Rehabilitation of the West Tailrace Tunnel, Churchill Falls, Labrador

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ABSTRACT: The Churchill Falls Hydroelectric Project was watered up in 1971. The west tailrace tunnel was dewatered for inspection after 28 years in service. Most of the 1700 m long tunnel was in excellent condition. However, a 50 m long zone of altered schistose rock had weakened and failed due to chemical and physical weathering. The paper describes the investigation and assessment of the rock conditions, including the nature of the alteration. The remedial work consisted of staged dewatering of the tunnel, preparation of rock surfaces, application of shotcrete, installation of rock bolts, drilling of drain holes and removal of the rockfall debris from the floor of the tunnel. The objectives were to isolate the altered rock from the flowing water and to reinstate the rock support that had been originally installed. The work was performed in the summer of 2001.

1 INTRODUCTION

The 5428 MW Churchill Falls Hydroelectric Project was commissioned starting in 1971 and is the largest underground hydroelectric power development in the world. Figure 1 shows the location of Churchill Falls and Figure 2 presents the general arrangement of the facility. Benson et al. [1] describe construction and rock mechanics aspects. Merritt [2] describes aspects of the underground support design for the overall project.

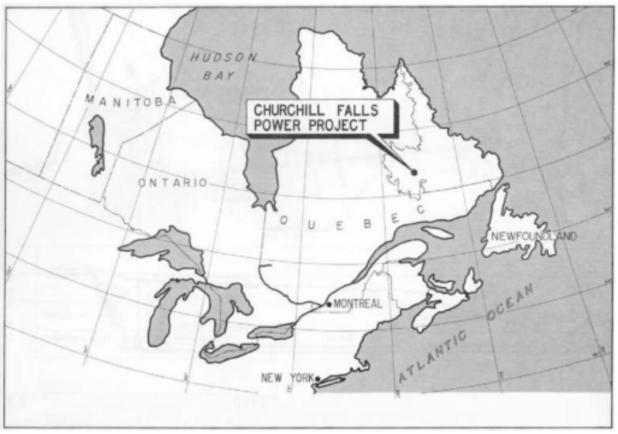

Figure 1. Churchill Falls location.

Churchill Falls (Labrador) Corporation (CF(L)Co) owns, operates and maintains the facility and has periodically performed maintenance and remedial work to ensure adequate long term performance. In 1998 CF(L)Co began a program of inspections in the two tailrace tunnels, which had not previously been inspected. The tunnels had apparently performed well during operation, with no evidence of blockage or unusual flow conditions. Nevertheless, CF(L)Co considered it prudent to confirm that no deterioration had occurred. The tunnels were inspected during planned periods of reduced power production, with one tunnel at a time being taken out of service.

The tailrace tunnels are 13.7 m wide, 18.3 m high, and each approximately 1700 m long. The design flow is 708 m^3/s at a velocity of 4 m/s. They are unlined for more than 80% of their length, with gunite (portland cement, sand and water, applied pneumatically) and rock bolts used locally for support, as well as concrete where shear zones were identified during construction. The tunnels were designed and inspected, and rock support installation was directed during construction, by Acres Canadian Bechtel.

2 TUNNEL INSPECTION

The east tailrace tunnel was inspected in 1998 and found to be in good condition. No remedial work was performed in the east tunnel.

The west tailrace tunnel was inspected in 1999. The tunnel was inspected from a boat, following a safety check of the arch. The water level was drawn

