Measurement of the mechanical properties of concrete at the Mactaquac Generating Station to enable modeling of future ASR impacts and mitigation

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Abstract

The Mactaquac Generating Station was constructed in the 1960s using, as concrete aggregate, greywacke sourced on site from blast rock excavated for the construction of the dam and headpond. In the 1980s indications of alkali-silica reaction (ASR) were first observed. Since that time extensive mitigation measures have been carried out to manage the ASR.

The Station was designed for a 100-year service life with projected replacement or upgrading of the mechanical components mid-way through the Station's life; many of the mechanical components are now approaching the end of their service life. After extensive review and consideration of the options: decommission, reconstruction, or rehabilitation; the decision was made to keep the station in service until the end of its original design life. Finite element models have been developed to predict the future impacts of ASR on the concrete structures and identify areas of greatest concern; the models will be used to plan and predict the required ASR mitigation activities.

The models predict the future properties of the concrete using the present mechanical properties and the expected changes due to ASR. Thus, a thorough understanding of the present mechanical properties of the concrete is needed. An extensive program of concrete core extraction and testing was recently undertaken to measure the mechanical properties throughout the station with consideration to the different classes of concrete (cement contents, water-to-cement ratios, aggregate sizes) and environmental exposure conditions. These measured values will be incorporated into the models to develop a more accurate prediction of the future performance of the concrete and the necessary mitigation measures.

***Keywords**: ASR mitigation; finite element model; generating station; mechanical properties; service life*

1. INTRODUCTION

The Mactaquac Generating Station is a six-turbine run-of-the-river hydroelectric station with a generating capacity of approximately 672 MW. The Station was constructed in the 1960s and commissioned in 1968 with three turbines in operation. The other three turbines were brought online later; unit 4 was installed in 1972, units 5 and 6 were installed in 1979 and 1980 [1]. The Station is located along the Saint John River approximately 20 km upstream of Fredericton, New Brunswick, Canada and approximately 150 km from the discharge of the River into the Bay of Fundy.

The Station includes the main dam (rockfill earthen dam), diversion sluiceway, main spillway, intake and powerhouse structures. The diversion sluiceway and main spillway each include five gates. The intake structure and main spillway are located immediately adjacent each other in the approximately 200 m wide intake channel that was excavated from bedrock during construction of the Station. The diversion sluiceway and main dam are located south of the main spillway and intake structure. The intake structure and spillways measure approximately 40 m in height. See Figure 1.1 for the general site layout.

