Tunnel and Shaft Construction for the Pingston Hydro Project

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ABSTRACT: The Pingston Hydro Project is a 45 MW hydropower project developed by private sector power development companies. The project involves 3.5 km of low and high pressure tunnels, a 450 m shaft, and 1225 m of steel liner. The tunnels are at internal pressures up to 480m head, and the gross head at the powerhouse is 590 m, making the head at this plant one of the highest in North America. The paper will discuss construction of the tunnels and shaft, in-situ stress testing and determination of the steel liner length, high pressure plug design and construction and related design and construction issues.

1 INTRODUCTION AND PROJECT DESCRIPTION

The Pingston Hydroelectric Project is a run-of-river hydroelectric project located approximately 60 km south of the town of Revelstoke in British Columbia, Canada (Figure 1) on the western shore of Upper Arrow Lake. Construction commenced in March 2001 and was substantially completed in March 2003 with commissioning in May 2003. The initial project development stage was for 30 MW capacity; powerhouse expansion to increase capacity to 45 MW will be completed in May 2004. Power from the project is sold to BC Hydro as part of the Green Power Generation procurement process. The project is one of the first projects to be developed and commissioned under the BC Hydro GPG program.

Figure 1. Project area location

Pingston Creek drains from the eastern edge of the permanent icefields of the Monashee Mountains and flows southwards parallel to Upper Arrow Lake (Figure 2); it is separated from the lake by Pingston Ridge. The creek is diverted via a headpond at El. 1035 m to the powerhouse at lake level. With a gross head of 590 m (maximum 480 m head in the tunnel), the Pingston facility ranks as one of the highest head plants in North America.

Various conveyance arrangements were evaluated for the project including:

- low pressure tunnel through Pingston Ridge; long shallow buried penstock down east slope of Pingston Ridge to Powerhouse;
- low pressure tunnel through Pingston Ridge; shaft; high pressure tunnel; short shallow buried penstock to Powerhouse;
- Long inclined tunnel; short shallow buried penstock to Powerhouse.

The second option was found to offer the best combination of cost and minimization of geotechnical risk from slope instability, which was shown to be present on the east slope of Pingston Ridge.

A section through the conveyance system through Pingston Ridge is shown in Figure 3. It consists of a 1835 m low pressure tunnel (Upper Tunnel) connected to a 1450 m Lower Tunnel by a 454 m 70° inclined pressure shaft with 630 m of shallow buried penstock leading to the surface powerhouse. The 2.4 m by 2.7 m tunnels were excavated by drill and blast and the 2.1 m diameter shaft was raise-bored.