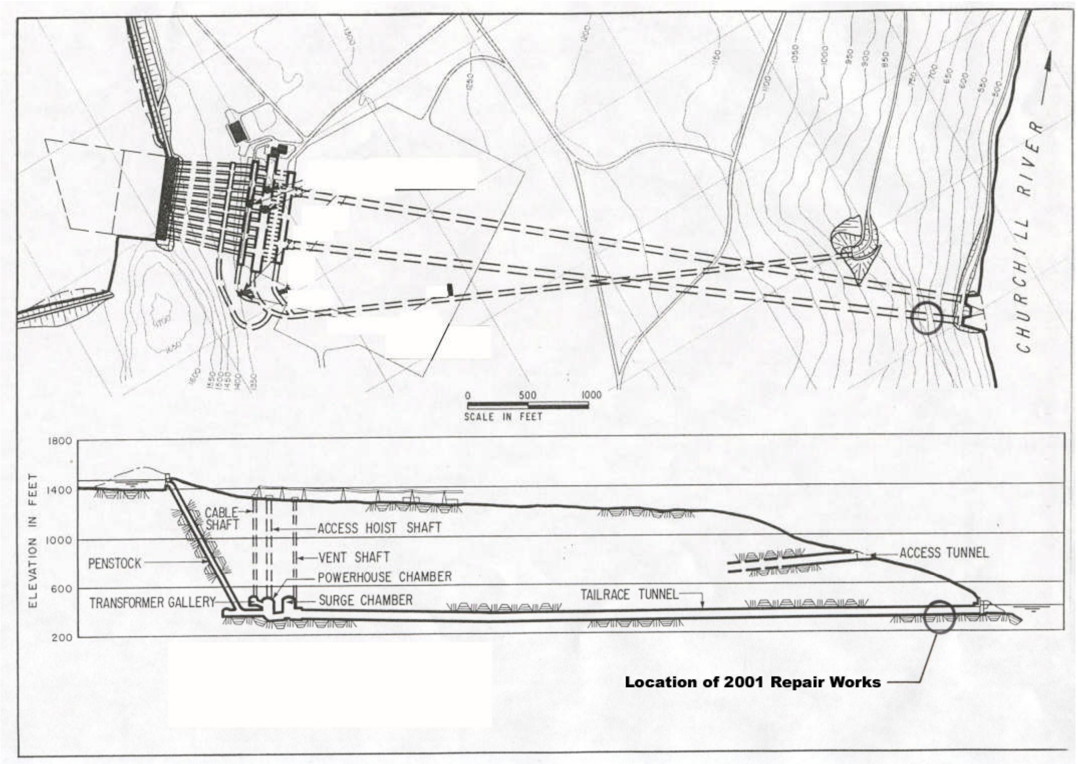

Figure 2. Project layout.

down in stages to permit inspections of both walls, as well as the arch. The tunnel was not completely dewatered, as it rises toward the downstream portal for most of its length as shown in Figure 2 and complete dewatering would have required considerable effort. The water depth was sounded at 15 m intervals to determine whether debris was present; this was reduced to as close as 1.5 m intervals when debris was identified. In addition, a diver inspected one area, described below, to better determine the nature and extent of debris.

Approximately 80% of the tunnel length is within massive rock and is unsupported. Of prime interest during inspection was the condition and performance of the remaining 20%, comprising mainly several gunited zones. These zones, generally consisting of weaker rocks, received a comprehensive treatment of gunite, rock bolts and wire mesh during construction. A major shear zone between stations 0+107 and 0+137 was treated with steel ribs, rock bolts, gunite, and concrete lining. It had performed well with no evidence of cracking of the concrete or gunite observed in 1999.

The condition of the rock and gunite was assessed by sounding with hammers and scaling bars. Rock bolts were assessed visually for signs of corrosion, and the tightness of bearing plates, washers and nuts was checked.

A zone of rock referred to as an altered zone was identified within the unlined portion of the tunnel between Stations 0+255 and 0+305 m, which had experienced weakening and local rockfalls. Loosening had progressed locally to a state that several 5 m long rock bolts had fallen from the arch. About 450 m^3 of rockfall debris, up to 3 m in dimension, had accumulated on the tunnel floor between stations 0+210 and 0+305. After sounding this area, a diver inspected the accumulation of debris to confirm the extent and size of the rock fragments.

Inflows to the tunnel were negligible during the inspection, with several areas of drips in the first 300 m from the portal, indicating a generally tight rock mass.

3 GEOLOGY OF ALTERED ZONE

The host rock is a gneissic assemblage intruded by gabbro, diorite, and minor syenite and pegmatite. The rock is generally very strong. Gneissosity is typically poorly developed, dips steeply and strikes approximately 60° to the tunnel direction. In the vicinity of the altered zone, the gneissosity is distinct, dipping to the downstream at 60°.

During construction the zone was mapped as a medium grained grey gneiss with moderately close joint spacing, varying from 25 mm to 300 mm. There was evidently no indication that the rock mass was weak or altered in any way. The area had been supported locally during construction with rock bolts and gunite, but most of the zone had been left unsupported and exposed. The construction specifications required a minimum of 50% of the blast hole traces to be visible on the final rock surface and a minimum of 80% of the final rock surface to be between the "A" and "B" lines after scaling. There was no record of the rock in the altered zone not meeting these requirements. Blast hole traces were noted throughout most of the unlined portion of the tunnel during the 1999 inspection.