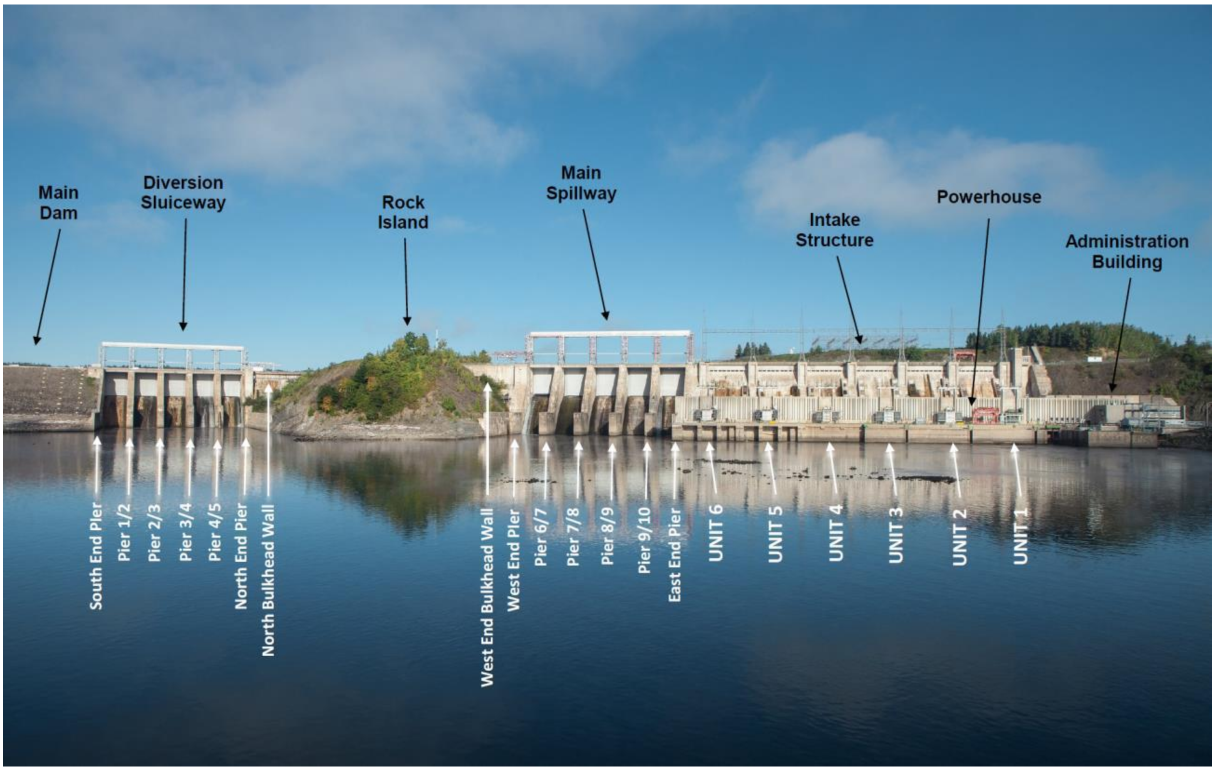


Figure 1.1: Downstream photo of Generating Station

The concrete structures; the spillways, intake and powerhouse, were constructed using locally-sourced aggregates and Normal Portland Cement. The coarse aggregate used was a greywacke excavated on site during the construction of the intake channel. The sand was a quartz and feldspar natural deposit excavated from nearby pits. The cement was produced by Maritime Cement Company Limited in Havelock, New Brunswick; the alkali content of the cement was approximately 0.62 – 0.81% Na_2Oe. Seven different concrete mix designs were used throughout the structures, they are summarized in Table 1.1 below.

Table 1.1: Concrete Mix Designs

Class of Concrete	Coarse Aggregate Maximum Size, in. (mm)	Approximate Cement Content, lb/yd^3 (kg/m^3)*	Design Minimum Strength 28 days, psi (MPa)	Design Minimum Strength 90 days, psi (MPa)
1	3 (75)	310 (185)	2,000 (13.8)	
2	3 (75)	395 (234)	3,000 (20.7)	
3	1 ½ (38)	435 (258)	3,000 (20.7)	
4	¾ (19)	460 (273)	3,000 (20.7)	
5	3 (75)	480 (285)	4,000 (27.6)	
6	1 ½ (38)	550 (327)	4,000 (27.6)	
7	¾ (19)	690 (411)	5,500 (37.9)	10,000 (68.9)

*Concrete pour records indicate that the cement contents varied by as much as 200 lb/yd^3 for Class 7, 100 lb/yd^3 for Classes 1 and 6, 85 lb/yd^3 for Class 4, and 25 lb/yd^3 or less for Classes 2, 3, and 5

Indications of stress-induced movements within the concrete, later identified as alkali-silica reaction, were first noted in the early 1970s. At this time a joint in the powerhouse was observed to be opening; its movement was monitored and theories began to be developed as the cause of the movement. However, it wasn't until the mid-1980s, approximately 20 years after construction began, that a more in-depth investigation into the cause of the movements was commenced. In 1985 gate 10, adjacent the intake structure, became jammed; this initiated a deeper investigation, revealing additional distress within the concrete and leading to the conclusion that alkali-silica reaction was the cause of the movement and distress occurring in the structures [1].

2. MITIGATION AND REMEDIATION OF ASR

The Station was initially designed and constructed for a 100 year service life. Extensive effort is expended each year to mitigate the impacts of alkali-silica reaction in an effort to extend the operational life of the structures. Until recently, different options were being considered for the future of the station: replacement of the powerhouse and spillways, decommissioning of the station, or rehabilitation of the existing structures to extend the life of the facility. After a thorough review of the options and investigation of the feasibility of maintaining the existing structures, the decision was made in 2016 to maintain the existing structures to achieve the 100 year design life. This will require ongoing mitigation of the impacts of ASR.

2.1 Prior ASR Mitigation Efforts

Historically, mitigation measures have included efforts to stabilize the existing structures, relieve stress in the concrete, and modify the mechanical components of the Station to accommodate the ongoing ASR expansion. The first stabilization project undertaken was the infilling of the east end pier stairwell with reinforced concrete. This reinforced the pier which had been damaged due to pressure from the expansion of the intake structure, causing Gate 10 to be jammed in 1985.

