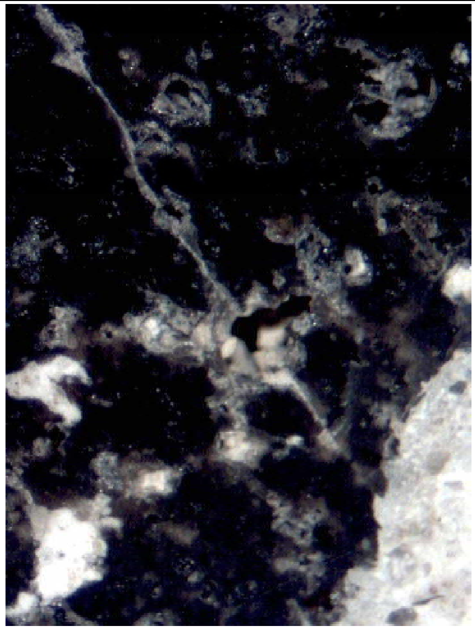
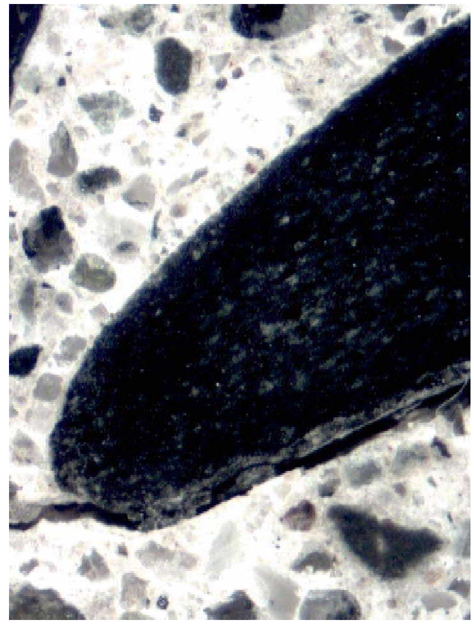
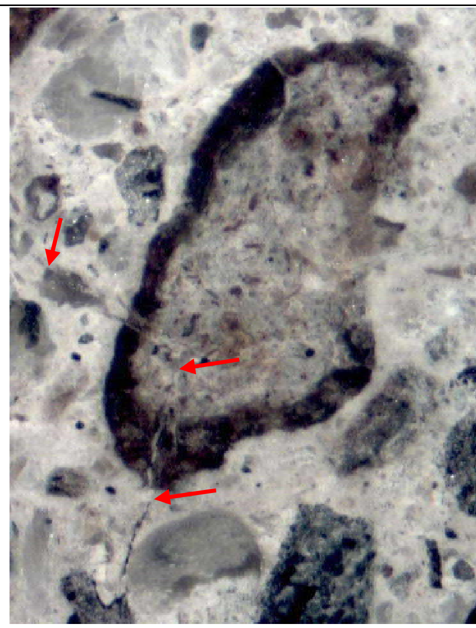
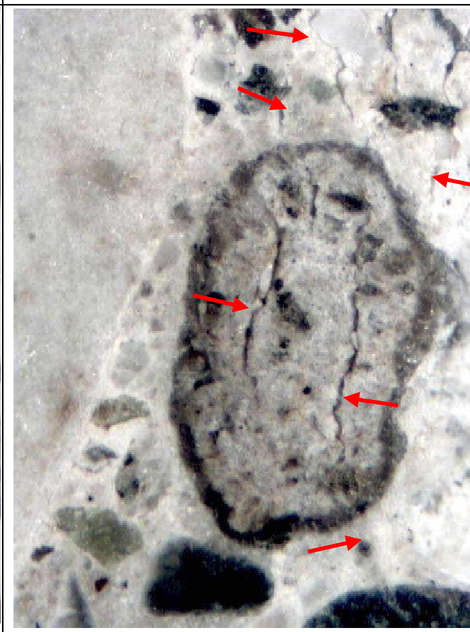
Project number: 20149024
September 24, 2020

