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# Overcoming Rock Squeeze and Overbreak in a large Diameter TBM Excavation

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ABSTRACT: The Niagara Tunnel Facility Project in Ontario, Canada is an upgrade of an existing hydro power plant, featuring a 10 km long diversion tunnel with an excavated diameter of 14.4 m. It was advanced with the largest hard rock TBM at the time through horizontally bedded layers of sedimentary rock (dolomite, sandstone and mudstone), which are characterized by a wide range of strength and anisotropic stiffness as well as by a pronounced time-dependent deformation behaviour. The in-situ stress field is marked by an exceptionally high sub-horizontal stress which is several times greater than the overburden pressure. In some areas the ground has a considerable swelling potential. This, in combination with an unfavourable jointing, resulted in overstressing of the rock mass and significant overbreak which accompanied the TBM advance over considerable lengths. The overbreak, extending up to 5 m above crown, required backfilling in order to restore a circular geometry and adequate bedding conditions for the later installation of a pre-stressed unreinforced inner concrete lining. The combination of difficult geotechnical conditions and unusual structural dimensions posed an unprecedented challenge to design and construction. The paper addresses the underlying geotechnical mechanisms and the solutions adopted to overcome these phenomena encountered during excavation and the later installation of the pre-stressed inner lining.

## 1 Project description

Ontario Power Generation Inc. (OPG) is constructing the Niagara Tunnel Facility Project (NTFP) to transfer water from the Niagara River to the existing Sir Adam Beck Generating station. The tunnel facility will transfer 500 $m^3/s$ of water from the river to the headpond of the existing generating facility. To accommodate this flow, an internal tunnel diameter of 12.6 m to 12.8 m diameter is required.

The water from the Niagara River will be withdrawn upstream of the waterfalls. The 10.15 km long water diversion tunnel will connect the intake and outlet structure and, by doing so, will pass under the city of Niagara Falls. Downstream of the outlet structure, an outlet canal of approximately 400 m length and up to 30 m depth will connect to the existing feeder canal. In order to minimise the impact of tunnelling on the surface infrastructure, the tunnel was excavated by a 14.4 m diameter open-type gripper Tunnel Boring Machine (TBM).

To avoid geologically and geotechnically difficult conditions in the buried St. David’s Gorge, the tunnel gradient descends to a depth of approx. 140 m below the ground surface and passes under the gorge within the Queenston Formation.


**Figure 1. Project site orientation**

A two pass lining ensures the specified 90 year life time with zero maintenance requirement. The initial rock support is installed a short distance behind the excavation face. The final lining of unreinforced concrete is installed later and far behind the excavation. To cope with the high swelling potential of the Queenston Formation and the aggressiveness of the ground water, the hydraulic communication and seepage between tunnel and rock mass had to be prevented and the final lining protected by a waterproofing membrane system. The final lining will be externally pre-stressed by grouting the interface between waterproofing membrane and initial lining to create a compressed concrete support ring, which is able to sustain internal water pressure without steel reinforcement (Grunicke and Ristić 2012).

The tunnel was driven from the Outlet towards the Intake. The overall configuration of the works is shown in Figure 1.

## 2 Geological Setting

The geology of the Niagara Region consists of sedimentary rocks comprising dolostones, dolomitic limestones, sandstones and mudstones that are locally referred to as shales. The different units are separated by primary bedding planes which occur at spacing of about 5 m to 20 m. Primary bedding planes may be sheared, slickensided and filled with clay.

The tunnel traverses the Lockport, DeCew, Rochester, Irondequoit, Reynales, Neahga, Thorold, Grimsby, Power Glen and Whirlpool rock formations. The majority of its length, however, is located in the Queenston shale, which consists of silty claystone to clayey siltstone.

The tunnel penetrates all the upper formations while descending from the Outlet, under the buried St. David's Gorge and rising again towards the Intake Structure, see Figure 2. The Queenston Formation is a moderately hard and dense mudstone. It is sensitive to the abrasive action of site traffic and erosive action of flowing water. The formations above the Queenston consist of very hard and dense to moderately hard and dense rocks depending on the sandstone content of each formation.