2.1.1 Slot Cutting

To relieve stress in the concrete of the intake structure and reduce the impact on the east end pier and main spillway, slots were cut between each of the intake units. Prior to cutting the slots, post-tensioned tendons were installed to enhance the stability of the structure; tendons were installed in 1988 – 1989, beginning with those closest to units 5 and 6 (near the East End Pier). Beginning in 1988, slots were cut between each of the units after tendon installation near those units was complete [2].

As the ASR progressed and the stress relief that was provided by the prior slot cuts diminished, additional slots were cut and the existing slots were re-cut to provide ongoing pressure relief [1]. Concrete expansion also causes the end piers of the spillways to tilt into the gates. To manage the tilting and pressure on the gates slots have been cut in these structures also [2].

2.1.2 Grouting

The ongoing expansion and movement of the concrete structures causes joints and cracks within the structures to open, allowing water movement through the structures. Differential ASR expansion has also caused the concrete structures, specifically the powerhouse, to debond from the bedrock and rise up in some areas, creating a void between the concrete and bedrock. Cores are drilled in the structures on a regular basis to permit the injection of cementitious grout, filling the cracks in the concrete and any voids created between the concrete and bedrock [2].

2.1.3 Spillway Gates

As noted above, the first operational difficulty observed as a result of ASR was the inoperability of one of the spillway gates. Since then, numerous repair and rehabilitation efforts have been undertaken to maintain the gates. The gates were made narrower, the sill beams were re-leveled, and the rollers, roller boxes, and seals were rehabilitated. Due to the vertical increase in height caused by ASR, the gate openings have become larger than the gates, preventing the gates from closing fully. Following the major rehabilitation of the gates, additional modifications to the spillway gates are undertaken on a regular basis to maintain operability [3].

2.1.4 Powerhouse and Mechanical Components of Units

The concrete deformation within the powerhouse is restricted on the upstream end by the intake structure and restricted on both sides by bedrock; thus a non-uniform expansion in the downstream direction is occurring. The non-uniform expansion caused a considerable stress concentration to develop in the penstock encasement concrete. To relieve the stress at this location couplings were installed in each of the penstocks. One coupling was installed each year from 1996 – 2001. The couplings are located between the intake structure and powerhouse, very near the powerhouse [1,2]. The installation of the couplings alleviated some of the downstream pressure on the powerhouse thereby reducing some alignment issues, reducing the stress in the stay vanes, and reducing concrete stresses in the draft tube piers [4].

The concrete expansion has also caused damage to the mechanical components of the units, causing ovalling of the discharge rings due to the non-uniform concrete expansion. The runner chambers were machined on several occasions to restore them to a near-round shape which allowed the runner blades to move freely and prevent significant diminishment in the efficiency of the units. After machining the runner chambers several times, the safe limit of material removal was approached and the stresses in the embedded steel parts were becoming critical. At this time, transverse slot cuts were cut in the powerhouse floor around the units to relieve stress in the concrete [1]. Numerous other adjustments of the mechanical components of the units have been undertaken to maintain operability amid the ongoing ASR movements.

The ongoing ASR has also caused damage to the structure of the powerhouse, leading to movement and rotation of the columns and misalignment of the overhead cranes. It has induced additional loads on, and caused significant damage to, the draft tube piers. The modifications to the powerhouse structure have included; the addition of supplementary supports to the generator floor, the reinforcing of the steel superstructure frame, modifications to the overhead crane, and other modifications [1].

2.2 Modelling the Impacts of ASR

The stress and movement within the structures has been modelled for several decades; these models have been used to identify the areas in greatest need of remediation and thus determine when and where to conduct further mitigation efforts. With the recent decision to extend the life of the station to at least the year 2068, the model is now required to simulate the future impacts of ASR 50 years into the future.

The ongoing time-dependent ASR-induced stresses, and resulting creep and strain in the concrete, have been modeled with GROW3D [4]. Modifications were made to the model in an effort to simulate the non-uniform concrete expansion and stress as a result of the non-uniform tensile strain.

However, the models have, until recently, used assumed relatively uniform compressive and flexural strengths and moduli of elasticity throughout the structures. Due to the differences in initial mix designs and the ongoing effects of alkali-silica reaction, the mechanical properties of the concrete are not uniform throughout the structures and they will not remain constant with time. Thus, this project is being undertaken to determine the present mechanical properties of the concrete and validate the model. Another study is underway at the University of New Brunswick to simulate future alkali silica reaction in cores extracted from the Station and predict the future mechanical properties of the concrete; the preliminary results of this study are discussed in greater detail in later sections of this paper.

