Construction of Niagara Tunnel Project

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ABSTRACT
The world’s largest hard rock Tunnel Boring Machine (TBM) was utilised to bore a 10.2km long water conveyance tunnel under the City of Niagara Falls, Ontario, Canada. The tunnel was constructed to divert an additional 500m$^3$/s of Niagara River water to the Sir Adam Beck (SAB) No.1 & No.2 hydroelectric generating stations. In the project area, the host rock mostly ranks as very strong although the Queenston formation which about 35 per cent of the tunnel is driven through, falls in to the medium/low rock strength group and has swelling potential when in contact with fresh water. To avoid the swelling potential of the Queenston formation during operation of the tunnel, a layer of waterproofing membrane was placed between the initial shotcrete lining and final concrete lining. The concrete lining was pre-stressed to balance the hydraulic pressure during operation. At the Niagara Tunnel Project site, a high horizontal in-situ stress regime exists. The horizontal to vertical stress ratio reaches up to 5 and tunnel had to be driven to 140m (460ft) below the ground surface to pass under the buried St. David’s Gorge. Dealing with deep crown overbreak induced by rock stress, complicated logistics associated several concurrent operations, single lane traffic, ventilation and dust control were among the project’s challenges. This paper provides an overview of the design and construction challenges of the Niagara Tunnel Project.

1 INTRODUCTION

It took the world’s largest hard rock tunnel boring machine, nicknamed Big Becky, four and a half years to drive a 10.150m long, 14.44m diameter tunnel under the City of Niagara Falls, Ontario, Canada. The tunnel was constructed to divert an additional 500m$^3$/s of Niagara River water to the Sir Adam Beck (SAB) No.1 & No.2 hydroelectric generating stations. This additional water will permit the existing Sir Adam Beck generating stations to generate 13% more energy, increasing average annual output by about 1.5 TWh, enough to supply 150,000 Ontario homes with clean, renewable hydroelectric power. In 2005, Ontario Power Generation (OPG) awarded the design-build contract to Strabag AG of Austria for construction of the Niagara Tunnel. The tunnel entered commercial service in March 2013 and is expected to operate maintenance free for at least 90 years. This report briefly reviews the tunnel construction.

2 THE TBM

A Robbins (Ohio, USA), open gripper, main beam TBM was utilized to excavate the Niagara Tunnel. The back-up unit was supplied by Rowa Tunnel Logistics of Switzerland. TBM and back up unit components were delivered from various countries around the world to the TBM assembly area at the tunnel outlet for the first time assembly and commissioning. Due to the tight schedule, preassembly in the factory was waived. The use of the Onsite First Time Assembly (OFTA) technique by
Robbins saved approximately 4-5 months on the schedule. After the contract was awarded to Robbins, the TBM was ready to bore within one year. The cutterhead utilized eighty-five, 500mm (20-inch) diameter cutters. The thrust pressure on each cutter was 35t. The weight of the TBM and trailers was about 4,000t. The cutterhead weighed about 400t. TBM and back up unit length totalled about 150m.

Excavated material was transported by means of a 1.07m (42-inch) wide conveyor belt to the disposal area. The drive motors of the conveyor belt were located in the outlet cut, however a booster station was used approximately half way in the tunnel at about CH4+000 (Note: All tunnel chainages are in metres from the outlet portal). Wet shotcrete, to be applied as the TBM advanced, was transported to the TBM by 12m³ capacity mixer trailers. A workforce of 25 miners were occupied 24/7 on the TBM and back up unit.

The TBM typically mined for 20 hours a day with 4 hours for maintenance. During maintenance time, cutters were checked for wear or damages and were changed if necessary. The TBM passed through various rock types from hard to moderate strength. Usage of cutters was dependent on rock type and varied from about 200 cubic metres to about 2,000 cubic metres per cutter. The overall cutter usage was 1,650m³/cutter. During the excavation of the Niagara Tunnel a total of about 1,000 cutters were changed.

The maximum TBM daily, weekly and monthly advance were 25.5m, 153m and 472m.