In the various formations above the Queenston mudstone, the unconfined compressive strength ranges from 12 to 242 MPa. In the Queenston Formation, strength values vary between 8 to 118 MPa, with average values between 38 and 45 MPa. Deformation parameters have high values with Young's Moduli from 10 GPa for mudstone to 50 GPa for the sandstone and dolostone.

Very little groundwater was encountered throughout most of the tunnel drive. Water entering the tunnel excavation was observed along the primary bedding planes separating the formations. Significant quantities of groundwater were encountered in the last few hundred meters of rising tunnel grade in the Lockport Formation near to the Intake. The groundwater is rich in chlorides and locally in sulphates.

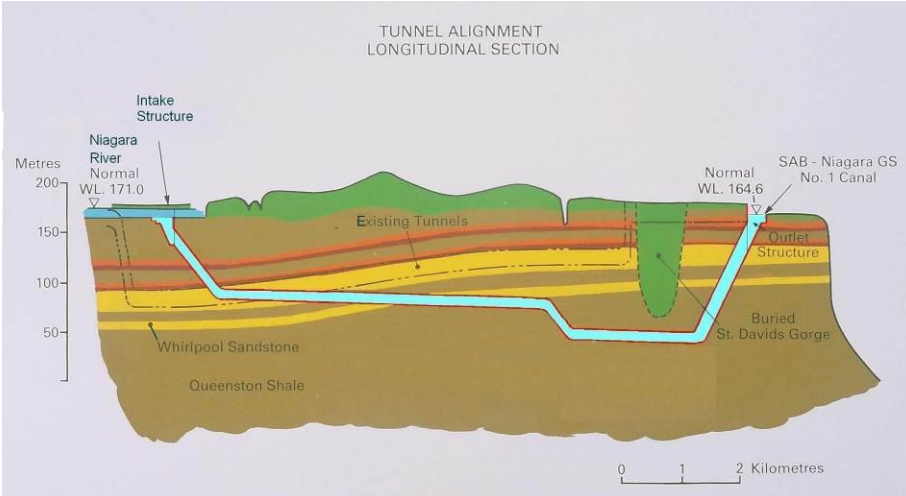


**Figure 2. Niagara Tunnel profile**

The sedimentary rock strata in the Niagara region are characterised by relatively high in-situ 'locked-in' horizontal stress, with a maximum horizontal vs. vertical stress ratio varying from 3 to 5 in the Queenston Formation and 7 to 8 in the overlying rock.

The stress field is reported to be relatively consistent trending in a north-eastern direction. However, the magnitudes of stress and the direction of the maximum stress can vary significantly depending on lithology, depth and topographical features. In the section between the Tunnel Outlet and St. David's Gorge, in situ testing showed that the horizontal stresses are 30% above the calculated vertical stresses. The in-plane in situ stresses increase as the tunnel proceeds upstream due to the tunnel orientation relative to the regional stress regime.

Rock squeeze, caused by the gradual recovery of locked-in strain energy, is predicted to take effect as time dependent lateral load on the tunnel support.

During the project development stage, a number of geological investigations including the excavation of an adit with 12 m diameter trial enlargement in the Queenston Formation were carried out. Based on the data obtained in the investigation and observation in the trial enlargement, the behaviour of the rock mass around the tunnel excavation is seen to be governed by the presence of relatively high in-situ horizontal stresses and primary bedding planes. Primary bedding planes were expected to promote delamination, crown slabbing and instability where they are located in the immediate vicinity of the tunnel crown. Below the contact with the Whirlpool Sandstone and below the St. David's Gorge is a weaker zone of rock with sheared bedding planes and increased slabbing potential.

In the Queenston Formation, tunnel crown, sidewalls and invert are all affected by slabbing and spalling if left unsupported, in particular close to primary bedding planes. The possibility of 3 m thick crown slabbing and 1 m thick sidewall spalling were predicted. The formations above Queenston can be affected by block failure along discontinuities mainly if left unsupported.