Several models are under consideration for the prediction of the future properties of the Station and planning of appropriate mitigation activities. NB Power has recently announced a technical collaboration with Hydro-Québec for the refurbishment of the Station [5]; this collaboration might lead to changes in the modelling and mitigation processes underway at the Station.

3. PRESENT CONDITION OF THE STRUCTURES

At present, numerous components of the Station are in need of repair or modification. As noted above, the ongoing alkali-silica reaction has caused deformations in several areas of the Station including the spillway and intake gate openings and components of the units themselves. The structures are restrained from lateral movement on both ends by bedrock (see Figure 1.1), causing a non-uniform expansion in the upstream-downstream direction as well as an increase in height vertically.

Much of the ASR-induced distress occurring on the outside faces of the concrete structures has been exacerbated by ongoing freeze-thaw damage, resulting in further surface distress; see Figure 3.1. Based on recent cores extracted from the outside faces of the structures we understand that the depth of surface distress and cracking ranges from less than 5 cm in some areas to in excess of 1 m in other areas. In general, the deterioration was most severe in the areas of the intake structure over the penstocks (see Figure 4.1 b)); this is in general agreement with the finite element models.

Figure 3.1: Intake Structure and Main Spillway

4. CONCRETE TEST SAMPLES

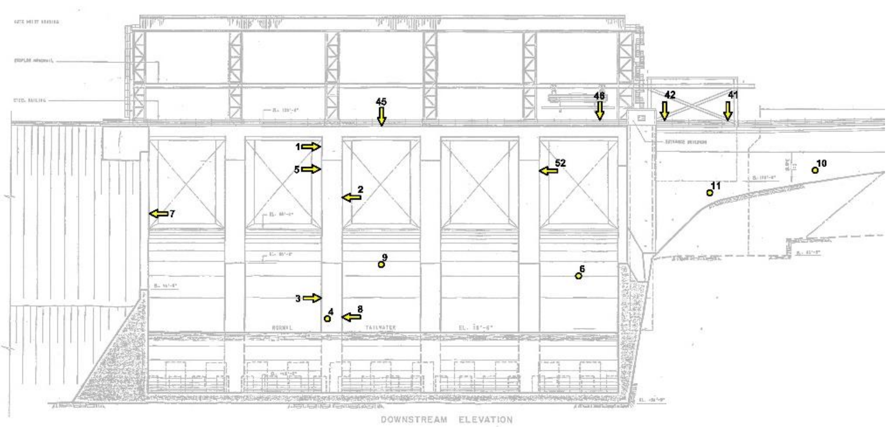
The present study included the extraction of 55 cores from the exterior faces of the structures; 16 cores were extracted from the diversion spillway, 17 cores were extracted from the main spillway, and 22 from the intake structure. The cores were extracted from various locations throughout the exterior faces of the structures; see Figure 4.1 for core locations on structures. Prior investigations by GEMTEC and the University of New Brunswick included the extraction of cores from the interiors of the intake structure and powerhouse.

The current investigation included 27 compressive strength tests, 16 splitting tensile strength tests, 20 modulus of elasticity tests, 14 petrographic examinations of the cores, and chloride penetration profiles from nine locations.

4.1 Visual Review of Cores

Prior to testing, all of the core samples were examined in the laboratory and photographed; fractures and other deficiencies were noted and the depth to sound concrete was reported. Core 2 (Figure 4.2 a)) is typical of many of the cores which were extracted intact and unfractured from the surface; the fracture in the photo was caused by coring activities only. Core 39 (Figure 4.2 b)) is one of the cores extracted over the penstocks; this core is from Unit 6, the unit closest to the East End Pier. All of the cores extracted from this area, mid-height near the centres of the penstocks, exhibited cracking and deterioration to depths ranging from 30 cm (Unit 1) to more than 1 metre (Units 3, 4, and 6). Unfortunately, limited testing was possible on these cores due to the magnitude of deterioration. The finite element models predicted that deterioration in the intake structure would be most severe in these areas; the yoke over the penstocks. See Table 4.1 for a summary of the depths to sound concrete at each of the core locations.

a)