Representative rock samples were collected during the inspection. Three samples from the altered zone were subjected to bulk density determinations. Their dry mass densities ranged from 2490 to 2670 kg/m^3, with respective porosities of 17 to 12%. This compares with typical densities in the order of 2700 kg/m^3 for these rocks [3]. Several samples from the altered zone had a somewhat rounded, weathered appearance, compared to the distinct, sharp edges of rock fragments from outside the altered zone.

Petrographic analyses were performed on several rock samples from the altered zone. Cavities comprised from 10 to 30% of the thin section areas, while anhydrite made up 1 to 11% of the samples, occurring on cavity walls and within sericite-rich bands. Biotite, with some alteration to clay, also occurred commonly on the edges of cavities. The cavities were up to typically 5 mm in dimension, with an extreme of 30 mm. The cavities were irregular but generally parallel to the mineral fabric. Anhydrite contents of 8% and higher occurred in biotite gneiss.

It is believed, from the evidence of the inspection, density measurements and petrographic analyses, that the cavities were originally occupied by anhydrite, likely in combination with biotite or sericite. The original rock mass did not appear weak as the anhydrite was in dispersed cavities rather than in continuous bands. Contact with the water in the tunnel at the unconfined rock surface resulted in alteration of anhydrite ($CaSO_4$) to gypsum ($CaSO_4 \bullet 2H_2O$), although gypsum was not identified in the petrographic analyses, and a corresponding volume increase. The volume change, subsequent dissolution of the gypsum and erosion of associated sericite, biotite or clay by the relatively high velocity water flow resulted in a weakened rock mass. This process migrated along the more anhydrite-rich bands or zones until large blocks of rock were left unsupported.

Joint spacing ranged from about 100 to 400 mm in the altered zone, compared to between 400 and 3000 mm in the typical massive rock for most of the tunnel length. In 1999 the zone was estimated to have a Q value [4] of approximately 4, which would require support of pattern rock bolts on 2 m centers and 50 mm of shotcrete.

During the remediation, described below, the penetration rate (penetration per unit time) was recorded for several of the holes drilled for rock anchors. On average, the penetration rate in the walls for the first metre was 30% higher than for greater depths. In the arch there was no difference in penetration rates near the tunnel surface and deeper; this may have been because of the difficulty of drilling overhead, or because weakened rock fell from the arch much more readily than from the walls.

4 TUNNEL REMEDIATION

It was concluded that due to the nature of the mineralogy and structure in the altered zone, erosion of the tunnel walls and arch would continue if remedial measures were not undertaken. The objectives of the remediation were: to replace the rock support that had fallen out due to the erosion and rock fall; to support the weakened exposed rock surface; to provide a barrier between the rock surface where anhydrite was present and the free water in the tunnel; and to perform the necessary work within the planned shut-down period of 68 days in order to permit power sales according to an established schedule.

Alternatives considered were to install a concrete lining, and to install rock bolts and cover the altered zone with shotcrete. The concrete alternative had several disadvantages compared to rock bolts and shotcrete:

- Large volumes of concrete would be required to fill the irregular shape of the eroded tunnel.
- The tunnel would have to be completely unwatered and the invert cleared of debris, then the formwork and concrete placement proceed from bottom to top, whereas bolts and shotcrete could be installed as the water level was drawn down, resulting in a significantly shorter construction period.
- Either a large amount of falsework would be required for formwork for the concrete placement, or steel sets (or equivalent), which could be embedded in the concrete while providing form support would be required. Either option would require a substantial time to erect.
- The tunnel arch and walls would likely require at least local support for safety before personnel could work at lower levels placing concrete. Using rock bolts and shotcrete, all support including "safety" and "permanent" support could be installed at one time, from top to bottom.

- It was concluded that a concrete lining could not be placed in one season with the work window available, whereas rock bolts and shotcrete could be installed over the entire zone in one season.

Consequently, specifications were prepared and a contract was awarded for the rehabilitation of the altered zone by means of rock bolts and shotcrete. Figure 3 shows a section of the tunnel and support.