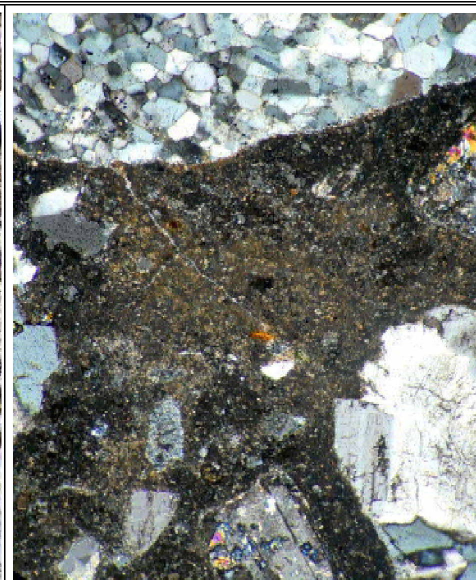
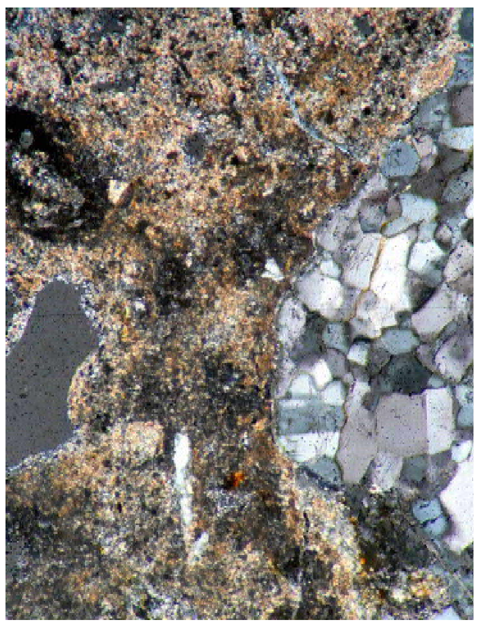
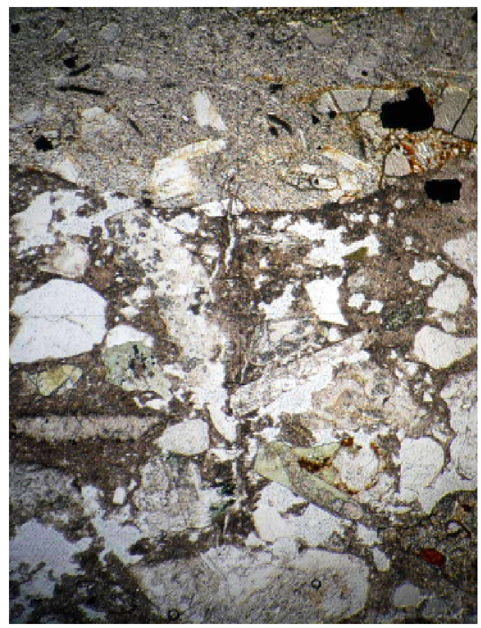
PROJECT:	**Massey Tunnel**		
Sample:	Core C8	**Location:**	

SAMPLE TYPE – GENERAL	A portion of a 94 mm diameter drilled core, approximately 90 mm length. No reinforcing steel observed in sample.
Aggregate maximum size	35 mm
Aggregate grading	Well-graded coarse and fine aggregates
Concrete consolidation	Concrete is generally dense and well-consolidated. Non air-entrained. One crack observed, parallel to surface, about 55 – 70 mm below outer surface.
Cement paste	Paste is generally firm to hard, and is light grey/cream in colour. Cement is well-hydrated. Non-air-entrained. Outer 5 to 15 mm of paste is carbonated.
Coarse Aggregate	Multi-lithic natural gravel, with particle shapes that range from subangular/irregular to well-rounded. Rock lithologies are dominated by plutonic (e.g., granite, diorite) and volcanic rocks, with lesser amounts of gneiss, quartzite, quartz sandstone, chert and siltstone.
Fine Aggregate	Fine aggregate is a natural sand composed of lithic fragments of granitic rock, volcanic rocks, gneiss, schist, quartzite, sandstone, chert, quartz and feldspar. Occasional mollusc shell fragments were observed in the fine aggregate fraction.
Description	The concrete is well consolidated and generally exhibits good contact between paste and aggregate. A few cracks are observed passing through the paste and around and into aggregates. Reaction rims are present on some coarse aggregate particles and numerous volcanic fine aggregate particles, in some cases associated with cracking within aggregate and/or in the paste. Overall, the size, extent and frequency of cracking is modest.
Defects	Carbonation of paste was observed in the outer 5-15 mm of the concrete, although quite concentrated along crack traces. Indications of ASR in the concrete were observed throughout the core, including reaction rims on aggregates; ASR gel located in cracks in the paste, in voids, in aggregate particles. A few debonded aggregates were observed, and many of the volcanic fine aggregate particles that had reaction rims were partially corroded. After cutting and polishing, fresh gel was observed on certain aggregate particles, dominantly volcanic fine aggregate particles.