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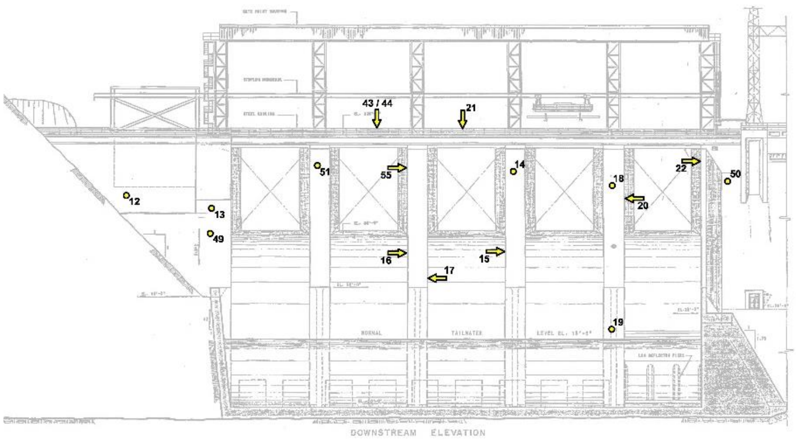

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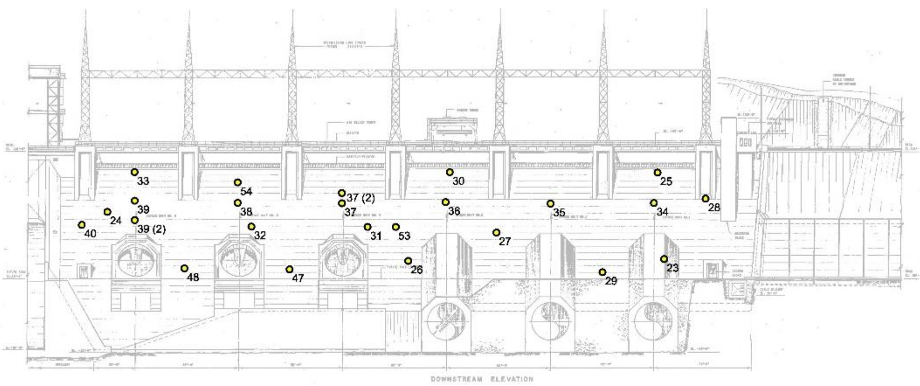

Figure 4.1: Core Locations on a) Diversion Spillway, b) Main Spillway, c) Intake Structure

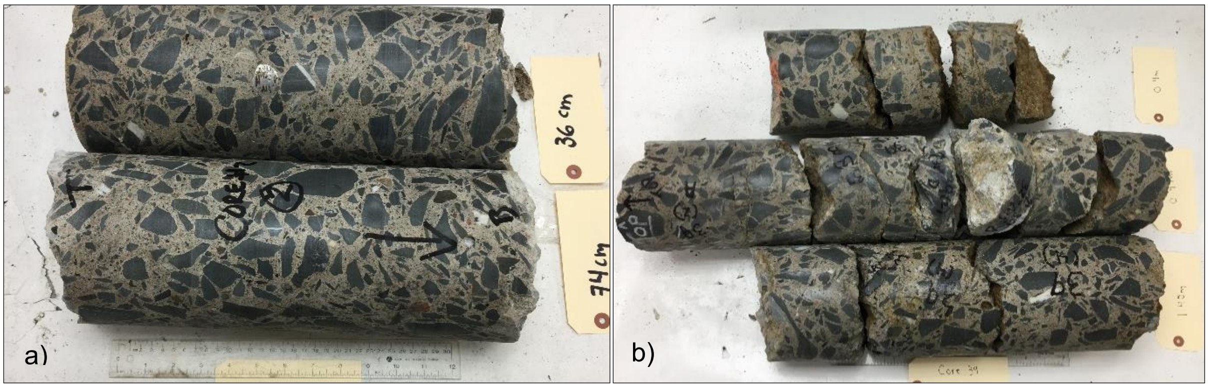

Figure 4.2: a) Core 2, from Diversion Spillway; b) Core 39, from Intake Structure, Unit 6 over Penstock

