Hydrojacking Case in Plutonic Rock Mass

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ABSTRACT

A hydrojacking phenomenon event took place in an underground hydroelectric project located in the Canadian Shield early in the 2000's. During the site investigation as well as during construction of the underground complex, the in situ state of stress raised the concern about the possibility of a hydrojacking event during the commissioning of the power tunnel at full water pressure. Two solutions were considered; one was constructing a longer steel lining throughout the weak in situ stress zone. The other solution was grouting of the same area, which was implemented several times before and after the first commissioning. This paper resumes chronologically the analysis and work carried out as well as the events that preceded the hydrojacking phenomenon.

RÉSUMÉ

Un soulèvement hydraulique a eu lieu dans un projet hydroélectrique souterrain situé dans le Bouclier canadien au début des années 2000. Lors de l'étude du site ainsi que lors de la construction du complexe souterrain, l'état de contrainte in situ a soulevé des inquiétudes quant à la possibilité d'un hydrosoulèvement lors de la mise en eau du tunnel à la pleine pression. Deux solutions ont alors été envisagées pour palier à cette possible situation. Une première solution était la construction d'un revêtement en acier sur toute la longueur de la faible zone de contrainte. Une autre solution était l'injection au coulis de ciment de la même zone et cette dernière solution a été mise en œuvre plusieurs fois après la première mise en eau. Cet article résume chronologiquement l'analyse et les travaux réalisés ainsi que les événements qui ont précédé le phénomène d'hydrosoulèvement.

1 ROCK MASS CHARATERISTICS

The underground complex is located in a topographic nose (Figure 1). The rock mass is a batholith of anorthositic composition with random intrusions of amphibolite, pegmatite and aplite.
There are two important intrusions, a gabbro body located in the power tunnel, penstocks and part of the powerhouse and a smaller amphibolite body in the access gallery (Figure 2). These two bodies play a very important role, specifically the gabbro, in defining the geological structures around the powerhouse, influencing their orientation and the level of the in-situ stress.

The gabbro body itself is less fractured and the level of in-situ stress is higher than in the surrounding rock mass, which is of excellent quality, high recovery and RQD percentages. The joints are generally closed, showing null or low permeability.

There are mainly three principal joint sets, vertical, subvertical and stress relief joints plus two secondary sets. The absence of underground water flow speaks about joints and rock mass quality.

Upstream of the powerhouse the rock mass main characteristic is a shear, which is dipping under the manifold and the powerhouse, governing the structures of the sector and the in-situ stress, altogether with the intrusives. The UCS test on anorthosite yielded an average of 205 MPa and a unite weight of 31 KN/m^3.

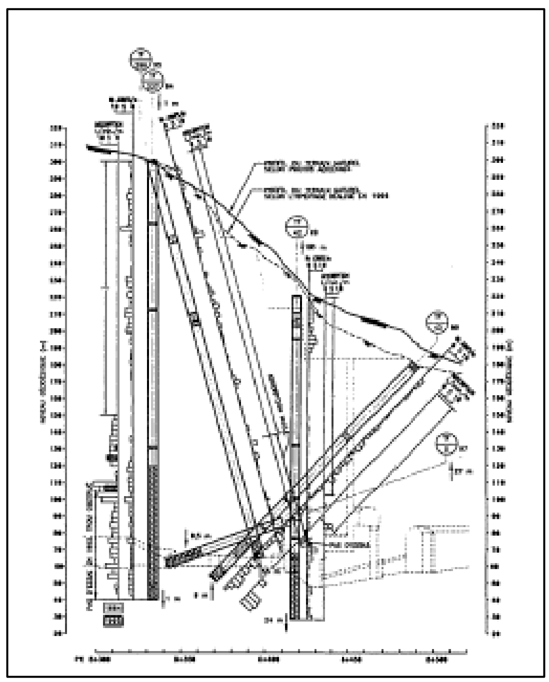

Figure 1: Site investigation at the powerhouse area.

2 IN SITU STRESS MEASUREMENTS

2.1 Hydrofracture Tests

During the 1994 site investigation, hydrofracturing tests were conducted in a vertical borehole (TF-227) from the ground down to the manifold. Figure 3 presents the test results. The arrow indicates approximately the depth location of the manifold-powerhouse (242-220m).

Between 190 and 220m of depth the in-situ stress is significant lower. At the time, it was stated that the rock mass was strong enough to withhold the hydrostatic pressure of 3,2 MPa.

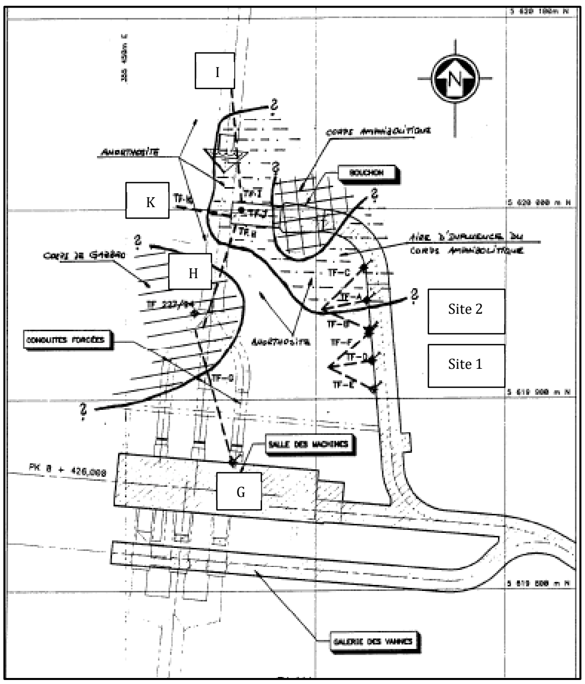

Figure 2: Underground excavation status in 1997. Doorstoppers and hydrojacking test locations. Interpreted intrusive bodies.