Photos	
1. View of the sample after sawcutting and polishing.	
2. Detail of polished sample illustrating carbonated paste extending into core from the surface to a depth of about 6 mm. Magnification 10x, field of view is 8.7 mm across.	

3. A crack is shown extending horizontally through the paste at a depth of about 5.8 cm below the core's surface. Magnification of 10x, field of view 8.7 mm long.	
4. Two felsic volcanic rocks with prominent reaction rims and fresh ASR gel (arrows) exuded since cut-polish preparation (i.e., about four days' time). Magnification 10x, FOV is 8.7 mm length.	

5a. An ASR-gel-filled crack is observed passing through a volcanic aggregate particle. 5b. Debonded aggregate. Both views are a t a magnification of 10x, FOV is about 8.7 mm long.		
6a, b. Two volcanic aggregate particles exhibiting pronounced reaction rims with ASR-gel-filled cracks passing through and extending into the surrounding paste. Left view magnification 15x, fov 6.3 mm and right view 20x, fov 5.0 mm in length.		

7a, b. Thin-section views of cracks in the paste that extend from reacting aggregate particles. Left view, seen in plane polarized light, shows an ASR gel-lined crack that extends from a volcanic aggregate, while the right view, seen in cross-polarized light, shows a crack extending from an aggregate particle that contains strained quartz. Both views 100x magnification with a field of view (lengthwise) of 1.5 mm.		
8a. Cross-polarized view of thin-section showing ASR gel-filled crack passing from siliceous aggregate; 100x mag. FOV = 1.5 mm length. 8b. Plane-polarized light view of thin-section showing ASR gel-filled crack issuing from volcanic particle at top of view. 50x mag. FOV 3 mm.		
SUMMARY	The concrete is well-consolidated with adequate proportioning and distribution of aggregates in the paste matrix. Paste interface is generally good, except in localized zones where debonding of aggregate has occurred.. The outer 5-15 mm of the concrete is characterized by carbonated paste. Signs of Alkali-Silica Reaction (ASR) are observed throughout the sample and include(a) reaction rims on some aggregates, (b) presence of ASR gel in cracks (c) cracks in aggregate and in paste (d) localized debonding of aggregate from paste.	

Petrographer: F. Shrimer, P. Geo. DATE: September 24, 2020

PROFESSIONAL PROVINCE OF F. H. SHRIMER BRITISH COLUMBIA GEOSCIENTIST

C.3 Aggregate Provenance Evaluation

GOLDER

TECHNICAL MEMORANDUM

DATE 4 December 2020 **Reference No.** 20149024-001-TM-Rev0

TO Brad Pease, PhD
COWI North America Ltd.

FROM Fred Shrimer **EMAIL fshrimer@golder.com**

AGGREGATE PROVENANCE EVALUATION, MASSEY TUNNEL CONCRETE

1.0 INTRODUCTION

As requested by COWI North America, Ltd. (COWI), Golder Associates Ltd. (Golder) has undertaken an evaluation of the provenance of concrete aggregates used in the manufacture of concrete for construction of the Massey Tunnel on Highway 99 between Delta and Richmond, BC.