Table 4.1: Depth to Sound Concrete

	Core #	Depth to Sound Concrete (cm)	Core #	Depth to Sound Concrete (cm)	Core #	Depth to Sound Concrete (cm)
Diversion Spillway	*1*	4	*5*	0	*9*	27
	2	0	*6*	>59	*10*	0
	3	30	*7*	8	*11*	20
	4	0	*8*	45	*52*	0
Main Spillway	*12*	0	*17*	17	*22*	0
	13	20	*18*	20	*49*	10
	14	25	*19*	35	*50*	56
	15	18	*20*	0	*51*	15
	16	0	*21*	6	*55*	0
Intake Structure	*23*	29	*31*	0	*39*	>125
	24	15	*32*	>72	*40*	9
	25	0	*33*	15	*47*	0
	26	25	*34*	30	*48*	0
	27	20	*35*	35	*53*	22
	28	18	*36*	120	*54*	40
	29	13	*37*	>95		
	30	0	*38*	>59		

4.2 Compressive and Splitting Tensile Strengths

The compressive and splitting tensile strengths measured in the cores extracted during this program are summarized in Table 4.2 and detailed in the following sections of this paper.

Table 4.2: Summary of Concrete Core Strengths

	Core #	Compressive Strength (MPa)	Splitting Tensile Strength (MPa)	Core #	Compressive Strength (MPa)	Splitting Tensile Strength (MPa)
Diversion Spillway	*1*	36.6	3.5	*7*	34.7	
	3	21.4		*8*	25.1	1.3
	4		2.4	*9*	22.4	
	5		1.9	*10*	24.7	
Main Spillway	*12*		3.4	*20*	38.8	
	13	18.6		*21*	40.6	
	15	28.9		*49*	23.4	
	16	25.5	2.9	*50*	31.2	2.7
	17	29.5	4.5	*51*	31.7	
	19	18.2		*55*		2.4
Intake Structure	*24*		2.3	*35*	30.5	
	25	27.8		*36*	24.2	
	26		2.6	*39*	30.3	
	28	15.3		*47*		3.2
	31	45.2		*48*	28.8	
	32	39.0	4.3	*53*	30.1	2.3
	33	32.5	2.0	*54*	28.2	3.0

4.2.1 Diversion Spillway

The cores extracted from the diversion spillway were predominantly Class 5 or 6 concrete (Cores 1, 4, 5, 7, 8, 9) with a design compressive strength of 27.6 MPa (4,000 psi) and a maximum coarse aggregate size of 75 mm (Class 5) or 38 mm (Class 6). The compressive strengths of these cores range from 22.4 to 36.6 MPa, averaging 29.7 MPa. The splitting tensile strengths range from 1.3 to 3.5 MPa, averaging 2.3 MPa.

Cores 3 and 10 from the diversion spillway are Classes 4 and 3 concrete, respectively. The design compressive strength of these concretes is 20.7 MPa (3,000 psi) and the maximum coarse aggregate sizes are 38 mm (Class 3) and 19 mm (Class 4). The compressive strengths of these cores exceed the design strength by 0.7 MPa (Core 3) and 4 MPa (Core 10).

4.2.2 Main Spillway

The cores extracted from the main spillway are much the same as those from the diversion spillway; seven of the cores (15, 16, 17, 20, 50, 51, 55) are Class 6 concrete with a design strength of 27.6 MPa, the other five cores (12, 13, 19, 21, 49) are Classes 2 or 4 concrete with a design strength of 20.7 MPa.

The Class 6 concrete cores had compressive strengths ranging from 25.5 MPa to 38.8 MPa, averaging 30.9 MPa; they had splitting tensile strength strengths ranging from 2.4 to 4.5 MPa, averaging 3.1 MPa.

The Classes 2 and 4 concrete cores had compressive strengths of 18.2 to 40.6 MPa (Core 21, deck over spillway), averaging 25.2 MPa. The splitting tensile strength of one Class 2 concrete core (Core 12) was tested, a result of 3.4 MPa was obtained.

4.2.3 Intake Structure

The outside face of the intake structure includes Classes 2, 3, and 4 concrete which all have design strengths of 20.7 MPa and maximum coarse aggregate sizes of 75 mm (Class 2), 38 mm (Class 3), and 19 mm (Class 4). The concrete over the penstocks, up to the inspection gallery, is Class 3 concrete; the concrete between the penstocks, up to the gallery, is Class 2; the concrete immediately under the inspection gallery is Class 4.

The Class 2 concrete cores (24, 26, 47, 48, 53, 54) had compressive strengths of 28.2 to 30.1 MPa, averaging 29.0 MPa and splitting tensile strengths of 2.3 to 3.2 MPa, averaging 2.7 MPa.