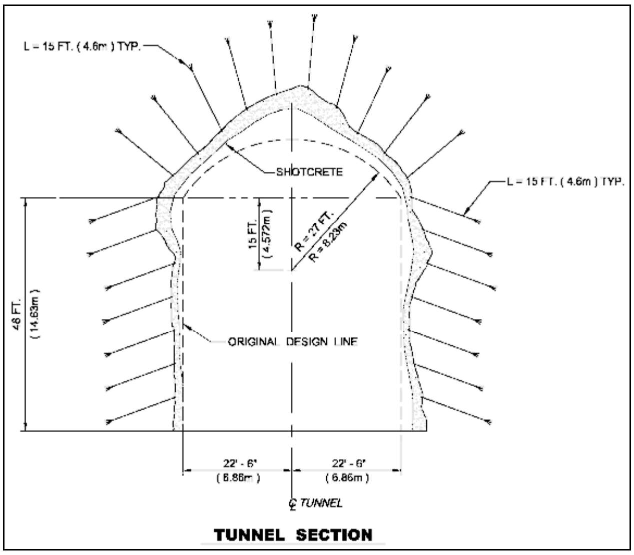

Figure 3. Tunnel Section.

4.1. Construction Procedure

A construction contract was awarded in May 2001 for the rehabilitation of the altered zone in the west tailrace tunnel. The Contractor mobilized all plant and equipment by truck to Churchill Falls. CF(L)Co installed stoplogs at both ends of the tailrace tunnel and the Contractor commenced pumping on June 24, 2001.

The water level was drawn down to approximately 6 m below the arch at the altered zone, and a 15 m long, 12 m wide sectional barge was assembled at the tunnel portal as the primary work platform. It was fitted out with a shotcrete plant complete with robotic shotcrete arm, wagon drills for rock bolt installation, and a personnel lift. The barge was taken to and remained at the work area, and a smaller shuttle barge delivered materials as required. Personnel moved between the portal and work area in small aluminium boats. Figure 4 is a sketch of the barges. Figure 5 shows the robotic arm for shotcreting, being tested at the portal. An initial inspection of the arch, scaling for safety, and installation of ventilation line were performed in advance of the support installation.

The water level in the tunnel was drawn down to an intermediate level after the arch support was completed, exposing approximately 6 m of the walls for support installation, and then the tunnel was unwatered for the final support installation on the walls and cleanup of the debris on the tunnel invert.

The value of the contract as performed was $3.13 million. The tunnel was out of service for 54 days.

4.2. Shotcrete Support

The design support was a minimum 100 mm thickness of steel fibre reinforced shotcrete and pattern rock bolts in the arch and walls. To ensure safety of personnel, the specifications required an initial minimum 50 mm layer of shotcrete on the arch prior to bolt installation. The robotic arm for remote application of shotcrete was also specified as a safety enhancement. Application of shotcrete in either one or two layers on the walls was dependent on a safety assessment of the rock conditions. In the event, the walls were shotcreted in a single layer after installation of rock bolts. A booster fan was used to enhance ventilation during shotcrete application. The barge was fitted with suspended "aprons" to catch rebound that would otherwise fall into the water.

Rebound was in the order of 30% when shotcreting the arch and 20% for the walls. Sloughing occurred occasionally due to poor nozzling technique, in particular due to inexperience with controlling the

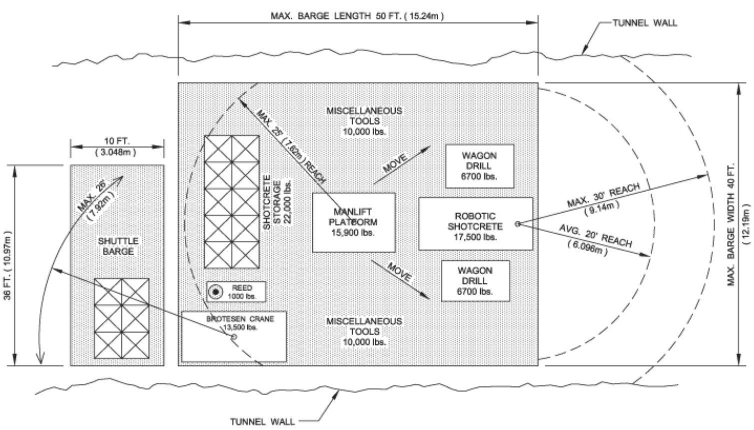

Figure 4. Barge arrangement.

robotic arm, and where excessive thicknesses built up over very irregular rock surfaces.