This work is further to previous petrographic examinations, conducted by Golder, of eight concrete cores that were understood to have been obtained from various structural elements of the tunnel under the direction of COWI. The Petrographic results were provided to COWI in reports dated September 10 to 24, 2020.

1.1 Background

As noted above, earlier examinations were carried out on eight concrete core samples provided by COWI. A general indication of aggregate characteristics was included in that work. There were indications that differing aggregates had been used in the manufacture of some of the concrete, and the use of different aggregate sources was considered relevant to the purpose of understanding the relative severity and continued potential for Alkali-Silica Reaction (ASR), also previously identified in some of the Massey Tunnel concrete.

COWI has also provided a written report that contains some additional information concerning concrete properties, including reported aggregate sources. That report was used as a background context to inform this evaluation, but not to guide this work.

1.2 Scope

As described in Golder's Change Order dated September 29, 2020, the scope of the current evaluation was to examine the eight concrete cores with regard to aggregate lithology and to provide a discussion of possible aggregate sources. The focus was to determine whether different aggregate sources were used for the manufacture of the concrete represented by the eight cores.

2.0 RESULTS

2.1 General Comments

Petrographic information concerning the lithology of the coarse and the fine aggregate fractions must take into account a number of considerations, as follows:

- In order to discern trends or 'markers' that are relevant for lithological identification of a source, the process must allow for a sufficient level of detail, for example, to a minimum of 1% or less. This requires identification of at least 100 particles. In a typical 3" diameter core containing aggregate of nominal size 19 mm, sufficient coarse aggregate may not be available to render such a count.
- Distinguishing between coarse aggregate, where only a small portion of the aggregate is visible, and fine aggregate may not be conclusive in some cases, because of uncertainty inherent in making that distinction. Owing to this, some coarse aggregate particles may be identified as fine aggregate grains, thereby introducing error in both the coarse and fine aggregate lithological identifications.
- Because of the two-dimensional aspect and small size of the coarse aggregate fragments, identifications of the lithologies are considered to be tentative only.
- At very small sizes, aggregate particles become increasingly difficult to identify. This may result in the fine aggregate being under-represented in its finest grain size ranges.

Owing to these factors, it should be assumed that the lithological proportions are estimates only.

2.2 Methodology

The approach that was used for the examination of lithologies of the aggregates included the following:

- Examination of the polished samples from the previous petrographic work;
- Preparation, where possible, of additional polished slabs; and
- Checks of identification using some thin-section mounts.

Rock or mineral particles were identified and counted in order to calculate a proportion/percentage for each rock or mineral type.

Where matched-face samples had been prepared, only one face was examined, so as to avoid double-counting particles.

Some faces could not be satisfactorily prepared, owing to paste condition, cracking, etc.

2.3 Coarse Aggregate Lithologies

The following table provides the lithological data for the coarse aggregates:

Coarse Aggregate Lithologic Type	Percent per Core							
	Precast elements			Approach ramps		Infill joints		
	P2	P4	P5	P7	C8	C1	P3	P6
Volcanic – mafic	11.2	11.9	6.3	14.2	13.8	18.3	10.6	8.1
*Volcanic – felsic	15.0	18.7	12.6	16.2	12.3	12.9	11.5	21.6
*Volcanic – pumiceous/glassy		1.1	1.1	4.0	4.6	1.8	2.5	2.7
*Fine-grained Granite-Diorite	2.8	4.0	4.2	2.7	4.6	0.9	4.1	2.7
*Granitic – grey/white/black	32.7	34.5	46.3	33.8	30.8	45.9	42.6	36.5
Granitic – grey/white/pink	2.8	2.8	1.1	2.0	10.8	3.7	0.8	2.7
Diorite – black/grey/white	14.0	5.1	6.3	7.4	10.8	11.0	8.2	6.7
*Gneiss	4.7	2.8	5.2	0.7	1.5	2.8	3.3	2.7
*Quartzite	7.5	5.6	9.5	6.1	4.6	1.8	3.3	13.5
*Quartz Sandstone	1.9	3.4	1.1				0.8	1.4
*Arkose-lithic sandstone-greywacke	0.9	2.8	4.2	0.7	3.1		3.3	1.4
*Chert	6.5	7.3	2.1	12.2	3.1	0.9	9.0	
TOTALS	**100.0**	**100.0**	**100.0**	**100.0**	**100.0**	**100.0**	**100.0**	**100.0**