3 MINING HISTORY:

On the first day of September 2006, Big Becky started pushing against the rock face at the tunnel Outlet portal. On March 30, 2011, the TBM substantially completed her journey to the Intake end of the tunnel. The excavation was spread over 1,671 days which included 439 days of shutdowns for planned maintenance such as conveyor extension or for modifications to the TBM and back-up trailers. Excluding the shutdown days, the TBM advance rate averaged 8.2 metres per day.

4 ROCK SUPPORT:

Primary support of the tunnel was installed based on designs by ILF of Innsbruck, Austria, the design partner of Strabag, and consisted of 4m long Swellex type rock bolts, 10cm×10cm×6mm welded wire mesh and 150×16 "C" channels from the 10 to 2 o'clock positions in the tunnel roof. The "C" channels were used as a template for a row of rock bolts and they were spaced between 0.7m to 1.1m longitudinally, depending on rock conditions. Bolts, "C" channels and wire mesh were installed immediately after excavation behind the TBM cutterhead, at the “L1” area. The whole circumference was then covered by a layer of 25MPa unreinforced shotcrete, 5 to 20cm thick depending on rock conditions, 40m behind of the cutterhead at the “L2” area of the back-up unit.

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Figure 4: Typical overbreak in Queenston shale above the cutterhead.
CONCRETE LINING:

A two-pass lining system was used for the tunnel. The above-mentioned L1 and L2 rock support formed the first pass or initial lining. The final or permanent liner, consisting of a waterproofing membrane and 60cm of unreinforced, 38 MPa (at 90 days), cast in place concrete, was placed well after TBM excavation when rock convergence had stabilised. The impermeable membrane and concrete liner were placed in two stages, Invert and Arch. The Invert portion formed about one third of the circumference (112 degrees), and the Arch covered the rest. Each had a separate support train. The Invert was placed first under a self-propelled 250m bridge unit to permit uninterrupted traffic to the TBM operation. The Arch form followed, running on the invert concrete and constructed such that traffic could continue below the forms throughout Arch membrane and concrete placement. Two 12.5m forms for both the Invert and Arch concrete operations permitted daily advance of 25m.

WATERPROOFING GEOMEMBRANE LINER:

To avoid the swelling potential of the Queenston formation in contact with fresh water during operation of the tunnel, the design required a layer of waterproofing membrane to be placed between the initial and final lining. The waterproofing system consisted of a layer of geotextile fleece, placed against the shotcrete liner to avoid direct contact between plastic membrane and the shotcrete surface, a double layer of 2 and 1.5mm thick polyolefin (FPO) vacuum testable membrane in the Invert, and a layer of 3mm thick FPO electrically testable membrane in the Arch. All seams were heat welded and 100% were tested in accordance with the design requirement.

To overcome internal hydraulic pressures during operation, ILF’s design required pre-stressing of the concrete liner by means of pressurized grout behind the waterproofing membrane. The gap between the concrete liner and the membrane was filled by contact grouting to...
prevent damages to the geomembrane during the pre-stressing process. All necessary hoses with detailed connections and fittings to the waterproofing membrane were installed in the concrete liner. Depending on the surrounding rock and tunnel depth the grout pressure varied up to the maximum of 15bars. To avoid excessive ovalization of the concrete liner in reaction to the pre-stress pressure, a laser scanning system combined with concrete surface 3D monitoring and tape extensometers, was used to monitor concrete deformations during the grouting process. Strain gauges embedded in a few sections of the concrete lining were also used to measure the actual stresses inside the concrete liner induced by the grout pressure.

Figure 9: Layers of membrane and concrete lining.

7 GEOTECHNICS, GEOLOGY AND ALIGNMENT:

The geology throughout the tunnel is varied, consisting of sedimentary sandstone, dolostone, siltstone, shale and mudstone formations of the Middle and Lower Silurian period overlying the Queenston silty mudstone of Ordovician period. All layers are near horizontally bedded. No major faulting is present in the project area.

The tunnel alignment is shown on Figure 1. The Intake is located in the river bed, about 30m below Gate #1 of International Niagara Control Works (INCW) and about 2km upstream of the Horseshoe Falls. The INCW was build in 1950’s to facilitate distribution of the Niagara River water in accordance with the 1950 Treaty between Canada and the United States for scenic flow over the Falls and for hydroelectric power generation.