## 3 Excavation and Support

The TBM was assembled and launched in a 300 m long and up to 30 m deep canal at the tunnel outlet. The TBM consists of main body, cutter head and grippers. Roof and finger shields are installed behind the cutter head to protect the working personnel and plant. The TBM as such is only about 25 m long; however a 140 m long backup system is required to manage all the logistics for tunnel excavation and support installation. The broken ground is conveyed to the TBM backup in the form of loose chips.

### 3.1 Excavation Support Installation

The arrangement of mechanical and electrical parts on the TBM allows for two locations where rock support may be regularly installed.

The position L1 is situated just behind the cutter head and finger shield of the TBM, between 6 m and 12 m back from the excavation face (see Figure 4). From this location, steel ribs can be installed in the form of light channels to the crown of the tunnel cross section. Steel wire mesh and rock dowels can also be installed at this location, where support measures are not damaged by the gripper loads exerted to the side walls of the tunnel. Shotcrete placement is possible from L1 to a limited extent. Sensitive mechanical and electronic components of the TBM, situated in L1 may be affected by dust and rebound material.

In general, support installation is carried out in several stages at two different locations in the TBM and backup system. The ground support required immediately behind the cutter head at L1 depends on the prevailing ground conditions. However, any support installed at L1 adversely affects the rate of tunnel advance; therefore any ground support not needed immediately behind the finger shield was installed from a working platform at position L2 some 40 m back from the excavation face.

Full support of the bored tunnel is in place at L2 with full round placement of shotcrete as well as other complementary activities.

Rock dowels served as rock reinforcement being installed across bedding planes and shear planes and improving block stability. In combination with steel ribs, wire mesh and shotcrete, loosening of rock as a result of discontinuities, slabbing and spalling can be controlled. Up to 6 m long rock dowels were drilled from the L1 location, just behind the TBM cutter head. Additional rock dowels to complete the systematic rock support were installed from the working platform situated between approximately 20 m and 40 m from the excavation face.

The initial design called for steel ribs (C-channels and H-beams) to be installed at regular intervals full round or only in the crown of the tunnel just behind the TBM cutter head with mechanical erectors. When installed only in the crown in form of C-channels, they distribute the load and therefore the support resistance of the rock dowels. Steel ribs installed full round against the excavated rock provided resistance against rock squeeze and enabled completion of the shotcrete placement at L2, where rock mass loosening could otherwise occur between the L1 and L2 positions on the TBM.

Steel wire mesh panels were installed behind the steel ribs in the crown. This was primarily to protect personnel against loosened rock and to reinforce subsequently installed shotcrete.

Shotcrete was considered the main supporting element of the initial lining. It was preferably placed at L2 using robots. Manual placement of shotcrete was possible in L1 in case steel support alone was insufficient to avoid progressive rock mass loosening. When placed full round, shotcrete is designed to resist invert heave, sidewall spalling and slabbing and all other short term design loads which may develop until the final lining is in place. In addition, shotcrete serves as a sealing layer to protect the ground from drying out or the abrasive action of water.

### 3.2 Support Types

Depending on the ground conditions, it was aimed to gradually increase the amount of support in a way that the reduction of tunnel advance rate is minimized, but the required level of safety for personnel working in the tunnel is always maintained.

In order to allow for gradually increasing support installation in a systematic framework, six principal support types were specified in the design. These were refined during tunnel driving.

Support Type 1 was proposed for use in stable rock conditions of Lockport and DeCew Formations. It consisted mainly of a sealing layer of 50 mm thick shotcrete reinforced with steel wire mesh.

Support Types 2 to 4 consisted of a more and more condensed pattern of rock dowel support and the use of steel channels and wire mesh at L1. A gradual increase in support resistance corresponded to an increased size and frequency of rock wedges which would otherwise cause overbreak along discontinuities behind the L1 position on the TBM. The support resistance was completed in L2 with reinforced shotcrete and additional rock dowels.

