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SEEPAGE CONTROL DURING TUNNEL DRIVING UNDER LAKE HURON

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ABSTRACT

The objective of this paper is to provide a case history on rock tunnelling beneath Lake Huron, Ontario, Canada. The project under review consisted of the construction of an 8.7 m diameter cooling water intake tunnel for the Bruce B Nuclear Generating Station owned and operated by Ontario Hydro.

Continuous consolidation grouting of the tunnel was carried out to control water inflow during excavation. A summary of design parameters and construction procedures is given.

KEY WORDS

Intake tunnel, seepage control, grouting, tunnel support, excavation.

INTRODUCTION

The Bruce Nuclear Power Development is located on the east shore of Lake Huron, 300 km northwest of the City of Toronto, Ontario, Figure 1. When completed, it will be one of the largest energy complexes in the world. Together, the two generating stations will be capable of producing 6400 MW of electricity. Construction of the Bruce B generating station began in 1976. The last of the station's four units is slated to come into service in 1987. Figure 1 shows the principal components of the project.

A concrete lined tunnel excavated beneath the lake forms a part of the station's cooling water system which includes the intake channel, forebay and pumphouses. A required 193 m^3/s cooling water flow is conveyed into the station via the intake system and then returned to the lake through an open discharge channel.

TUNNEL ARRANGEMENT

The tunnel profile is illustrated in Fig 2. A brief description of each tunnel component follows:

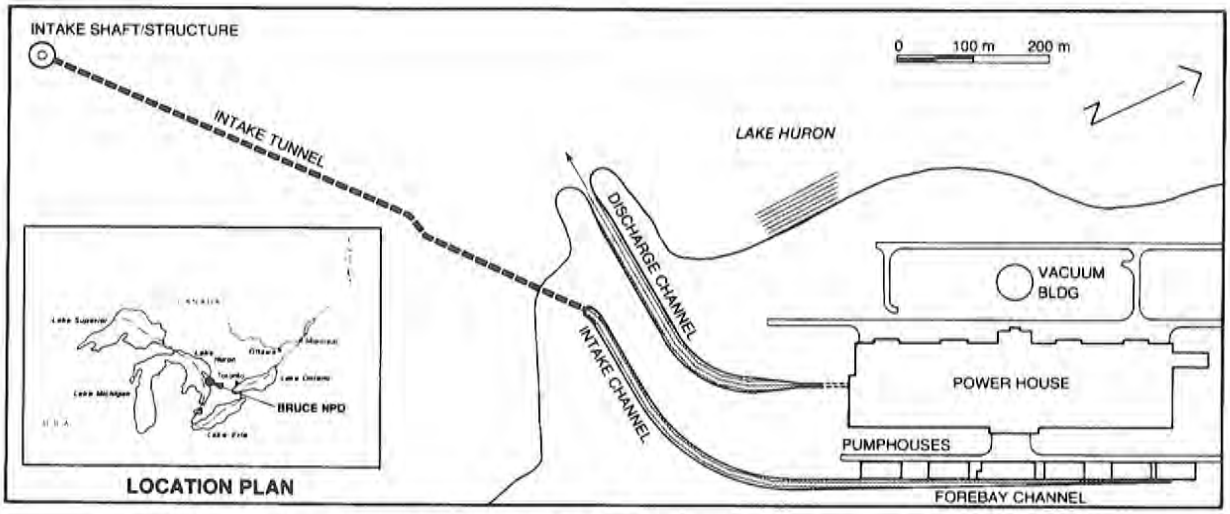

Fig.1. Project location and layout

- Offshore Intake Shaft. The 30 m deep shaft is circular in shape with a diameter of 9.5 m. A rock plug, 14 m thick, at the top of the intake shaft, was left in place until the tunnel lining was completed. After tunnel flooding, the upper 10 m of the plug was blasted and removed by marine operation. The lower 4 m of the plug was allowed to slump into the sump below.

- Horizontal Tunnel. A 567 m long, near horizontal tunnel slopes down from the intake shaft to the outlet ramp at a grade of 0.25 percent. The tunnel has vertical sidewalls and an arched roof with a span of 8.7 m, Fig. 3.

- Outlet Ramp. The outlet ramp (235 m long), slopes down at a grade of 14 percent from the outlet transition to the horizontal tunnel. The cross-section is the same as for the horizontal tunnel.