* denotes rock types that are associated with Alkali-Aggregate Reaction

The following observations are made regarding the coarse aggregate lithology data:

- The coarse aggregates are dominated by igneous rock types, and of those, plutonic rock types such as granite and diorite are the most commonly observed. Typically, plutonic rock types account for approximately 42% to just over 60%.
- Volcanic rocks make up between 20% and 34% of the coarse aggregate. Of that amount, pumiceous or "frothy" volcanic rocks accounted for 0 to 4.6% of the aggregate, with seven of the eight cores containing some of this rock type.

- Gneiss makes up from <1% to just over 5% of the coarse aggregate.
- Quartzite ranges from 1.8% to 13.5% of the coarse aggregate fraction.
- Varieties of sandstone, including quartz-rich arenite, arkose, lithic sandstone and greywacke, ranges from 0 to 6.2%, while chert content ranges from 0 to 12.2% of the coarse aggregate fraction.
- Cores P2, P3, P4 and P7 are indicated to contain significantly more chert in the coarse aggregate than other cores, where it is somewhat uncommon (i.e., C1, P6).
- Core C1 appears to contain a low quartzite content, and low chert, relative to the other cores.
- Most of the cores contain the pumiceous volcanic rock type, with the exception of P2, where none was detected. Cores P4 and P5 each contained small amounts, but P7 and C8 contained 4.0 and 4.6% each.

2.4 Fine Aggregate Lithologies

As with the coarse aggregate fractions, the fine aggregate lithologies also varied from core to core:

Lithologic Type	Percent by Core							
	Precast elements			Approach ramps		Infill joints		
	P2	P4	P5	P7	C8	P2	P3	P6
Volcanic – mafic	10.4	3.3	4.6	2.5	3.3	10.4	6.2	5.9
*Volcanic – felsic	4.0	3.3	2.3	3.8	2.0	4.0	4.9	2.7
*Volcanic – pumiceous/glassy	0.8		0.6	1.3	4.0	0.8	4.9	3.2
*Granite – light coloured	16.8	7.2	11.4	20.6	21.3	16.8	13.9	18.9
Diorite-plutonic – dark coloured	3.2	1.7	4.0	3.1	4.7	3.2	3.5	3.2
*Quartzite/Sandstone	4.8	5.0	3.4	3.8	1.3	4.8	4.2	0.6
*Chert	3.2	2.2	3.4		2.0	3.2	4.2	
Quartz	41.6	63.9	54.9	49.3	50.0	41.6	43.0	53.0
Feldspar	4.8	5.6	7.4	4.4	6.7	4.8	6.9	7.6
Mica	7.2	6.1	6.3	5.0	2.7	7.2	6.9	3.8
Epidote	1.6	1.7	1.1	3.1	1.3	1.6	1.4	1.1
Hematite	0.8			3.1		0.8		
Shell					0.7			
Wood	0.8		0.6			0.8		
TOTALS	**100.0**	**100.0**	**100.0**	**100.0**	**100.0**	**100.0**	**100.0**	**100.0**

* denotes rock types that are associated with Alkali-Aggregate Reaction

The following observations are made regarding the fine aggregate lithologies:

- Plutonic rock lithic fragments ranged from less than 10% to over 25%.
- Volcanic rock fragments accounted for between 7.5 and 16.0%; pumiceous volcanic fragments made up 0 to 4.9% of that amount.
- Quartzite and sandstone were grouped together in the fine aggregate classification due to the difficulties associated with distinguishing these lithologies in small-sized grains. They accounted for 0.6 to 5.0% of the fine aggregate.
- Chert ranged from 0 to 4.2% of the fine aggregate.
- Quartz grains made up between 41.6% and 63.9% of the fine aggregate; feldspar grains accounted for between 4.4% and 9.5% of the fine aggregate.
- Grains of mica (mostly biotite) ranged from 2.7% to 7.2% of the fine aggregate.
- Epidote ranged from 1.1% and 3.1% of the fine aggregates in the cores.
- Other fine aggregate constituents included hematite/magnetite, wood and shell fragments.