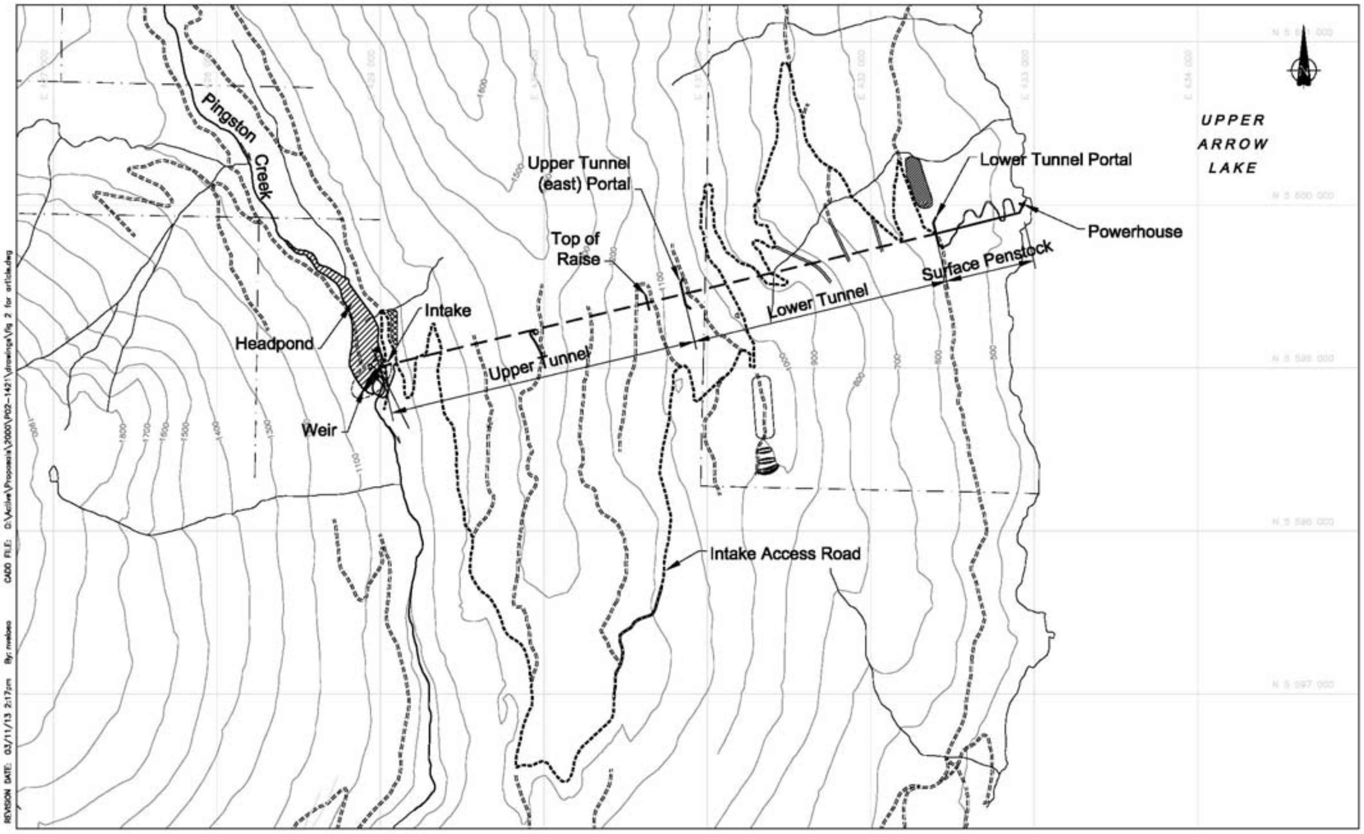

Figure 2. Plan of Project Area.

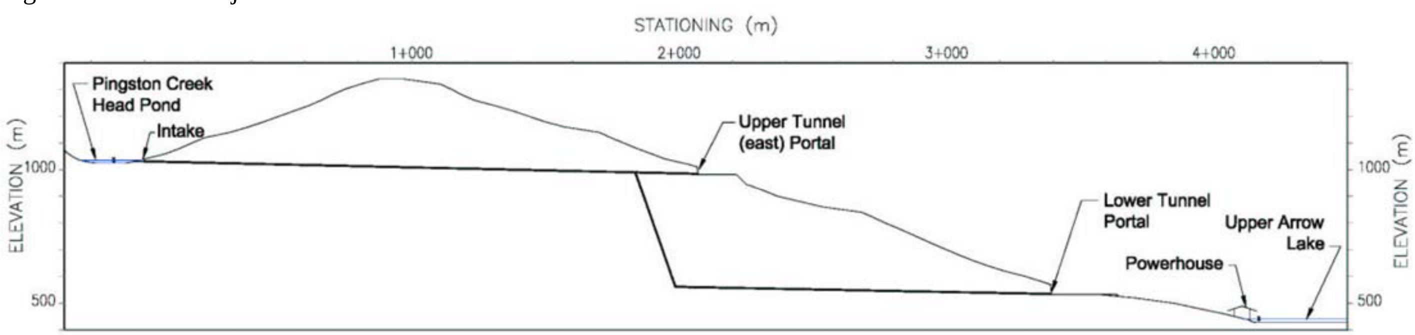

Figure 3. Section through Conveyance System.

Key issues of the project included design, construction and selection of the location of the 480 m head Lower Tunnel Plug, design and construction of the plug in the Upper Tunnel and assessment and prediction of the hydrogeological impacts of the project. Tunneling conditions proved to be variable, with poor conditions being encountered in the shaft, Lower Tunnel and downstream part of the Upper Tunnel. Conditions proved to be particularly challenging in the shaft, where substantial amounts of lining were installed following completion of raise boring.

2 GEOLOGY

The project area lies entirely within strong highly anisotropic metamorphic rocks of the Shuswap Metamorphic Complex. Rock types include quartz mica feldspar gneisses, locally schistose, with minor amphibolites and some interfoliated quartzites. Quartz veining and pegmatites are common through the sequence. Uniaxial rock strengths generally lie within the range of 50 to 100 MPa. The project lies within the lower plate of the eastward-dipping Columbia River Thrust Fault which lies along Upper Arrow Lake and is a dominant regional feature. Faults and shearing parallel to this fault are common. Similarly, the foliation in the rocks dips to the east at 20 – 30 degrees, approximately parallel to the fault.