Prior to applying shotcrete the rock surface was washed using water at 500 kPa (75 psi) pressure, with the nozzle held within a metre of the rock surface with the robotic arm. Hand scaling was kept to a minimum for safety.

Shotcrete thickness was controlled by setting depth gauges on the shotcrete anchors and probing during application. The latter method was problematic because of the dust and rebound and, in the arch in particular, the remoteness of the application with the robotic arm. Several check holes drilled after the shotcrete was completed, indicated average thicknesses of 140 and 170 mm in the arch and walls, respectively. Two check holes, low on the west wall, showed thicknesses of 90 to 95 mm; the shotcrete was accepted without thickening to the minimum specified 100 mm.

The shotcrete specifications were based on ACI 506 [5], with specification requirements as summarized in Table 1.

Figure 5. Robotic arm for shotcrete application.

Type 50 sulphate resisting cement was specified to prevent possible reaction with the sulphates identified in the rock. The choice of dry or wet mix was left to the Contractor. He chose dry mix, due to the remoteness of the working face some 300 m from the portal.

Shotcrete with the components listed in Table 2 was supplied in 1000 kg "superbags". Powdered accelerator was added at the batching plant, and water was drawn from the tunnel.

Drain holes were drilled 6 m deep on an average 4.3 m pattern each way after shotcreting, to prevent build-up of pressure behind the shotcrete.

Table 1. Shotcrete specifications.

Property	Age, days	Specified Limits
Maximum water:cement ratio, wet mix		0.45
Maximum aggregate size, mm		10
Slump, mm (wet mix), CSA A23.2-5C		80 ± 30
Minimum compressive strength, MPa, CSA A23.2-14C	7 28	20 30
Maximum boiled absorption, %, ASTM C642	7	8
Minimum volume of permeable voids, %, ASTM C642	7	17
Minimum flexural toughness, ASTM C1018	28	$I_5 = 4.0$ $I_{10} = 6.0$ $I_{20} = 10.0$
Core grade, ACI 506.2		Max 3.0 Mean ≤ 2.5
Minimum steel fibre content, kg/m^3		45
Minimum silica fume content, % of cement		10

Table 2. Shotcrete components as supplied.

Material	Content, kg/m^3
Type 50 cement	440
Silica fume	44
Coarse aggregate 5 – 10 mm	330
Concrete sand	1386
Dramix ZP305 steel fibre	45

4.3. *Rock Bolts and Shotcrete Anchor Bolts*

Rock bolts, Grade 510 MPa (75 ksi), 22 mm in diameter and with mechanical anchors, were supplied in lengths of 4.6 and 6.1 m, with the longer bolts installed in areas of extensive rockfall where the tunnel size was significantly larger than the original design dimensions. The bolts were specified to be installed on a pattern of 2.1 m each way, tensioned to 10% of their ultimate strength to provide a snug fit of bearing plate (on a mortar pad) to the rock, and then fully grouted with cement grout with a minimum specified unconfined compressive strength of 30 MPa at 7 days.

The bolt spacing was reduced and the longer bolts were installed in areas of intense foliation and large overbreak. Approximately 25% more bolts were installed in the right wall than the left, due to the unfavourable joint orientation.

The rock bolt bearing plates were fitted with shotcrete spiders similar to those described by Ripley et al. [6]. Shotcrete anchor bolts, 15 mm diameter and 500 mm long, also with spiders, were installed at intermediate locations between rock bolts to provide an average spacing between shotcrete anchor points of 1500 mm.

4.4. Debris Removal

As noted previously, about 450 m^3 of debris was deposited on the tunnel floor in the vicinity of the altered zone. The constriction created by the debris caused a slight increase in head loss in the tunnel. The relatively high design velocity of 4 m/s increased the impact of the debris on the head loss.

Removal of the debris was the final item of work prior to demobilization from the tunnel, as removal could only occur after complete dewatering. If the debris was not removed the tunnel could be put back in operation several days earlier than otherwise. In order to determine whether removal was beneficial, an energy loss analysis was performed.

During the tunnel inspection in 1999 the tunnel overbreak was measured at several locations within and beyond the altered zone. This permitted calculation of the average cross section area and an estimate of the roughness. The universal chart for pipe hydraulic friction loss [7] was used to calculate a difference in head loss of 46 mm with the debris left in place and removed. As a check, Rahm's empirical method [8], based on measurements in several Swedish tunnels up to 150 m^2 cross section (compared to 178 m^2 and up to 240 m^2 for the tailrace tunnel design line and at the maximum overbreak section in the altered zone, respectively) was applied, yielding a 67 mm difference in head loss. The lower calculated difference was used to estimate the energy loss to demonstrate the benefit of debris removal.