- Outlet Transition. The outlet transition constitutes a 27 m long transition between the tunnel and the forebay. Its diameter increases linearly from 8.7 m to a maximum of 13.7 m at the portal face.

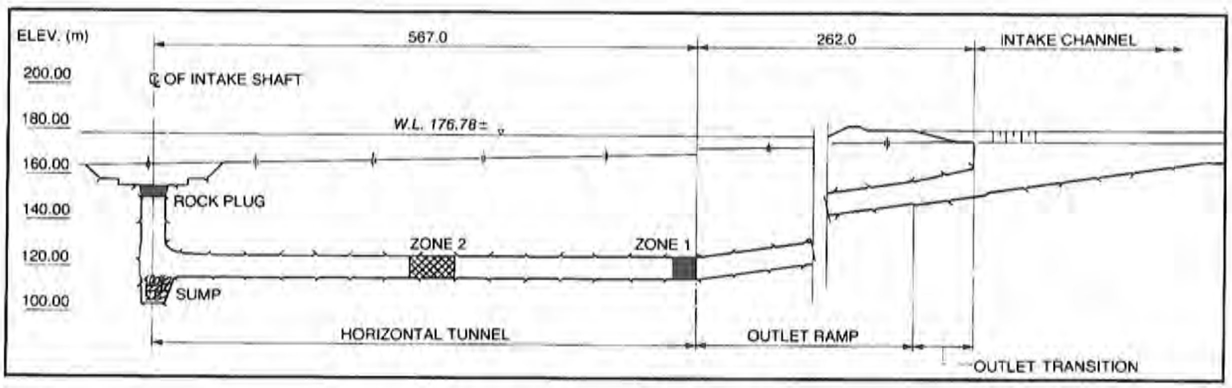

Fig.2. Tunnel profile

DESIGN CONSIDERATIONS

The primary design consideration was that the tunnel should be aligned to minimize its length, while also satisfying hydraulic and geotechnical requirements. Furthermore, for the given alignment and length, the tunnel diameter was to be based on the minimum sum of construction cost and capitalized cost of pumping over the life of the plant.

The selection of the length and horizon of the tunnel was based on the following criteria:

- to locate the intake at a sufficient lake depth (minimum 15 m) to ensure an adequate supply of cold water of reasonably constant temperature to the generating station and to prevent ice clogging of the intake structure as well as to minimize the entrance of fish;

- to maintain an adequate distance between the intake and the discharge channel so that recirculation of heated water is avoided;

- to place the tunnel within a massive rock unit requiring only minimal rock support, permitting efficient excavation and avoiding excessive water inflow.

PREDICTED TUNNELLING CONDITIONS

The interpretation of the results of the geological investigation resulted in positioning the horizontal portion of the tunnel within the bedded sedimentary rock, which is a Coral Limestone of the Paleozoic Age. The general condition of the limestone was termed as structurally sound. A brecciated, folded syncline was to be expected within the middle portion of the horizontal tunnel, Fig. 2, Zone 2.

A 50 m thick rock cover above the tunnel consisted of a series of dolomite beds of variable thickness and quality. The rock cover thickness along the ramp decreased from 50 m to 12 m. It was also recognized that the bituminous layers present within the horizontal beds would control breakage resulting in sections of flat roof in the ramp.

At least three sets of very pronounced vertical joints were identified in the inclined drill cores. The results of water pressure testing as well as experiences during the driving of the Bruce A tunnel (constructed a few years earlier), indicated that open rock conditions can be anticipated at various depths to well below the tunnel grade. These tests showed that an appreciable amount of seepage into the tunnel excavation was to be expected and that consolidation grouting would be required along the whole length of the tunnel.

TUNNEL SUPPORT REQUIREMENTS

The initial design of the rock support was based on the span of the tunnel and its orientation relative to rock structure. It was generally assumed that the roof shape would be controlled by horizontal bedding planes and that the shape of the side walls would be controlled by the vertical joints.

The specified rock support consisted of temporary, provisional and permanent systems. Temporary support, consisting of a pattern of ungrouted bolts 2.5 m long at 1.5 m spacing, was used in the tunnel proper. Figure 3 shows the rock bolt pattern in relation to the grout curtain. Provisional support consisted of grouted rock bolts and shotcrete. Its main purpose was to stabilize isolated areas where the temporary rock support was inadequate due to the presence of less competent rock. The permanent rock support was intended to support larger spans of the tunnel, such as the intake transition. It consisted of grouted rock bolts and concrete or shotcrete lining.