Initial investigations, which were primarily focused towards the surface penstock option, were carried out by geological mapping along the proposed corridor with limited drilling along the alignment of the low pressure tunnel. Subsequent investigations were directed towards the Lower Tunnel with some additional deeper drilling. More detailed reconnaissance of surface features was carried out, suggesting the likelihood of shallow post-glacial sagging of the slopes dipping towards the lake. An assessment of the geotechnical risks to a surface penstock were evaluated particularly in relation to the experience of other British Columbia hydropower projects in the Columbia valley (Moore, 1999), and at Wahleach in the Fraser valley (Moore et al, 1992). It was concluded that the underground option would be adopted in order to avoid these risks. The high cost of drilling through the 400 m of cover over the pressure tunnel, and the fast-tracking required to meet the power purchase deadlines determined that further investigations, including in situ stress measurement for the steel liner design, would be carried out during construction.

From installed piezometers and surface observations, it was concluded that the water table was close to ground surface. The tunnels were expected to intercept groundwater flow from the high ground on Pingston ridge to the north of the tunnel which would be directed parallel to the strike of the main structural features towards the tunnel. Thus it was anticipated that local high inflows could be expected during construction where faults or shears parallel to the foliation, or major joint systems, were intercepted. Groundwater discharges at the surface were masked by the veneer of sagged ground of higher than average rockmass permeability.

3 CONSTRUCTION

3.1 *General*

The tunnels were excavated from three portals (Figure 3) - the Upper Tunnel was excavated downgrade from the intake at the west end (completed intake structure shown in Figure 4), and upgrade from the east portal which would be used later during operation for maintenance access. The Lower Tunnel was excavated upgrade from the outlet portal, located approximately 110 m in elevation above the powerhouse.

Construction of the Lower Tunnel and the Upper West Tunnel began in April 2001. The Upper East tunnel advance started in July 2001. The Lower Tunnel excavation was completed in late April 2002 and the Upper Tunnel breakthrough occurred in early May 2002.

The raise bore was excavated from a large cut-out in the south side of the Upper Tunnel, 250 m from the east portal. The pilot hole was drilled at 70° to the east, 445 m down to the Lower Tunnel for a period of 7 days starting March 16, 2002 and the reaming carried out over 39 days starting May 14, 2002.

Figure 4. Intake structure and headpond

The rock types and structural attitudes encountered during excavation of the tunnels were similar to those predicted. However, shear zones parallel to foliation were more extensive and groundwater inflows were higher than anticipated. Concerns prior to construction were that the surficial sagged zone either might be much deeper than the mapping suggested, or that it would be underlain by a basal shear plane. Tunnel excavation did not show any open or relaxed zones indicative of deep disturbance. No major through-going structure was recognizable in the three headings; the Upper Tunnel, shaft and Lower Tunnel. A very weak and weathered shear zone which required considerable support was encountered in the Upper Tunnel and was further encountered in the shaft where it was associated with several vertical faults. This major shear zone separated the weaker, more fractured and faulted rock to the east from the more competent and less tectonically disturbed rock encountered in the western drive.

The weakness of the foliation planes, even where not sheared, and the overbreak above the spring line, resulted in the need for pattern bolting in the crown throughout the Lower Tunnel. In the Upper Tunnel foliation dips were flatter, and there was more amphibolite in the sequence thus the pattern bolting requirement was less.

3.2 *Tunnel Excavation and Lining*

The tunnels were all driven by drill and blast. The Upper East and Lower Tunnel drives were excavated using tracked methods. Drilling was carried out using “Long-Toms”, with mucking by track mounted overshot muckers and Hagglund cars. The tunnels were excavated at a 2% downgrade to the east, with a 100 m section at the east end of both tunnels excavated at 0.5% grade to assist the braking of the locomotives at the tunnel exits. Rounds were mostly taken in 1.8 m lengths, as determined primarily by the mucking operation. In general, two Hagglund cars were coupled in order to allow a full 1.8 m round to be mucked in one trip.

Conditions in the Lower Tunnel were characterized by moderate to high seepage inflows and changeable ground conditions. The Lower Tunnel was very wet and changed quite abruptly from good to poor conditions. Total seepage inflows into the Lower Tunnel reached a maximum of almost 6000 l/min during tunnel excavation, and gradually reduced to approximately 3000 l/min. Typically, initial “flush-flow” seepage inflows would reduce with time, although there were some areas where sustained high inflows occurred.

The Upper East tunnel was characterized by weathered and sheared rock, resulting in relatively slow production, and low to moderate seepage inflows. Little seepage was encountered in the first 300 m where drained conditions with local perched water tables existed. Beyond this, up to 3000 l/min was flowing from the tunnel during construction, much of this from the tunnel floor or lower walls. Rounds were taken in 1.2 to 1.8 m lengths, depending on the face and drilling conditions.