The Class 3 cores (25, 31, 32, 33, 35, 36, 39) had compressive strengths of 24.2 to 45.2 MPa, averaging 32.8 MPa and splitting tensile strengths of 2.0 and 4.3 MPa (two cores tested).

One Class 4 core was tested for compressive strength (Core 28), a result of 15.3 MPa was obtained.

4.2.4 Summary of Strength Results

With few exceptions, the compressive strengths of the cores reached or exceeded their design strengths. The splitting tensile strengths were generally in the range of, or slightly less than, the strengths that would be anticipated for non-ASR-impacted concrete with these compressive strength results.

4.3 Other Laboratory Testing Underway

Modulus of elasticity testing is underway on six cores from each of the spillways and eight cores from the intake structure. These tests will be complete at the time of presentation at the ICAAR 2020 conference.

Fourteen core samples have been prepared for petrographic analysis; four from each of the spillways, and six from the intake structure. These results will be complete for presentation at the ICAAR 2020 conference.

5. POTENTIAL FOR FUTURE EXPANSION AND CHANGES IN MECHANICAL PROPERTIES

Laboratory testing underway at the University of New Brunswick includes the submersion of concrete cores in sodium hydroxide solution to accelerate alkali-silica reaction and evaluate the potential for future expansion in the structures. By submerging the cores in a solution rich in alkali hydroxides the reaction is accelerated because it is limited only by the availability of reactive silica in the aggregate particles; alkali hydroxides and water are abundantly available.

The cores were extracted from the upper and lower galleries of the interior of the intake structure; see Figure 5.1 for a cross-section through the intake structure indicating the locations of the lower and upper galleries.

The mechanical properties (compressive and splitting tensile strength and modulus of elasticity) of the cores were measured upon extraction from the structures (initial reading). The remaining cores are being stored in sodium hydroxide solution (1 Molar) in the laboratory at 22˚C, 38˚C, 60˚C, and within the lower gallery of the intake structure (11˚C to 14˚C). The length change of the cores in NaOH solution is measured regularly; when they reach 1500 μs, 3000 μs, 4500 μs, and 6000 μs the mechanical properties are being measured again. The maximum strain in the concrete structure at the present time is estimated at 6000 μs; therefore a measured strain of 6000 μs will be equal to approximately double the present strain.