The outlet of the tunnel is connected by a 300m long canal to the existing canals near the Sir Adam Beck Pump Generating Station (PGS). The tunnel alignment was changed during the excavation to shorten driving of the tunnel in Queenston formation. To pass under the buried St. David’s gorge, as mandated in the Environmental Assessment, the maximum depth of the tunnel is 140m. The St. David’s gorge is an ancient Niagara River valley that is backfilled with glacial deposits.

The TBM first pushed in a southwest direction from the outlet cut. Descending on a 7.82% grade, the tunnel reaches the low point at CH 1+411, where five dewatering shafts with finished diameter of 0.75m are located. After two horizontal curves starting at CH 3+298 and an upward slope of 0.01%, the tunnel drives towards the south to CH 7+393 where it makes left turns towards the southeast and under the Niagara River to the Intake. Within the north-south section of the tunnel, between CH 3+304 and CH 3+931, the tunnel elevation rises up for 42m, reaching a depth of about 100m. Finally from CH 9+567, with an incline of 7.15%, the tunnel drive ascends to the Intake. All the curves have a minimum 1,000m radius to accommodate the continuous muck conveyor.

At the Niagara Tunnel Project site, like everywhere in southern Ontario, a high horizontal in-situ stress regime exists. The highest stress is in NE/SW direction. The horizontal to vertical stress ratio reaches up to 5 with an average of 3. In the project area, the host rock mostly ranks as very strong. The uniaxial compressive strength (UCS) of the rocks varies from 8 to 240 MPa, with the majority between 100 and 180MPa. The Queenston formation falls in the lowest strength group of the rocks. Unlike the first 300m of both intake and outlet ends, the rest of the tunnel is driven in impermeable rocks. Some very minor quantities of corrosive, saline, connate water exist in the other formations but had no effect on mining.

Figure 10: TBM Operator’s cab.

8 OVERBREAK:

Some overbreak at the crown occurred during TBM mining. Most of the overbreak occurred in the Queenston formation. In total about 50,000m$^3$ in the crown and about 10,000m$^3$ of overbreak in the invert occurred during or shortly after the TBM mining. The maximum depth of overbreak at the tunnel roof reached about 4.5m in the Queenston formation around CH 3+754.
This overbreak necessitated some modifications to the TBM. Cables and hoses on the TBM were covered immediately behind the cutterhead, to protect them from falling rock. The rock support had to follow the overbreak profile, which was not circular. Stronger rock drilling booms with higher reach were installed at the L1 area of TBM. Sliding access platforms at L1 were replaced with man lift baskets, to enable reaching into the overbreak areas.

Figure 11: Restoration of overbreak.

Soon after two kilometres of TBM advance, and experiencing some deep crown overbreak, it became obvious that ground restoration activities would be needed to make the tunnel section circular as designed for waterproofing membrane installation and application of pre-stress without inducing bending moments in the concrete. Crown restoration carriers needed to be designed, manufactured and installed to backfill the overbreak properly and safely. A grout carrier was also needed to fill cavities within the rock and possible gaps between the rock, rock support system and backfill material. These additional carriers launched into the tunnel after the invert concrete bridge.

9 INSTRUMENTATION AND MONITORING:

Instrumentation was used to verify the stability and ground reaction to the excavation. Also some instrumentation was used to verify the effects of excavation on adjacent structures. Eight multiple point borehole extensometers (MPBE) and six vibrating wire piezometers were installed at the tunnel low point (CH 1+416) as per Owner’s mandatory requirements in the contract. Low point instruments were cabled through a bore hole to the surface with the data logger installed at the top of the bore hole for convenient access and readings. Permanent stand pipe piezometers were installed at CH 8+002 and CH 3+369 to measure the water pressure inside the tunnel. For flow quantity verification, an ultrasonic flow meter system is installed inside the tunnel on the concrete liner close to the Outlet structure.

For verification of rock support performance during construction and for safety purposes, arrays of temporary MPBE’s were installed at CH 3+612, CH 5+200 and CH 6+050. Temporary 3D monitoring points were installed regularly on the initial lining shotcrete every 200m along the tunnel. Intervals for 3D monitoring were reduced to 25m in sections where deemed to be necessary. Tape extensometer arrays were also used immediately after TBM excavation until the TBM trailers passed the area.