The Upper West Tunnel encountered better geological conditions in comparison with the eastern drives. The permeability was very low due to the wider joint spacing and absence of shearing. Foliation was generally tight and dipped into the face throughout the drive. Rounds were taken almost exclusively in 2.4 m lengths.

The Upper West Portal is located at a relatively high altitude (El. 1035 m) and it was expected that heavy snowfall, resulting in difficult access and portal working conditions, would curtail the extent of this drive. Also, given the 2% downward slope of this drive, accumulated drill/wash water and seepage water at the face was expected to inhibit progress. Excavation to approximately 550 m was planned, however due to slow progress from the Upper East, access to the west was maintained throughout the winter and the tunneling from this portal continued through to breakthrough at 1180 m (990 m from the east portal). The entire drive was accomplished with multi-stage scoop mucking with re-muck bays used to store the previous round located approximately every 350 to 400 m.

The rock encountered in the tunnels was mapped and classified on the Rock Mass Rating (RMR) system. Rock support was designed according to the following five classes:

- Class I, RMR rating 86 to 100 (Very Good) – no support required;
- Class II, RMR rating 71 to 85 (Good) – spot bolting only;
- Class III, RMR rating 61 to 70 – 1.8 m pattern bolts at 1.5 m centres on crown, spot bolting as required on walls;
- Class IV, RMR rating 41 to 60 – 1.8 m pattern bolts at 1.2 m centres on crown, spot bolting as required on walls, followed by 50 mm thick fibre reinforced shotcrete on crown and shoulders;
- Class V, RMR rating <40 – 1.8 m pattern bolts at 1.2 m centres on crown and upper half of walls, followed by up to 150 mm fibre reinforced shotcrete.

The distribution of the support classes, as mapped during tunnel excavation, is presented in

Figure 5. The contrast between the tunneling conditions on the east and west side is apparent; approximately 80% of the Upper West drive was in Class I and II, while less than 30% of the eastern drives were within these rock classes.

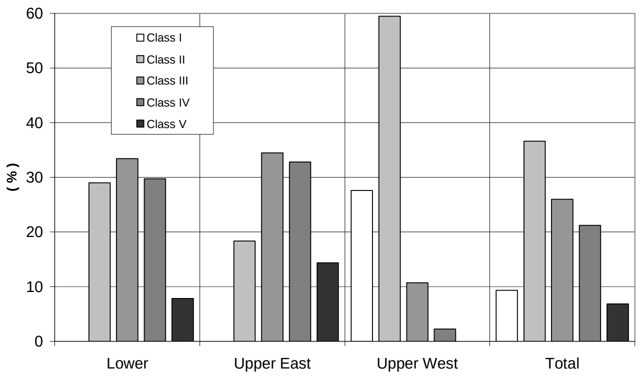

Figure 5. Tunnel support encountered in each heading

3.2.1 *Rock Support During Tunnel Excavation*

Rock support was provided principally with rock bolts and, where required, steel fibre reinforced shotcrete. In areas of high seepage water inflow "flash-set" plain shotcrete (approximate set time of 30 seconds) with welded wire mesh reinforcement was used.

In areas of high water inflow, most of the holes drilled for bolt installation produced significant amounts of water, making it difficult to push resin cartridges into the holes. In these situations, Split Sets were installed as a means of temporary support, allowing work to proceed. The Split Sets also provided drainage to the rock so that rock bolts could be installed later to meet the permanent support requirements.

In the Upper East tunnel, some problems were encountered with rock raveling in the back of the tunnel, causing up to 4 m of overbreak. Much of this was caused by a combination of close sub-vertical jointing and thin clay coatings or weak micaceous partings. Spiling was installed in 4 to 5 m lengths across the width of the back in these areas to prevent further raveling.

Through a section of particularly sheared and wet ground in the Lower Tunnel (ch 440 to 580 m), five groups of 4 to 6 steel sets each, with wooden lagging and blocking were installed for temporary support.

3.2.2 *Final Rock Support*

Upon completion of tunnel excavation, a program of permanent support installation was carried out. For the unlined sections of the upper and Lower Tunnels (ch 0 to 1850 in the Upper Tunnel and 1250 to 1450 in the Lower Tunnel), permanent support was required to prevent long term erosion or deterioration of the rock. In the Lower Tunnel where a free standing penstock was to be installed (ch 0 to 1225), rock support was required to provide protection to the pipe. The project was based on a minimum 50 year design life.

Final rock support in the Upper Tunnel consisted primarily of local application of shotcrete on the arch and walls and locally applied concrete on the invert. Two concrete liners were installed in the Upper Tunnel in areas of weak weathered shear zones. The 30 m long liners were poured utilizing 2.4 m diameter liner plate culverts.