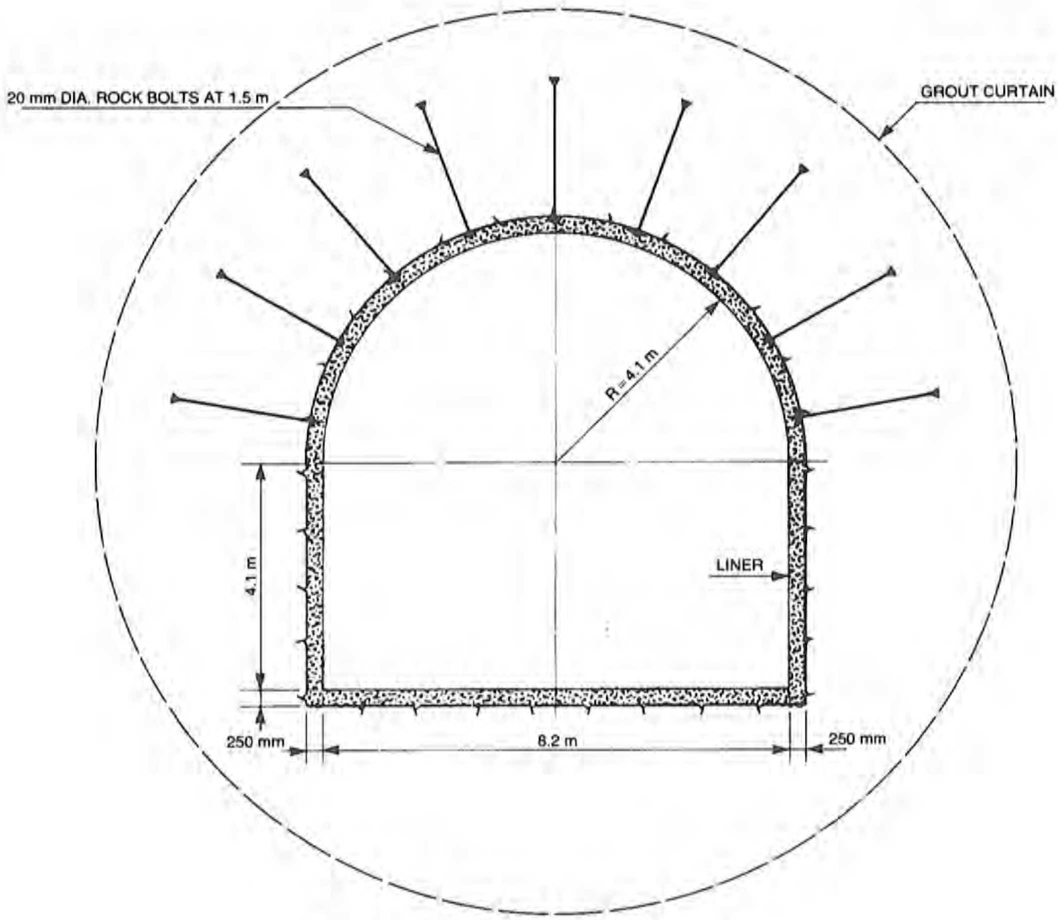

Fig.3. Tunnel cross section

TUNNEL LINING

The primary role of the lining was to reduce hydraulic head losses. Its secondary purpose was to maintain the integrity of the rock structure during operation, thereby limiting the risk of ravelling. Three proposals were studied to find the alternative with the lowest total of construction and capitalized operating cost. It was determined that the concrete lining scheme had the lowest total cost. The unlined and shotcrete-lined alternatives were 10 percent and 20 percent more expensive, respectively because of the hydraulic and construction factors.

The lining within the horizontal tunnel and outlet ramp consisted of 250 mm thick, non-reinforced concrete, Fig. 3. It was thought that the outlet transition with its flared shape would tend to be more likely to suffer blast damage. Hence, the lining in this portion of the ramp was designed to carry full rock load, resulting in a 700 mm thick reinforced concrete liner. The intake elbow and transition were lined with 140 mm thick shotcrete.