Support Types 5 and 6 were developed to deal with overbreak due to sidewall spalling, invert heave and slabbing and to provide resistance in L1 to squeezing rock mass condition expected in the Queenston Formation with rock pressure generally exceeding rock mass strength. Both types included full round steel ribs and rock dowels within L1. In addition, Support Type 6 allowed shotcrete placement immediately in L1 to provide full coverage of the excavated rock surface behind the TBM cutter head. Additional placement of shotcrete in L2 was proposed to increase support resistance.

## 4 Construction Challenges

Some 80% of the original diversion tunnel alignment was situated in the Queenston Formation, a sequence of silty claystone to clayey siltstone.

Within the first kilometre of the tunnel, the geology proved to be highly variable and challenging. Particularly difficult tunnelling conditions were encountered from about ch. 0+806 m, which was initiated by a significant overbreak in tunnel crown at the boundary of the Queenston Formation.

The geotechnical conditions at the time of the overbreak were characterized by gravity induced structural failure of the rock mass in the crown due to joints and reduced horizontal stress in the vicinity of the St. David's Buried Gorge. The crown overbreak exceeded 3 m over the roof shield and propagated up to the bedding plane between competent Whirlpool sandstone and Queenston Formation.

Steeply dipping and slickensided, persistent natural discontinuities oriented sub parallel and sub perpendicular to the tunnel axis led to the formation of large rock blocks that terminated at the overlying lithological contact with the Whirlpool. The rock mass behaviour during the tunnel drive of the next approximately one kilometre was characterised by a structurally controlled failure mechanism (slabbing at weak bedding planes and key blocks) causing severe rock mass loosening of 3 m and more. Rock mass failure occurred within less than 6 m of the excavation face and in front of the location L1, which was the earliest proposed location for placement of rock support.

Within the stretch of tunnel following the passage of St. David's Gorge up to ch. 8+600 m and beyond, the induced horizontal stresses exceeded the rock mass strength in some places. Failure of the rock mass above the cutter head as well as stress induced failure of the invert was observed. Overbreak of 3 m and more over the tunnel crown (see Figure 5) and loosening of up to 1 m depth below the invert were caused by the unfavourable geotechnical conditions within the Queenston Formation in this tunnel section. Deformation had already occurred by the time the rock mass was exposed at the L1 position. As a result of insufficient stand-up time, deep notches developed almost immediately after excavation, although ongoing convergence and squeezing pressures were measured and observed for months following advance.

## 5 Modifications due to tunnelling conditions

Following the initial overbreak at ch. 0+839 m, the tunnel drive was paused as the conditions became inappropriate for safe operation of the TBM.

As a result of the limited stand-up time, the progressive rock mass loosening up to 3 m above the tunnel crown and beyond (which was the limit of reach for the support installation equipment in L1) had to be controlled. Later pre-stressing of the final lining requires good bedding conditions. The loosened rock mass was therefore removed and replaced by shotcrete.

Based on operational and geotechnical considerations new support types were developed to deal with tunnelling conditions described above.

The additional Rock Support Type 4S was designed to cope with the conditions that caused rock mass loosening in the crown of the tunnel to a depth of more than 3 m from the theoretical excavation profile in L1. Since natural rock arching could not be expected to limit the overbreak to 3 m, a support ahead of and above the cutter head and roof shield was required to keep large blocks in place and prevent further rock mass relaxation and overbreak.

This goal was achieved by placing steel pipes in the form of a "spile umbrella" above the cutter head of the TBM. The spile umbrella consists of 20 to 40 pieces of 9 m long pipes. Rock dowels, steel channels, wire mesh and shotcrete, installed at L1, secured the spile umbrella in place while allowing further advance of the tunnel excavation. While limiting the magnitude of the overbreak occurring above the spiles, the material below the spiles loosened and had to be removed, see Figure 3.