For the Lower Tunnel, in the area of steel sets, final support was provided by applying a minimum 150mm reinforced shotcrete lining within the steel sets. This arch was designed to carry all rock loads so that the sets were no longer relied upon for support. An extensive program of contact grouting was carried out to fill void spaces behind the shotcrete arch where lagging and blocking had been installed.

Local areas of invert lining were installed to support locations of poor rock which were judged to be susceptible to long term erosion. Lining was also installed locally to provide a suitable surface for transporting equipment for future tunnel maintenance. An option to place concrete over consolidated tunnel muck was considered but rejected following a costing exercise.

Upon completion of tunnel lining, an extensive tunnel cleaning program was carried out to minimize the risk of damage to the turbines from particles and to minimize requirements for future cleaning of the rock trap.

3.3 *Raise Bore Excavation and Lining*

Drilling of a 12 inch pilot hole was the first stage of the raise boring. Careful logging of the cuttings was carried out to gather as much information as possible about the ground conditions. Circulation losses and observation of the cuttings indicated that poor ground conditions were present near 260 to 290 m depth.

Since the potential for excessive leakage from the conveyance system during operation had been identified as a significant concern, a constant head test and subsequent falling head test was conducted on the blind pilot hole (prior to breakthrough into the Lower Tunnel) to determine the permeability of the rock surrounding the shaft. The data was used to assist development of a hydrogeological model of the system.

A down-the-hole survey of the pilot hole was undertaken in order that the Lower Tunnel could be steered towards an intersection point. To drain the pilot hole in a controlled manner (static pressure in the pilot hole was approximately 40 bar) a remotely operated "bar and arm drill" was used to drill through the final 3 to 5 m of rock from the face of the Lower Tunnel into the pilot hole. After this drill-through, seepage water from the shaft area through the pilot hole was measured at a steady 1700 l/min.

A 2.1 m diameter reamer head was attached to the end of the pilot hole drill string and was pulled up at an average rate of 12 m/day to complete the shaft excavation. Observations during reaming confirmed the instability of the ground over the 260-285 m zone.

A minimum shaft diameter of 1.8 m was required to keep the flow velocities below 4 m/s during operation, based on water flows for the expanded 45 MW project. Excavating the shaft to the 2.1 m reamed diameter allowed for up to 15 cm of liner thickness through the disturbed ground.

The cuttings were mucked from the Lower Tunnel by overshot mucker and Hagglund cars, ensuring that there was always an opening at the bottom of the shaft to allow seepage water to flow and preventing choking of the base of the raise bore.

Upon completion of the reaming, it was clear that some support would be required in the shaft, but the extent and type of support was uncertain. A remote survey of the shaft was undertaken to gather data for planning the support work. To conduct the investigation, a transport vehicle was built to carry a wide-angle borehole camera down the shaft. The buggy was built by welding together two used bicycle frames at an angle, allowing the wheels to ride along the shaft walls at 90°. The buggy was lowered down the shaft with an electric winch and steel cable, and the camera returned a signal to the top of the shaft through 500 m of cable. The video feed was monitored in real-time and recorded. The survey provided sufficient information to delineate two main areas of concern, from 170 m to 190 m and from 260 m to 300 m. The camera survey also showed that almost all the 1700 l/min seepage inflow was coming from the 270 m to 290 m section of the shaft.

Design of the main section of shaft liner (260 to 290 m) primarily focused on constructability, with a number of options considered for constructing the liner in this area of high water ingress. In terms of design, the shaft liner was primarily required for stability purposes. A key early decision was that the liner would not be required to prevent or reduce leakage from the shaft. Preliminary hydrogeological modeling had indicated that the predicted rate of leakage from the shaft (and the upper and Lower Tunnels) was not anticipated to lead to unacceptable impacts on the groundwater regime, in particular to stability of the eastern slope of Pingston Ridge. Also, the anticipated loss of water was considered acceptable economically with respect to the project power generation. The elimination of the requirement for designing to limit leakage greatly simplified the design and construction of the liner, which was of great importance given the difficult construction conditions and the impact on schedule.

Based on these requirements, the selected liner design consisted of a lightly reinforced, 15 cm (minimum) thick full circumferential liner (Figure 6). The liner was designed to be leaky to minimize required concrete thickness and reinforcement. Furthermore, high pressure consolidation grouting was not carried out, with only a modest program of low pressure grouting being required.

A number of alternative methods for gaining access to the shaft to carry out the lining work were considered, with an Alimak raise climber eventually being selected.