CONTRACTING PROCEDURES

Pre-qualification of Contractors

Potential contractors for Ontario Hydro tunnel projects must participate in a pre-qualification process. By this process, contractors with suitable experience, expertise and financial basis can be determined at an early stage. Another advantage with the pre-qualification process is that interested contractors are advised of an upcoming project in a timely manner, allowing sufficient time for proper planning.

Based on the preliminary project description, the contractors were required to provide detailed information on experience, proposed work methods, equipment, schedules, interpretation of geological conditions, proposed quality assurance program as well as financial statements.

Disclosure of Geotechnical Information

The factual results of the geotechnical investigations and their interpretation were included in the Tendering Documents. Ontario Hydro accepted responsibility for the factual information but not for its interpretation.

Provision for Seepage Control

The geological investigations indicated that water control would be a major part of the construction work. To control seepage during excavation, a provision was made in the Technical Specifications for feeler hole drilling and consolidation grouting. It was estimated that one-third of the excavation time would be spent on grouting.

An 11 m^3/min inflow provision was included in the unit price of the excavation. Seepage in excess of this was to be treated as an additional pay item.

CONSTRUCTION ASPECTS

Tunnel Excavation

Excavation of the tunnel was done by the drill and blast method. By this method, excavation was advanced by a continuous series of cycles. Normally the excavation cycle consisted of the following steps, Fig. 4.

(a) Grouting, to control seepage, was advanced a minimum of 30 m beyond the tunnel face. After grouting, excavation was advanced 24 m. This provided 6 m of grouted rock beyond the tunnel face.

(b) Excavation was done by full face blasting. The number of rounds of advance before installation of rock support was dependent on rock conditions and stand-up time.

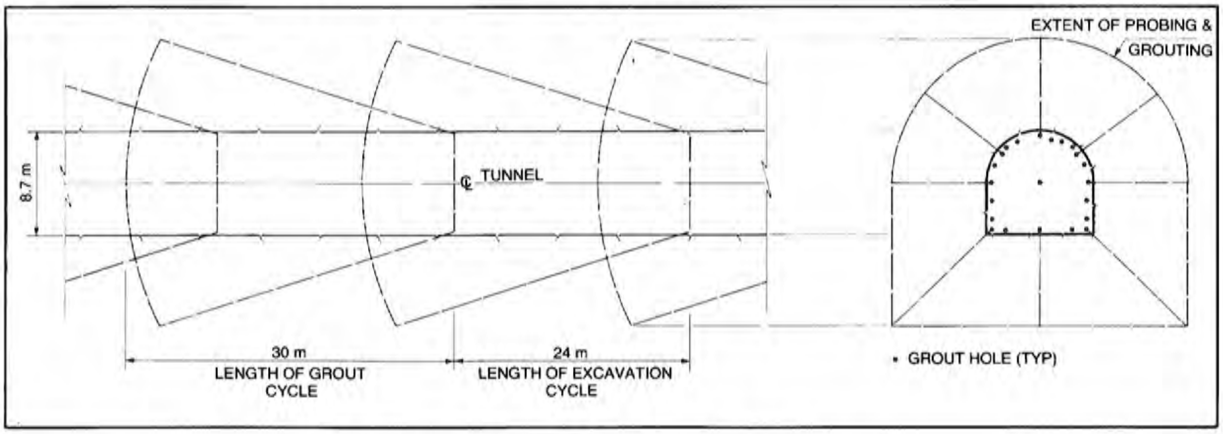

Fig.4. Excavation and grout cycle

Excavation of the 13.7 m wide portal face was done by a two-stage blasting method. Initially, a large, centrally located pilot, approximately 7.5 m wide, with short blast round lengths, was excavated. This was followed by the slashing of the perimeter to the final design line, Fig. 5. The two-stage method provided the advantage of utilizing low explosive loading per delay. This resulted in minimizing damage to the rock in the portal face.

Once the blasting method was perfected and after the tunnel had advanced beyond the ramp transition, full face blasting rounds were used, Fig. 6. For this tunnel, typical blast rounds consisted of approximately 115 holes per round drilled to depths that varied from 3.5 to 4.5 m. The blast pattern consisted of a parallel burn cut followed by production, buffer, perimeter and lifter holes. Drilling and blasting of the intake shaft was initiated by the use of Alimak raise climbers. The shaft was benched down in approximately 2.5 m lifts.