3.0 AGGREGATE SOURCES IN THE 1950s

3.1 General

Information on historic gravel sources that supplied aggregates to the Vancouver region may be sourced through review of archived materials, government records (e.g., primarily provincial and municipal), newspapers and industry journals, and industry sources. In general terms, the aggregate sources that were in use during the 1950s were smaller than those in use today, and tended to be located much closer to the Vancouver-area market, sometimes even within the market area.

It may also be noted that, although there were some bedrock quarries (e.g., Gilleys Quarry, Coquitlam) that were utilized for production of aggregates, the concrete industry in British Columbia has historically made use of natural (i.e., fluvial/glaciofluvial) sand and gravel deposits for supply of concrete aggregates, until the last few decades. In the 1950s, it would be expected that concrete aggregates would have been manufactured from such sources.

The number of pits was greater in the past, and transportation of materials was done by two primary means: barge and truck. Many smaller pits were operated on a short-term, ad hoc basis, and arrangements were more ‘casual’ by contrast with the operating and regulatory regime that typifies today’s aggregate operations.

Nonetheless, a number of larger, established aggregate operations were supplying aggregate products to the Vancouver market. These include:

- Deeks-McBride pit in North Vancouver
- Gilleys Bros pit at Mary Hill, Port Coquitlam
- Lafarge pit, Friday Harbor, Washington

- Various pits in the Coquitlam River valley – Cewe, Allard, S&S, Lafarge
- Various pits in Maple Ridge in the vicinity of 240th Street – Carr Sand & Gravel, Allard, H & R
- Britannia Pit, Hwy 99 south of Britannia Mine
- Various pits in south Surrey and Langley
- Construction Aggregates pit, north of Langdale

3.1.1 Geology of Aggregate Sources

Golder has not completed a detailed compilation of lithologies for any of these pits as part of this study; however, based on extensive examination of aggregate samples from numerous British Columbia aggregate pits and review of published and unpublished geological reports dealing with quaternary and bedrock geology of the South Coast, a generalized perspective of the dominant geological composition of lower mainland aggregate sites can be provided. Based on these inputs, the following general trends and observations can be provided:

- It is likely that most of the gravel pits noted above would contain some amount of Coast Mountain granitic rock. The more distant the pit from the "North Shore mountains", the lower the amount of these rocks.
- Most of the gravel pits cited above would contain some proportion of volcanic rocks. This would have been the case in the pits that were sourced for aggregates in the 1950s, much as it is for currently producing pits.
  - The pumiceous volcanic rocks remain somewhat of an enigma, since the most apparent geological source for such rock is likely to be recent volcanic deposits in the Whistler region.
- Quartzite and sandstone are not too common in 'North Shore' gravel deposits, and are more likely to have an eastern provenance. This means that such rock types would be more likely to be found in deposits that have some amount of source rocks to the east, in the Cascade/Skagit ranges, or further upstream for Fraser River sources.
- Chert is generally uncommon in North Shore gravels. Well-rounded, striped and banded cherts are more commonly associated with Fraser valley gravel deposits. Some of the chert currently found in Fraser River deposits may have ultimate provenance in the Rocky Mountains, although there are other chert deposits along the course of the Fraser.
- Gneissic rocks, or similar rocks that exhibit some foliated or aligned structure, are found within Fraser River-sourced deposits and in North Shore mountain-sourced deposits.
- Shell fragments and woody pieces are not specifically indicative of various sources, since virtually all gravel deposits contain organic material such as wood, and shell fragments have been observed in gravel deposits throughout the South Coast region.