The contractor for the shaft lining (J.S. Redpath of Sudbury) selected a slip-form lining system that progressed from the top to the bottom, thus eliminating any requirements for temporary support (as required for a bottom-up approach) in this area. The liner was poured in 15, 2 m long slip form sections. Vertically, the forms were split 0.5 m from the bottom with the lower section called the "curb" and the upper, 1.5 m section, called the "main". The 0.5 m curb section was poured, followed immediately by the installation of the main form panels, which would rest inside the poured concrete surface of the previous section. The main pour would then proceed through a small window in the formwork located at the highest point on the hanging wall. The concrete was intermittently vibrated using a pneumatic form vibrator. Concrete was delivered from a batch plant located at the top of the shaft through a 75 mm diameter steel slickline bolted to the footwall. Six threaded ports for grouting and for drain hole drilling were fitted into each of the pour sections.

Control of the 1700 l/min leakage between 270 m and 290 m was a key concern of the contractor and design team. Large, individual seepage sources were collected in steel boxes and pipes fixed to the walls of the shaft and routed through three 2 inch diameter steel pipes that were carried to the bottom of the liner, cast into the north wall. Diffuse water sources were controlled using geotextile drainage fabric (Nilex NuDrain), which was pinned to the rock surface and fed into a drainage collection system. This drainage system proved to be extremely effective in controlling inflows and minimizing damage to the concrete.

Grouting was carried out on completion of the shaft lining and was commenced by grouting the drainage system from the discharge pipes at the bottom of the liner. Following grouting of the drainage system, contact grouting was carried out through the grout hole rings (six holes each). Once a ring of grout holes were drilled (each 3 m long from the inside of the liner), a network of injection lines were attached to the six ports and grout was injected one hole at a time, proceeding clock-wise around each ring. Maximum injection pressures which were slightly above hydrostatic groundwater pressures were maintained. Once one hole was completed, the injection valve was closed, maintaining the injection pressure in the hole, and the next injection valve was opened to start grouting of the next hole. A second grouting pass of the holes was generally attempted to see if any more grout could be injected.

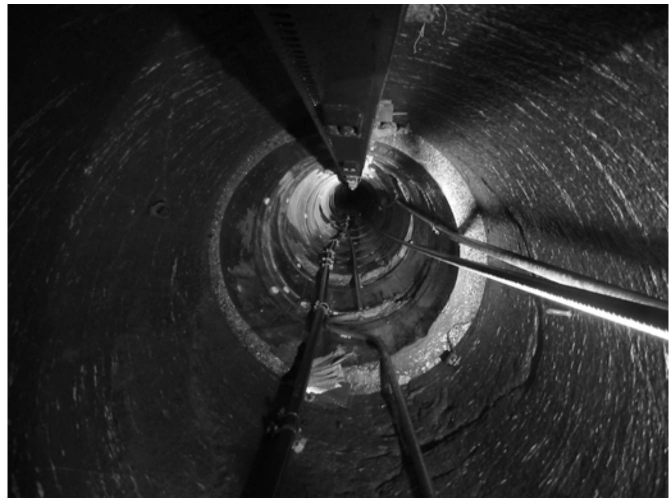

Figure 6. View of upper terminus of main shaft liner

Grouting proceeded from the bottom to the top of the liner and proved to be effective as observed inflows were progressively channeled upwards in the liner. Once all the grouting was complete, the seepage stopped almost completely for 1-2 days while pressure increased behind the liner. With increasing pressure, seepage began to reappear, generally at the locations of construction joints. As indicated previously, it was decided that the liner was to be designed to leak in order minimize hydraulic particularly during watering and dewatering. Drainage holes were drilled upon completion of grouting. Many of the holes encountered significant amounts of water, which decreased with time, and within a week, the pressure stabilized at a constant seepage flow, similar to that before grouting.

In addition to the main concrete liner, 12 additional, more localized zones required concrete lining and numerous areas required bolting. The contractor elected to pour concrete in these areas rather than use shotcrete. These areas were all supported with 15 cm of reinforced concrete in partial or full circumference sections. Grouting was not required in these areas.

3.4 *Plug Design and Construction*

A detailed description of the design and construction of the two tunnel plugs is beyond the scope of this paper, however a summary of critical design and construction issues is provided in this section.

3.4.1 *Lower Tunnel Plug*

A plug was required in the Lower Tunnel to serve as the upstream terminus of the steel penstock (Figure 7). Selection of the plug location and design of the plug was to be determined during tunneling, based on observed ground conditions and in-situ measurements taken during construction. Based on cover considerations, it was anticipated that adequate confinement would be available near ch 800. Upon reaching this location and identifying a suitable plug location, a hydrojacking testing program was carried out. The testing results indicated that the hydrojacking pressures were lower than what was considered acceptable by the design team. It was decided that an alternative location would be sought further upstream in the tunnel where greater depth of rock cover would be available. Due to poor rock conditions, the next suitable plug location was not identified until ch 1225.