**Figure 3. Spiling in area with overbreak, Support Type 4S**

In areas with stress induced failure, where overbreak was gradually developing up to 3 m, a different approach was adopted. After removing the loose rock, flexible steel channels, rock bolts and wire mesh were installed in the crown and at the haunches, and shotcrete was placed from L1 by handheld nozzling (Support Type 4R). In order to prevent progressive failure as a result of crown slabbing, shotcrete and rock dowels were placed as close to the TBM cutter head as possible, see Figure 4.

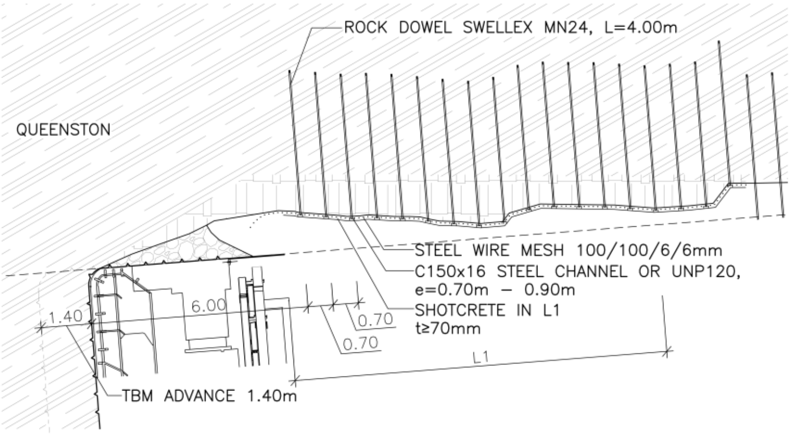


**Figure 4. Rock Support Type 4R**

A third additional Rock Support Type 4T included a slot in the shotcrete lining in order to allow for delayed horizontal deformations without spalling. Longer rock dowels were placed at the haunches (i.e. the convex shapes of the overbreaking profile), aiming at minimizing further progressive propagation of the overbreak.

To facilitate the spiling operations as well as rock support within the areas of excessive overbreak, mostly beyond the reach of the original equipment, several modifications were carried out in the L1 area of the TBM. The new support types, 4R, 4S and 4T, included the placement of shotcrete immediately above the cutter head. The installation of spiles at an angle of approximately 5 degrees from the tunnel required an additional drilling boom behind the cutter head. The L1 area was equipped with two telescopic work platforms to provide access to the overbreak areas as well as for hand sprayed shotcrete.

In order to partially mitigate the impact on the construction schedule, the horizontal and vertical alignment of the tunnel was changed by minimizing its length within the Queenston Formation. This has shortened the tunnel alignment by 200 m with benefits to the diversion capacity of the tunnel.

## 6 Restoration of overbreak

To enable successful pre-stressing of the unreinforced final concrete lining, the main targets of the restoration works were to establish a circular shape of the tunnel cross section, reinstate the stiffness of the rock mass around the tunnel lining and to reduce the permeability or hydraulic conductivity of the loosened zone around the tunnel.

The restoration works were carried out from various platforms termed “restoration carriers”. These carriers were fitted with drilling rigs, working platforms and shotcrete arms. They were deployed after the invert concrete installation.

Prior to restoration works, water pressure tests were carried out to evaluate the rock mass permeability. Where loosened rock mass was identified, cavity grouting was undertaken prior to commencement of filling of overbreak.

Overbreak with volumes not exceeding 15 m³/m and depths less than 2 m were filled with shotcrete, which was placed in 300 mm thick layers and secured with a grid of rock dowels.

Larger overbreak, exceeding 15 m³/m and referred to as Type 2, was backfilled behind a suspended formwork, which followed the original circular tunnel shape. The formwork was held in place with suspension bars connected to a grid of vertical rock dowels in the tunnel crown, see Figure 5.


**Figure 5. Installation of fill of overbreak Type 2**

After installation of the prefabricated steel elements of the formwork, additional reinforcement bars and expanded metal sheet were placed on the elements. To shape the final formwork, capable of carrying the weight of backfill, shotcrete was placed from the underside of the suspended steel elements.