The primary geological data that may provide some indication of sediment source direction and thereby source deposit may be as follows:

- Absence or presence and proportion of quartzite, sandstone and chert;
- Proportion of pumiceous volcanic rocks;
- Degree of roundness and sphericity of the aggregate particles; and
- Quartz content may provide a sense of travel distance; thus, lower quartz content could correspond with a North Shore source, while higher quartz content could be considered to be consistent with a Fraser valley source.

Information provided in a report completed by Christiani and Nielsen (1959) and forwarded by COWI indicated that the following sources of concrete aggregate were used for various concrete types at the Massey Tunnel project in the latter 1950s:

CONCRETE TYPE	REPORTED SOURCE	CORES
Precast Tunnel concrete	Mary Hill Pit	P2, P4, P5
Approach Ramps	Deeks-McBride, Seymour River	P7, C8
Infill Joints, inner/outer walls	Unknown	C1, P3, P6

Although these aggregate sources are provided in the Christiani and Nielsen report, there is no way to verify that there were no changes, deviations, or substitutions of aggregate throughout the length of the project. Furthermore, the sources of aggregate used for the infill joint and wall concretes are unknown.

A series of tests of the aggregates were reported in the Christiani and Nielsen excerpt, and they generally indicated that the materials tested would be classified as ‘durable’ and ‘strong’ aggregates; the data are presented below.

PARAMETER	SEYMOUR PIT		MARY HILL PIT	
	COARSE	FINE	COARSE	FINE
Soundness (%)	0.55		0.46	0.72
Soft particles (%)	nil		not performed	
Surface coatings	nil		not performed	
Reduction in alkalinity	25.00		48.80	
SiO_2 concentration	1.43		4.33	
Los Angeles abrasion (%)	18	n/a	not performed	n/a
Deval Abrasion (%)	not performed		2.1	not performed
Specific Gravity	2.71	2.69	2.69	2.68
Absorption (%)	0.62	0.72	0.5	0.6

Petrographic examination was not reported in the Christiani and Nielsen on the gravel or the sand aggregates; thus, no information was available regarding the geologic composition of these aggregates.

Only the Specific Gravity values and the chemical data generated (in ASTM C289) can be linked, albeit indirectly, to aggregate geology.

The values for Specific Gravity are fairly similar between the two sites, although slightly higher for the Seymour material. While this slight offset in SG is not 'definitive', it generally aligns with the perspective that Fraser valley-sourced aggregates may have slightly lower SG values. This reflects an increase in siliceous rocks such as chert, quartzite and sandstone, which typically have a lower SG, and a decrease in mafic volcanic rocks, which have higher SGs.

The silica concentration data from C289 may be of some assistance in linking aggregate geology, but this is also thought to be indirect and tenuous at best.

Overall, these data are not considered sufficient on their own to provide clear indications regarding the geological composition of either aggregate source.

4.0 SOURCES OF POTENTIAL UNCERTAINTY FROM CORE EXAMINATIONS

A number of factors should be considered before relying on the foregoing observations and discussion. Some of these have been noted previously, and are included below:

- The concrete cores provide a relatively small sample size for such an analysis, particularly in terms of the coarse aggregate material. Where sawcut slabs are relatively thin, some aggregate particles could be counted twice, skewing the results;
- Where the concrete quality is not high, sample preparation efforts may not be sufficient to prepare good-quality surfaces suitable to the examination. When the surface is of inadequate quality, identification of the rock and mineral types becomes less precise, and some margin of error must be accommodated;
- Distinguishing between fine and coarse aggregate particles is sometimes difficult due to uncertainty regarding the third dimensional view;
- The accuracy of the percentages of various rock types identified in the concrete cores is reduced because of the two-dimensional aspect of the particle;
- Chemical reaction of some particles that are consumed to a large degree reduces the ability to identify them; and
- Grain size-specific mineral species identification becomes complicated.

These factors can reduce both the precision and the certainty of identifications of the aggregate lithologies in the concrete samples.

5.0 DISCUSSION AND RECOMMENDATIONS

Following on the evaluations, examinations of the concrete, the review of historic records, geological reports, and consultation with industry representatives and researchers, the following conclusions are provided.