Fig.5. Portal face excavation

Fig.6. Full face excavation

Summary of Observed Seepage into Tunnel Excavation

Water seepage into the tunnel originated from various sources. These were bedding planes, sub-vertical joints, permeable beds and from holes drilled for rock bolts. Water seepage was minimized by grouting, while excess water was pumped from the tunnel via a weir box on the ground surface. A graphical plot of the rate of post-grouting water inflow into the tunnel vs tunnel chainage is shown on Fig. 7. Examination of this plot indicates a maximum pumping rate of approximately 7.5 m^3/min which corresponds to approximately 0.008 m^3/min per metre of tunnel.

Prior to grouting, estimates of the water seepage were made for each drilled grout cycle, resulting in a cumulative total of 112 m^3/min for the full tunnel length. Pressure grouting was successful in reducing this seepage by about 93 percent.

Chemical Grouting

In the initial stage of the project, an attempt was made to seal the finer water bearing fissures with chemical grout. Grouting was attempted with dissolved calcium chloride and diluted sodium silicate. These two chemicals combined in a reaction to form a gel which was to seal the fine fissures in the micro fractured rock. However, after a trial attempt, it was concluded that the method was not satisfactory and that cement grouting would produce better results.

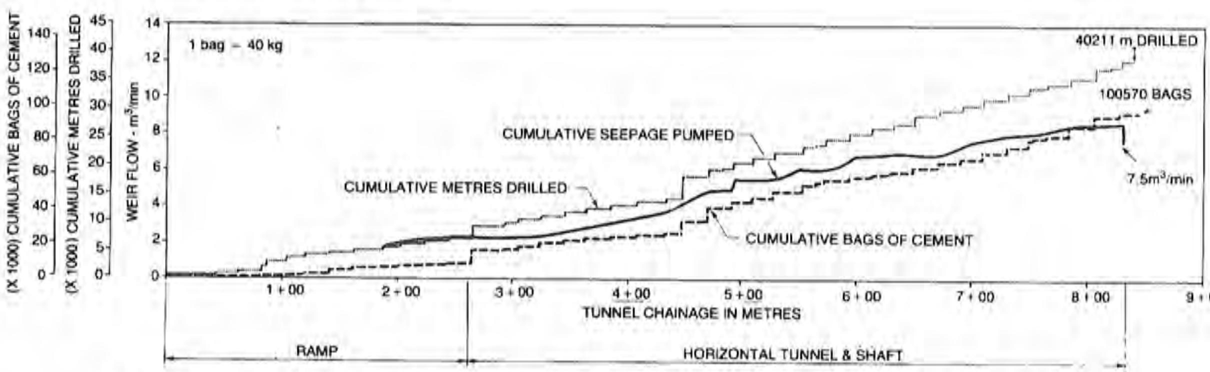

Fig.7. Summary of drilling, grouting and pumping quantities

Drilling and Cement Grouting

The contract specifications required the drilling of a minimum of two feeler holes ahead of the tunnel face prior to each excavation cycle. The purpose of this was to determine, in advance, groundwater conditions, presence of gases and structural features within the rock mass.

Each normal grouting cycle consisted of 15 to 46 grout holes, depending on local conditions. A typical grout hole layout at the tunnel face is shown in Fig. 4. Grouting was considered to be composed of two steps: Step 1 - the design of the grout hole layout, length, number and orientation of holes; and Step 2 - the injection of a consolidation material, consisting of a neat Portland cement grout. On rare occasions additives to the cement grout such as sawdust, grain and coarse sand were used. The procedures employed during grouting are outlined below in point form:

(a) Upon drilling of grout holes by percussion methods, the holes were flushed with water and then furnished with the mechanical packers. An estimate of water discharge from the holes was made. Grout hoses were generally connected first to those holes discharging the greatest volume of water.

(b) Grouting was started with a thin mix with a water to cement volume ratio of 5. The initial grout flow was approximately $0.12\ m^3/min$. Normally, grouting pressures of approximately 1.4 MPa were used in the horizontal portion of the tunnel.

(c) The grout mix was gradually thickened if the grout flow did not decrease appreciably. If flow rates still were not decreasing during use of the thickest cement mix, sand was added. The completion of grouting of a hole (refusal) was deemed to have been achieved, when the grout flow rate was less than $0.006\ m^3$ min.