A second hydrojacking testing program was commenced upstream of the proposed plug location. Results from this testing indicated that hydrojacking pressures were still lower than required by the design and, in fact, were similar to the values measured at ch 865, where rock cover was approximately 100m less. Further testing at ch 1420 m, under still higher cover indicated that hydrojacking values were not increasing.

The stress measurements presented a significant challenge to the design team. It was clear that even if the plug was located at the extreme upstream end of the Lower Tunnel (at the location of greatest rock cover), the measured hydrojacking values were still insufficient to meet the design criteria. While a detailed discussion of the stress measurements, analyses and design deliberations is beyond the scope of this paper, it was eventually concluded that a lower factor of safety could be tolerated. Information considered in reaching this conclusion included the very good rock conditions, (indicated by very wide joint spacing, an absence of shearing or faulting, and very low hydraulic conductivity), hydrojacking values from the Upper Tunnel and precedence from other projects.

Figure 7. Lower Tunnel Plug – upstream end.

In order to achieve a high level of confidence in the construction and subsequent performance of the tunnel plug, a number of key features were incorporated into the plug construction,

- Self compacting concrete (SCC) was used for the entire plug. This eliminated the requirement to gain access into the extremely confined areas for concrete vibration and produced a highly workable, high strength and low shrinkage mix which was highly suitable for this application. This also allowed larger pours and fewer construction joints;
- The plug was constructed and poured in four equal 6 metre sections. Each section was poured in a single continuous pour, thereby completely avoiding longitudinal construction joints;
- Ultrafine grout was used for high pressure consolidation grouting. Highly stable, low viscosity grout mixes with extremely good

penetration properties were achieved using mixes prepared with ultrafine grout.

The plug was completed over a period of approximately 2 months with completion near the end of October, 2002.

3.4.2 *Upper Tunnel Plug*

The Upper Tunnel plug was to be designed to allow passage of maintenance equipment into the tunnel for cleaning the rock trap and carrying out maintenance in the Upper Tunnel and shaft (Figure 8). A door, of minimum size 1.8 m (wide) by 2.0 m (high) was required for access of the equipment. After consideration of a number of alternatives, a hinged rectangular door, fitted inside a steel frame and liner was selected for the design. The static head acting at the plug was 5 bar. The plug length was 6 m, with the upstream 2 m of the plug being steel lined, and the downstream 4 m consisting of reinforced concrete.

A significant challenge in constructing the plug was to minimize disruption of access into the Upper Tunnel, where a continuous supply of materials was required for permanent support activities to continue in the shaft and Upper Tunnel. This required a modular construction sequence, with much of the plug construction being carried out in late December 2002, when work in the shaft was halted for 10 days.

Figure 8. Upper Tunnel Plug – upstream end showing door hinge on left.

4 CONVEYANCE FILLING

As discussed previously, the impact of the underground excavations on the existing groundwater regime during operation was considered to be a key concern. It was necessary to understand the potential impact of the project on existing groundwater conditions – particularly as slope stability might be affected – and to understand the potential losses of water available for power generation. To achieve this, a hydrogeological model was developed and calibrated using available data from:

- in-situ testing carried out during the pre-construction geotechnical investigations and during construction;
- using measurements of groundwater inflows into the tunnel which were collected at regular intervals during tunnel excavation.

Since much of the pre-construction investigations were carried out at shallow depths, emphasis was placed on collecting information during construction which could be used to calibrate the model. The tests conducted on the pilot hole for the raise bore (discussed previously) were particularly important in helping to calibrate the model. Filling of the conveyance system was also seen to be a useful opportunity to collect information on the performance of the system and for comparison with the hydrogeological model predictions.

A carefully developed tunnel filling procedure was considered essential in order to minimize damage to the tunnel liners, plugs and rock and also to carry out tests to assess the performance of the hydrogeological model. A filling schedule was developed with a duration of 8 days. The schedule included rapid filling of the lower penstock and slower, more controlled filling of the Lower Tunnel, pressure shaft and Upper Tunnel. Control of the filling rate was achieved using controls on inflows via a gate valve at the intake, and with fine adjustments achieved by spilling excess water at the powerhouse using needle valves on the turbines. Regular measurements of total system inflows (intake inflow minus discharge at the powerhouse) and system pressure (measured at the powerhouse) allowed the measurement of total system seepage losses against system pressure. These values were compared at regular intervals to predictions developed using the hydrogeological model. For ease of comparison, all values were plotted, allowing immediate comparison against predictions (Figure 9). At a number of key steps, system inflows and outflows (at the intake and powerhouse) were closed and the response of the conveyance system measured. These tests provided an accurate measurement of leakage losses from the tunnel and shaft (measured by the falling level of water in the shaft/tunnel).