Assuming an overall efficiency of 88%, the benefit due to debris removal was 335 kW at the maximum design flow of 708 m^3/s. The removal cost was estimated at between $10,000 and $50,000, depending on the amount of large debris that would require splitting before removal (prior to unwatering during the remediation, only the upper surface of the debris had been inspected by a diver). The higher cost could be recovered in very few years, even with a low incremental energy cost, and the decision was made to remove the debris.

5 SHOTCRETE QUALITY

The quality of the in place shotcrete is dependent on the materials, the supply, delivery and application, and, to a greater extent than many structural components, the experience and training of the personnel. The shotcrete quality for the tunnel remediation was assessed by qualifying the nozzlemen and by testing during construction.

5.1. Pre-Construction Testing

A pre-construction testing program was implemented to evaluate the proposed materials, equipment and personnel before mobilizing to Churchill Falls. This was considered a mandatory requirement due to the remoteness of the site.

Testing was performed at the Northern Centre for Advanced Technology (NORCAT) facility at Falconbridge Mines, Onaping, ON. The Contractor used this opportunity to have four nozzlemen take a one-week course prior to the qualification trials. The nozzlemen were required to shoot test panels in various orientations, with shotcrete anchor bolts and spiders installed. Not all testing was completed, but the nozzlemen demonstrated their competence. The outstanding panels were shot on site near the tailrace tunnel portal prior to any shotcrete being placed underground.

5.2. Production Testing

The Contractor was required to shoot one test panel for each 40 m^3 of shotcrete and a minimum of one panel per day. Panels were stored on site for 3 days, then moved to the site laboratory for sampling.

Samples were cored or sawn from test panels on site, then shipped to St. Johns for testing. Construction test data are summarized in Table 3.

Table 3. Production shotcrete test data.

Test	No. Tests	Min	Max	Ave	Specification Requirement
UCS, MPa, 15d	83	27.8	44.9	36.5	N/A
UCS, MPa, 28d	92	33.2	61.6	49.1	30 Min
Core Grade	89	1.0	2.5	1.04	2.5 Ave, 3.0 Max
Boiled Abs, %	19	4.1	7.1	5.9	8 Max
Perm Voids, %	19	8.6	18.4	13.9	17 Max
I_5, 24d	2	2.9	3.5	N/A	4 Min, 28d
I_{10}, 24d	2	4.9	6.7	N/A	6 Min, 28d
I_{20}, 24d	2	8.8	13	N/A	10 Min, 28d

Record tests for unconfined compressive strength, UCS, were performed in St. Johns. Due to the logistics of shipment, the earliest time for which consistent record testing could be performed was 15 days.

Two of 19 tests for permeable voids exceeded the specified maximum value.

Two flexural toughness tests were performed on beams at 24 days. Two beams were also tested at 28 days but the dial gauge support failed in one case and erratic readings, not considered representative, were recorded in the other case. One 24-day test failed to achieve the specified 28-day toughness indices, while the second beam met the I_{10} and I_{20} requirements. Considering the overall quality of the shotcrete, all shotcrete was accepted despite the few tests not complying with the specification requirements. The shotcrete was judged to be of excellent quality.

6 CONCLUSIONS

The west tailrace tunnel at Churchill Falls was inspected after 28 years of operation. The great majority of the largely unlined tunnel was in excellent condition. However, a 50 m long zone had experienced deterioration due to a combination of chemical and physical weathering. The potential for this development was not evident at the time of construction.

Remediation of the altered zone was performed using rock bolts and shotcrete. The work was carried out from a barge, working in stages in the arch and walls. This procedure resulted in a relatively fast, efficient methodology for support installation. The tunnel was watered up and returned to service 54 days after dewatering started.

7 ACKNOWLEDGEMENTS

Dynatec Corporation, Toronto, ON, performed the remediation. Quality testing was performed by Newfoundland Geosciences, St. Johns, NL. CF(L)Co's independent consultants were Drs. R.P. Benson and A.H. Merritt.

The authors thank Churchill Falls (Labrador) Corporation for permission to publish this paper.

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