A total of 49 regular grout cycles were required to grout the entire tunnel length. Figure 8 shows water flows versus grout acceptances for all the grout cycles. Examination of this figure shows that the points are about equally distributed on either side of a line representing a seepage of $4.5\ m^3/min$ per $100\ m^3$ of grout accepted/cycle and that most of the points lie within the $\pm 15\ m^3$/cycle grout range as shown in the figure. This figure indicates that the water flow from drilled grout holes was a good indicator of the amount of grout required.

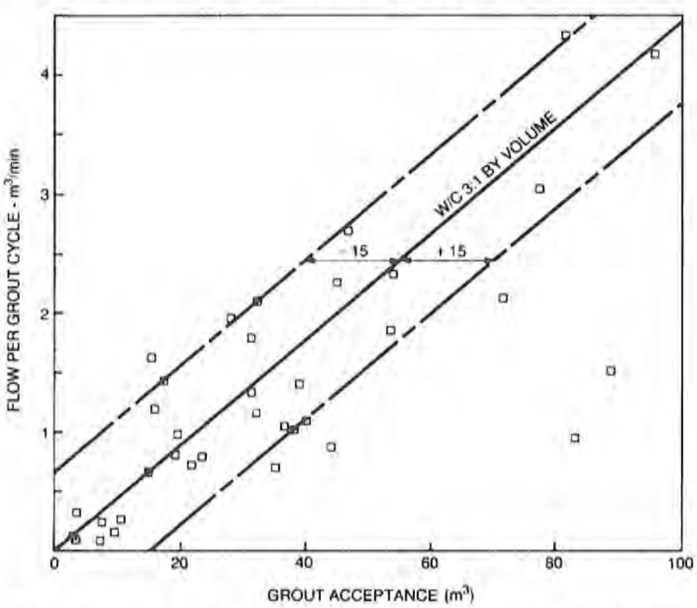

Fig.8. Grout acceptance vs water inflow

SPECIAL SEEPAGE CONTROL PROCEDURES

Normal tunnelling, consisting of grouting, excavation, and provision of rock support, was at times interrupted due to the presence of very difficult geological conditions. Such conditions were encountered in two zones, which caused the disruption of the construction schedule and required unique solutions in order to continue construction.

Zone 1 - Tunnel Diversion

A feeler hole drilled at the bottom of the outlet ramp in preparation for a subsequent excavation cycle encountered open, water bearing, rock conditions. Continued exploratory drilling conducted from within the tunnel indicated that this feature was located 12-18 m ahead of the tunnel face in the lower left corner. A definite orientation of the geological discontinuity based on drilling information could not be determined. However, exploratory drilling, both by percussion and diamond methods, indicated that rock quality ahead of the tunnel face was variable. Some of the exploratory holes intercepted open, water bearing features. The flows out of some of these holes were estimated to be approximately 3.5 m^3/min, Fig. 9.

Attempts to grout this zone after injecting about 140 m^3 of cement and sand proved unsuccessful. Some holes could not be grouted by normal methods and had to be plugged with a thick cement sand grout.

The hydraulic characteristics of the zone were evaluated by opening all ungrouted holes and then successively closing the holes while pressure within the zone was monitored. During this time, the quantity of accumulated water pumped from the tunnel face was also monitored as a function of time. From the gathered data the conclusion drawn was that there was no direct hydraulic connection with the lake above. The monitored pressure drop corresponded to friction losses as water percolated down through the fissure and jointing network toward the zone.

After review and evaluation of the exploratory findings, the grouting attempts and the hydraulic characteristics of the feature, it was decided that it would be more economical to divert the tunnel around this zone and then driving the tunnel within the same horizon towards the original location of the shaft. Supplementary work associated with the diversion consisted of additional probe drilling, consolidation grouting, provision of increased rock support and stage excavation.

Fig.9. Zone 1, flow from drill holes

Fig.10. Zone 2, major flow

Zone 2 - Fold Area

A brecciated fold zone, approximately 230 m long, was encountered in the mid-portion of the tunnel, Fig. 2. The preconstruction investigations had indicated that a poor to fair rock quality was to be expected in this zone. As anticipated, drilling for grouting and rock bolting became more difficult due to such rock conditions. The blast damage to the rock also became more prominent.