As shown in Figure 9, the conveyance filling showed a close agreement between measurements and predictions for the filling of the Lower Tunnel and shaft. Filling of the Upper Tunnel, showed a divergence between prediction and measurements during the initial hours of filling. Upon completion of filling of the Upper Tunnel, a system test showed the model under-predicted seepage losses but subsequent tests showed a reducing disparity. This disparity is considered to be due to re-saturation of the surrounding rock, drained during the two years of underground excavations. Long term (steady state) predictions were found to have an acceptable level of accuracy.

Shortly after the successful completion of conveyance filling, a malfunction occurred in one of the main valves at the powerhouse. The repair of the valve required immediate dewatering of the tunnels and shaft. Tunnel dewatering was successfully completed, which allowed an early inspection of the tunnels and base of the shaft. The inspection revealed that the liners, the tunnel plugs and the rock withstood the demands of watering and dewatering extremely well, with no visible signs of damage.

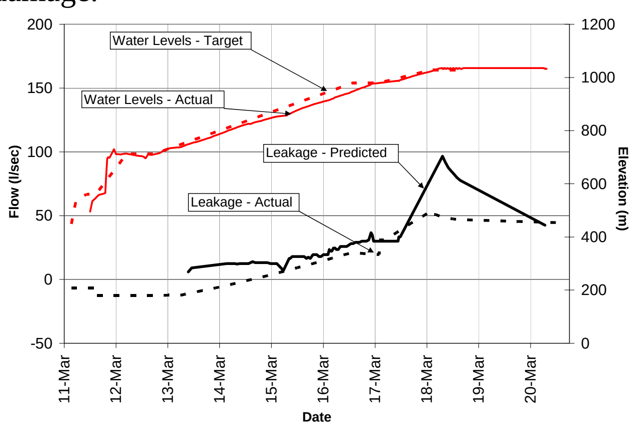

Figure 9. Tunnel filling curve - predicted and actual response

5 CONCLUSIONS

The significant aspects of the underground works carried out as part of the Pingston Hydro project are as follows:

- The project is a very high head development located close to a geologically complex area;
- Quartz gneisses were uniformly present throughout the underground works with consistent east-dipping foliation;
- Foliation shearing and major jointing in much of the Lower Tunnel, shaft and part of Upper Tunnel resulted in high water inflows;
- Hydrojacking tests in the lower power tunnel showed that in situ stresses were unusual with minimum stress being much less than overburden;
- Design decision made to accept low FOS against hydrojacking at the plug upstream of the 1225 m steel liner because of the good rock and tight conditions; very careful design and construction of plug was needed;
- Difficult concrete lining required in a 30 m section of sheared and faulted ground in the inclined shaft;
- Groundwater modeling based on in situ tests and assessment of tunnel construction inflows were carried out to assess level of potential leakage against acceptable losses and the potential for slope instability;
- Monitoring during conveyance filling to check against the model results showed good correlation;
- Inspection after early dewatering showed the absence of any rock instability or deterioration and no plug problems.

The project was developed jointly by Canadian Hydro Developers of Calgary and Brascan Power of Toronto. Design and project management was carried out by Canadian Projects Limited (CPL) of Calgary; Golder Associates provided tunnel design and construction inspection services and materials QA/QC services; EBA Engineering provided design and inspection services for the Diversion weir; and Graham Rawlings Consulting Ltd acted as geological advisor. AMEC Earth & Environmental provided rock support design services under a contract with Thyssen Mining and Construction Ltd (TMCC). TMCC carried out tunnel construction and raise boring, while JS Redpath installed all shaft linings.

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6 *REFERENCES*

Benson, R.P. 1987. Design of Unlined and Lined Pressure Tunnels. *Canadian Tunnelling*: 37–65

Moore, D.P., Ripley, B.D., and Groves, K.L., 1992. Evaluation of Mountainslope Movements at Wahleach, in Geotechnique and Natural Hazards, Bitech Publishers, Vancouver, Canada, pp 99-107

Moore, D.P., 1999. Rock Slopes and Reservoirs. In “Slope Stability and Landslides”, Proc, 13th Vancouver Geotechnical Society Symposium, Vancouver, pp 1-18.