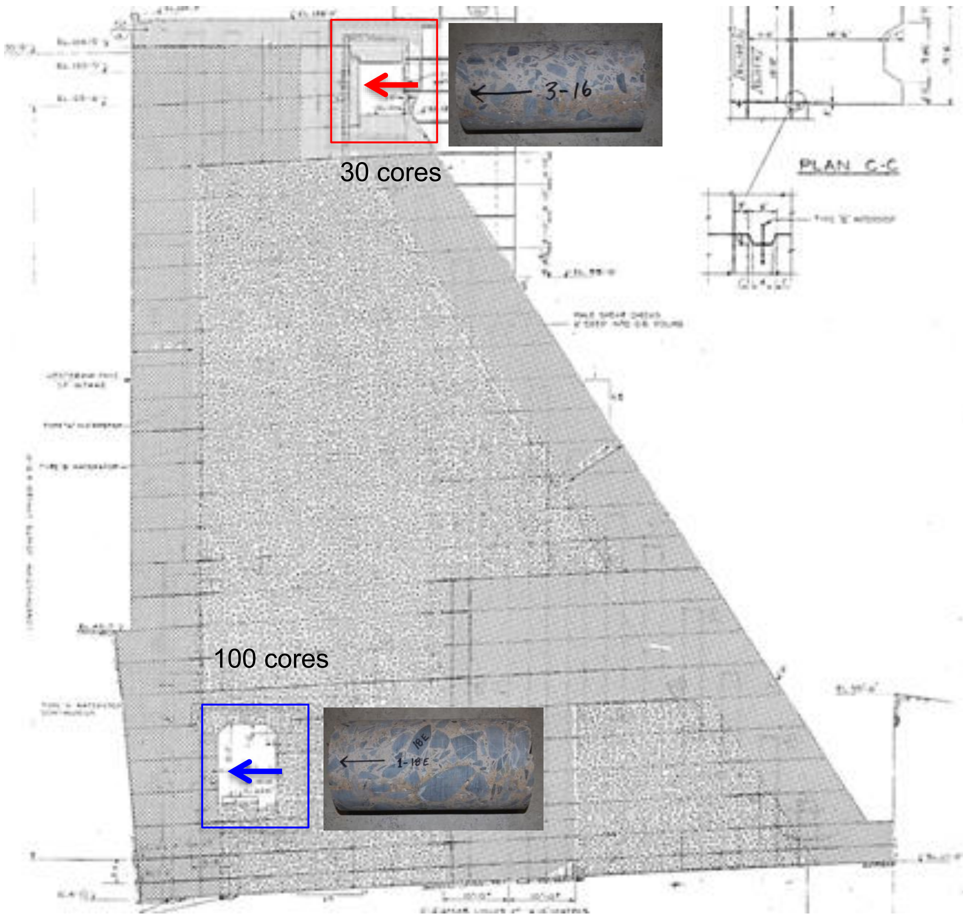

Figure 5.1: Cross-Section of Intake Structure showing Cores Locations in Upper (red) and Lower (blue) Galleries

5.1 Initial Properties

The initial mechanical properties of the cores extracted from the lower and upper galleries (elevation +20 ft., and +123 ft., respectively) are summarized in Table 5.1. The cores from the lower gallery are Class 2 concrete, those from the upper gallery are Class 4 concrete.

Table 5.1: Initial Mechanical Properties of UNB Cores

	Compressive Strength (MPa)		Modulus (GPa)		Splitting Tensile Strength (MPa)	
	Range	*Average*	*Range*	*Average*	*Range*	*Average*
Lower Gallery	15 – 26	20	7.3 – 15.4	11.1	1.6 – 2.7	2.3
Upper Gallery	17 - 29	25	8.5 – 14.9	11.6	1.9 – 2.6	2.3

5.2 Changes in Mechanical Properties due to ASR

As the cores reach the strain levels noted above (1500 μs, 3000 μs, 4500 μs, and 6000 μs) they are removed from the sodium hydroxide solution and the mechanical properties are measured again. Much of the testing is still underway; none of the cores have reached 6000 μs and only those stored in 60˚C solution have reached 4500 μs. The results thus far indicate that little to no reduction in compressive and splitting tensile strength is occurring as the strain increases; however, a decrease in modulus of elasticity has been observed, see Table 5.2.

Table 5.2: Mechanical Properties as a Function of Strain

		Initial (0 µs)	1500 µs	3000 µs	4500 µs	6000 µs
Compressive Strength (MPa)	*12˚C*	22.5	23.0	--	--	--
	22˚C	22.5	22.3	28.0*	--	--
	38 ˚C	22.5	22.0	22.0	--	--
	60 ˚C	22.5	22.3	23.3	25.0*	--
Modulus (GPa)	*12˚C*	11.4	10.3	--	--	--
	22˚C	11.4	6.7	12.4*	--	--
	38 ˚C	11.4	7.0	8.9	--	--
	60 ˚C	11.4	7.9	8.5	9.7*	--
Splitting Tensile Strength (MPa)	*12˚C*	2.3	1.8	--	--	--
	22˚C	2.3	1.8	--	--	--
	38 ˚C	2.3	1.9	2.2	--	--
	60 ˚C	2.3	1.9	2.0	2.1	--

*Only one core was tested

5.3 Rate of Expansion

As the cores submerged in NaOH solution continue to react and expand, the length change is measured at regular intervals, and the rates of expansion of the various cores can be estimated. The rates provide an indication of the availability of reactive silica in the cores and the potential for future expansion of the in-situ concrete. The results indicate that the rate of expansion is generally highest in the cores extracted from the southern end of the main spillway (near gate 6) and lowest in the cores extracted from the northern end of the intake structure.

6. CONCLUSIONS

The results collected thus far indicate that the compressive strength of the concrete is generally equal to or greater than the design strength of the concrete. Furthermore, little to no decrease in compressive strength is anticipated as the ASR continues.

Reductions in splitting tensile strength of between 5% to 20% are anticipated as the alkali-silica reaction continues.

The modulus of elasticity is the most sensitive, among the properties measured, to the negative impacts of alkali-silica reaction; reductions of 20 – 40% are anticipated as the reaction continues.

The depth of surface deterioration on the exteriors of the structures ranges from 0 (no damage) to more than 1.2 m. This deterioration is caused by a combination of alkali-silica reaction and freeze-thaw damage. The depth of deterioration is greatest in the areas over the penstocks. This is aligned with the modeled damage locations.

Additional modulus of elasticity results and petrographic analyses from the cores extracted from the exterior faces of the structures will be complete at the time of presentation at the ICAAR 2020 conference.

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