At one point, a blast hole started to produce water flow. The rate of flow gradually increased to the point where it was apparent that the blast round could not be loaded and that remedial work would have to be done. An attempt was made to control the flow by installing a packer in the hole. This caused the water to break out through an adjacent hole, producing a flow of about 1.3 m^3/min. As the flow increased, all equipment and men were removed from the tunnel and all efforts were concentrated on the pumping operation. The flow from the tunnel before encountering this feature was about 4.5 m^3/min. The total flow increased to about 22 m^3/min. This eventually decreased to approximately 17 m^3/min. Accurate readings of the new flow could not be obtained initially, since the weir box was designed to handle only about 11 m^3/min.

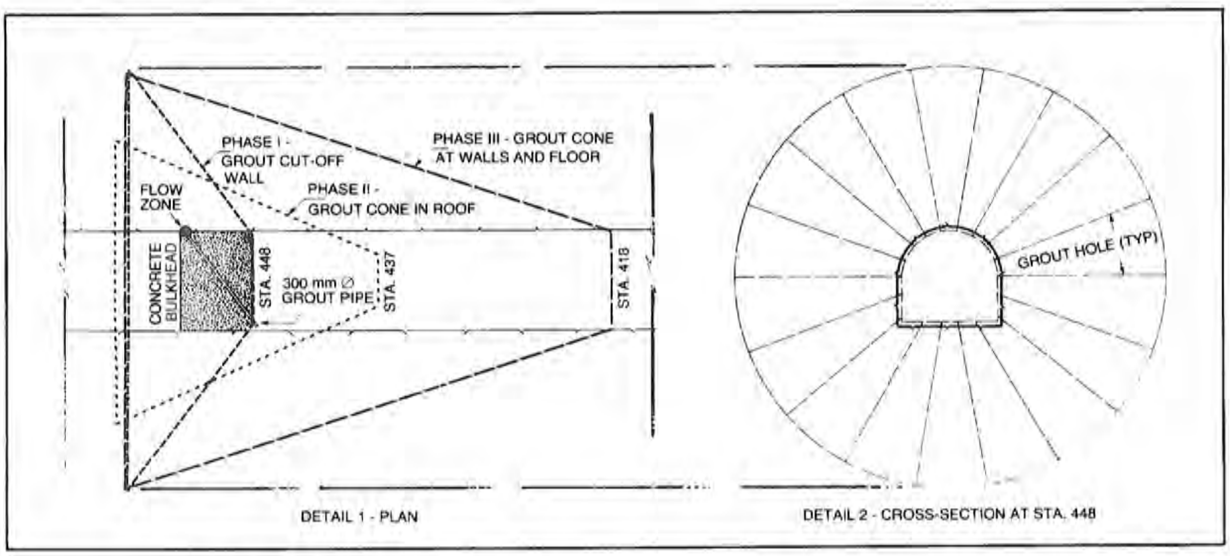

Fig.11 Zone 2, summary of remedial treatment

The inflow caused the erosion of 3 m^3 of brecciated material into the tunnel. A photo of the flow condition is shown in Fig. 10. Remedial measures to control the flow consisted of constructing a 4.5 m thick concrete bulkhead, placed directly against the tunnel face. A grout cut-off wall extending a full 360 degrees around the tunnel and a grout cone, consisting of the grouting of a series of long holes, was done to consolidate the surrounding rock. Final grouting of the major flow was made through a 300 mm pipe installed through the concrete, directly into the flow zone. A total of 240 m^3 of grout was required for grouting of this zone. The remedial works are illustrated in Fig. 11.

SUMMARY

An intake tunnel has been successfully constructed beneath Lake Huron to provide cooling water for the Bruce B Nuclear Generating Station.

An important consideration in this tunnel was the control of water seepage into the excavations. The cement grouting and seepage water pumping procedures adopted at the project were satisfactory in controlling the water flows.

The amount of encountered water inflows in the probe and grout holes was manageable within the specified construction methods. From a mining point of view, a much greater seepage quantity could have been allowed without affecting excavation efficiency. However, reduction of water inflows was necessary to facilitate the installation of the concrete liner and to provide safe working conditions.

Prior to construction, it was estimated that approximately one-third of all mining time would be related to seepage control. Upon completion of the project, it appeared that this initial assessment was valid.

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REFERENCES

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