1) Different sources of concrete aggregate were used for the manufacture of the concrete represented by the eight cores;

2) There is no clear, definitive association evident between the aggregates identified in the concrete cores and specific aggregate pits that were in operation in the 1950s. Certain geological components may serve as ‘indicators’ of aggregate sourcing, and might be considered as inferring or trending towards a source site, but based on our current understanding and the information that is available and has been generated to-date, it is not possible to provide definitive conclusions.

3) In general, our review and research on aggregate sources indicates that the Seymour Deeks-McBride and Gilleys Bros Mary Hill pits mentioned in the Christiani and Nielsen report are among the possible sources.

4) The ability to distinguish between multi-lithic coarse aggregates on the basis of a small number of small-size cores is dubious. This is because the number of individual rock types is many, and there is often a gradational composition between the rock types; furthermore, the size of the particles is at most about 20 mm, thus limiting the ability to distinguish individual rock types accurately. When having only a two-dimensional aspect to examine, the ability to provide concise identification is reduced, compared with having the actual (three-dimensional) aggregate particle in hand.

This could be improved if (a) additional cores were extracted for examination and/or (b) the current cores were disaggregated by dousing in HCl acid, thereby providing aggregates without attached paste for examination.

The following is provided for further consideration:

- The presence of the pumiceous volcanic rocks in both the fine (more common) and coarse (less common) aggregate fractions of many of the cores may be an ‘indicator” rock type, as is the case with quartzite, sandstone and chert. This material is thought to potentially be associated with recent volcanic rocks produced from the Mount Garibaldi (about 17,000 BP).
- The proportion of volcanic rocks and of granitic (plutonic) rocks may also be of some use in indicating a source or a general location of sources, that is, “North Shore pits” or “Fraser Valley pits”.
- Anecdotally, one industry representative whose roots in the aggregate supply business regionally extend into at least the 1940s indicated that aggregates sourced near Albion Hill (Maple Ridge) had been used in some of the concrete at the Deas Island tunnel. Such an aggregate source would have had a greater proportion of ‘eastern provenance’ lithologies.
- Anecdotally, COWI advised that some of the filler sand used in the concrete at the project was sourced from a pit in North Delta. This material was apparently required to blend with a coarser sand.
- In the 2000 Petrographic examination reports, preliminary comments were made suggesting that the Britannia Pit might have been a possible source for the aggregates. That site, while not in active production currently, is still largely undeveloped. Obtaining a sample of sand and gravel from that site to compare the aggregate petrography might prove useful.

- The sources of aggregate that were mined in the 1950s have, by and large, been exhausted and are closed. There is no direct opportunity to obtain samples of aggregate products and thereby to conduct petrographic examinations to serve as a basis for comparison of lithologies of the aggregate in the concrete with that taken from the pits. Nevertheless, obtaining gravel and sand from another part of the Seymour River fan is considered quite possible, and may render representative geologic data.

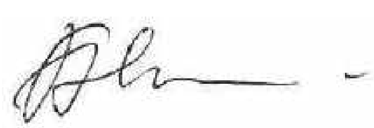
5.1 Recommendations

- For further consideration, the following items are advanced as potential methods of enlarging the understanding of the aggregate source question: Obtain larger and more core samples of the subject concretes to enable better characterization of the coarse aggregates in particular;
- Attempt to procure samples of gravel from the Seymour River fan-delta that could represent the material extracted and processed at the Deeks-McBride operation there. Subject those samples to detailed petrographic examination and other tests for comparison with the data obtained as reported in the Christiani and Nielsen report;
- Determine whether any natural sand-gravel materials are exposed near the former Mary Hill pit site, sample and test;
- Similarly, determine whether concrete of the same vintage and from the same pit sources might be identified in structures that could be core sampled and examined for comparison.

6.0 CLOSURE

We trust that this report provides the information required. Should you have questions regarding the contents of this report, please contact us.

Golder Associates Ltd.



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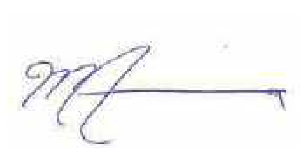

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