The maximum rock movement recorded by single instrumentation was 90mm at CH 3+595 by 3D monitoring point.

Thorough monitoring of the concrete liner deformation was performed in order to prevent damages to the concrete liner from excessive pre-stressing grout pressure. In both contact and pre-stress grouting stages, the concrete liner deformation was monitored in order to capture all deformation data. Based on the host rock characteristics, the actual concrete compressive strength and the as built concrete liner thickness in every grout section, the maximum allowable pressure and resulted deformation were calculated. In these calculations stress loss due to creep and shrinkage over the time of the pre-stressing and Tunnel water-up were observed. Stationary method of concrete liner deformation measurement including Tape Extensometer and Stationary Survey Points were used for every concrete bay in the contact grouting stage and partially in prestress grouting stage. To measure the actual stress in some selected bays embedded strain gages were used. The mobile deformation measurement method consisted of a laser scanner mounted on a carrier continuously monitoring the deformations of the whole concrete bay, and a cable connected to the grouting control room which controls the grouting operations. Finally the actual monitoring results at the early stages of grouting were used to adjust the calculated limits.

Figure 12: Concrete deformation monitoring system carrier.
10 ADDITIONAL WORKS:

1 Some additional rock support was installed in the tunnel where the necessity was defined by either instrumentation or visual inspection of the tunnel. Additional rock support included 4m and 6m long Swellex bolts, fully grouted 4m and 6m long IBO type bolts, additional wire mesh and shotcrete. Type and density of additional rock support varied depending on the rock conditions at the specific location. Bolts were installed using an Atlas Copco Boomer 135 jumbo drill or the crown restoration carriers.

2 In some areas additional wire mesh with short bolts, were necessary to protect workers and equipment from loosened and falling shotcrete. Even slight movement of the host rock could crack the unreinforced shotcrete and create the potential for falling. Such locations were defined by regular joint OR/Strabag inspections.

3 Because of invert heave some damages to the shotcrete occurred. The damage was repaired before the invert concrete operations got close to the area, by removing the damaged shotcrete and rock and backfilling with new shotcrete. In some cases additional 6m long fully grouted bolts were also installed.

4 In the first 300m of tunnel at the Intake end, to control water seeping from the river bed into the tunnel roof, a 7×8m pilot tunnel was driven and the surrounding rock was grouted before the TBM arrived. As a result, during the TBM drive in this portion of the tunnel there was no issue of excessive seeping water in the permeable rocks of the Lockport and Decew formations.

5 Two way traffic in a circular tunnel with 24/7 logistics needs was another of the project’s challenges. Several elevated passing lanes were deployed to allow vehicles to pass and turn around. In the design of some of the carriers, emergency vehicle parking and space for passing were added. The traffic logistics were exacerbated by the additional unexpected overbreak repair activities which also required supply of material.

6 To overcome the logistical difficulties associated with the delivery of fresh concrete to the Invert and Arch carriers, and in coordination with City officials, four concrete drop shafts were utilized. The first drop shaft which was located within the outlet site property and had no effect on the City roads, was at the tunnel low point (CH 1+411) dewatering shafts. Other locations were at CH 3+369, 5+318 and 8+002. To deliver concrete and shotcrete for crown restoration for the last half kilometre of the tunnel, a concrete drop structure at the Intake portal was also utilized. Material was trucked to the point of use from each of the drop shaft locations. The low point and CH 5+318 shafts were driven with larger diameters to accommodate fresh air supply duct into the tunnel for ventilation.

7 TBM, back up unit and some of the other carriers and bridges with an approximate total weight of 10,000t of steel were dismantled at the intake cut and lifted to the surface by use of 500t and 250t crawler cranes. The Arch and grouting carriers were driven back through the tunnel and dismantled at the outlet cut to facilitate earlier flooding and removal of the cofferdam at the Intake area.
ACKNOWLEDGMENT:

The authors would like to thank Ontario Power Generation (OPG) for their permission to publish this paper.

REFERENCES:


All figures are courtesy of Ontario Power Generation.