## 7 Geotechnical mechanisms

The ground reactions encountered at the Niagara project were characterized by several failure modes and combinations thereof. The high in-situ sub-horizontal stress resulted in overstressing of the rock mass and consequently in brittle failure and notching. For isotropic ground masses, Martin 1999 gives an approximation for the assessment of the depth of the notches:

$$\frac{R_f}{a} = 0.49(\pm 0.1) + 1.25\frac{\sigma_{max}}{\sigma_c} \tag{1}$$

where: $\sigma_{max} = 3\sigma_1 - \sigma_3$, with $\sigma_c$ being the rock strength, a the tunnel radius and $R_f$ the depth of notch measured from the tunnel axis, see Figure 6, left.

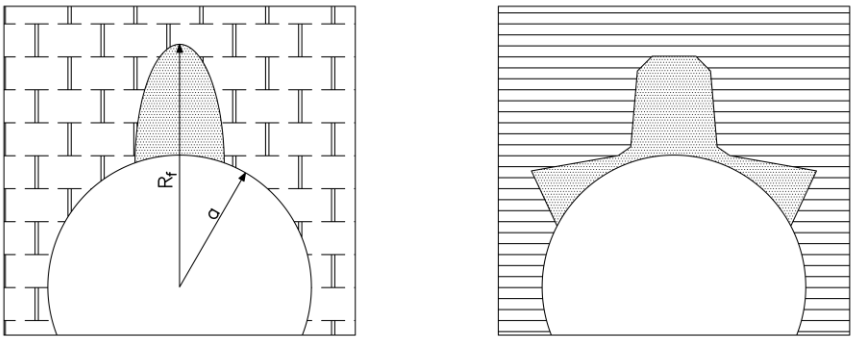


**Figure 6. left: notching in isotropic (modified after Martin 1999), right: failure in laminated rock mass (modified after Perras 2009)**

The shape of the failure is, however, influenced by the anisotropy which is inherent to the horizontally and alternately bedded sedimentary rock. Influencing factors are, amongst others, joint orientation and joint surface properties, bedding plane spacing, stiffness differences between alternating rock layers and resulting stress shadows and bedding plane strength.

Perras 2009 shows that shape and dimension of the notches are governed by the combination of rock mass failure, both within the bedding planes and the intact rock mass in between. Depending on the relation of bedding plane spacing to tunnel radius, some initial overstressing concentrates at the haunches, followed by slabbing or failure of a crown beam or chimney shaped failure, see Figure 6, right. This underlines the importance of securing the haunches by bolting, in order to minimize further gravity driven failure of the overstressed rock mass. The adapted support of the Niagara Tunnel therefore aimed at stabilizing the haunches in order to reduce further progressive propagation of the notches and the extent of rock mass overstressing.

# 8 Conclusion

Tunneling in horizontally layered sedimentary rock in combination with high horizontal stresses can lead to overstressing and notching. The phenomena encountered and the influence of in-situ stress field and anisotropy of the rock mass can be described in theory, but the actual ground response in-situ depends on a range of influencing factors such as fracture density, joint orientation (see Fig. 6) and joint properties, the presence of water and the size of the tunnel excavation.

The phenomena are generally independent of the excavation method and independent of the TBM type (shielded TBM or open gripper TBM). A shielded TBM with the immediate installation of a segment ring may provide good initial working safety against small scale wedge failures, but the bearing capacity of a segmental ring is limited. While immediate support in the L1 area of an open face TBM may involve time consuming efforts (see Fig. 3 and 4), it provides good flexibility for the adaptation of support measures such as selective bolting of haunches.

Support measures cannot be designed exclusively based on geotechnical desk-studies. At the NTFP, constant observation and monitoring and the interpretation of ground and support response is of utmost importance in order to identify any localized failure trend and in order to adapt or reinforce support measures.

# 9 Acknowledgements

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