2.2 Doorstopper Tests

In May of 1997, only the access gallery, and the top headings of the gates gallery and of the powerhouse were excavated. An underground verification of the stress level for the penstocks steel lining lengths, was planned with doorstopper cells tests which were carried out on two adjacent sites (1 and 2), located in the access gallery at a depth of 242m. Three boreholes in each site (Figure 2) oriented towards the penstocks location.

The rock cores showed that this portion of the rock mass, close to the intrusives bodies, was intruded with different sizes of gabbro veins mixed with weak veins of amphibolite.

In general, but particularly in this sector, the in-situ stress field was influenced by the variability of the rock deformation produced by the described local heterogeneities. Therefore, there was no reliability of the stress measurements results. Table 1 presents a summary of the data for each testing site, including the principal stress with their orientation.

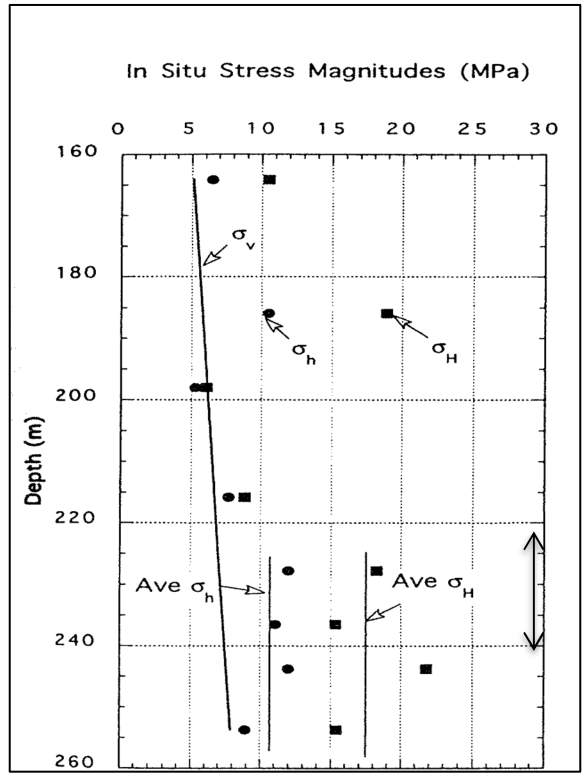

Figure 3: Hydrofracturing stress results.

Table 1: Doorstopper Tests

Site	Principal Stress	MPa	Azimuth	Dip (°)
1	σ_1	12,4	70	-33
	σ_2	4,9	349	14
	σ_3	0,6	99	53
2	σ_1	8,5	80	-17
	σ_2	4,8	151	48
	σ_3	2,5	3	37

Independently of the reliability of the tests results, it can be seen that the value of σ_3 was low. This was the first signal of the possible oncoming problem and it was not ignored. At the time of the doorstopper testing, it was noticed that, at the intersection of the access gallery with the future power tunnel, a shear structure was crossing through the gallery's end face.

The shear is made of an old cemented structure over which a new shear with gauge was superimposed. This shear influenced the rock quality at the end of the access gallery walls.

2.3 Hydrojacking Test

The results obtained with doorstopper measurements combined to a numerical simulation led to the conclusion that the low vertical stress was no high enough to avert a hydrojacking event. Considering this fact, latter in 1997, hydrojacking tests were carried out at the end of the access gallery (Figure 2). Four boreholes were drilled covering the area of influence of the shear. The obtained results are presented in Table 2, which shows a variety of stresses in function of the boreholes orientation and of the intercepted geological structures.

The stresses are as low as 1,2 MPa. Borehole G shows the highest stresses because it was drilled almost entirely into the gabbro body, as well as most of Borehole H. The lower results were obtained into the anorthosite portion of the rock mass with its mentioned heterogeneities.

Table 2: Hydrojacking Tests.

Borehole	Depth (m)	Stress (MPa)
G [3]	34-39	3,1 [1]
	43-48	4,1
	56-61	4,0
	63-68	6,5
H [4]	18-23	1,2 [1]
	24-29	2,1 [1]
	37-42	4,7
	47-52	3,6
I [5]	22-27	1,8 [1]
	35-40	1,7 [1]
J [6]	12-17	[2]
	22-27	1,5 [1]
	29-34	1,2 [1]

[1] Value lower that water head of 3,2 MPa.
[2] No test, very high permeability.
[3] From powerhouse towards the penstocks.
[4] From access gallery towards penstocks.
[5] From access gallery towards upstream.
[6] Down vertically at the access gallery.

3 GROUTING TESTS

As the tests and the conclusion were on its way, the construction of the 8,3 km long power tunnel started. At that moment the project was oriented to the steel lining solution for the low in situ stress problem, but grouting was brought into discussion.

Early in 1998, a grouting test was performed in a portion of the power tunnel between the access gallery and the manifold, in half of the invert and crown, together with the tunnel's east wall. Three boreholes were drilled before the grouting to assess the hydraulic conductivity of the rock mass by water pressure tests. These three boreholes together with two newly drilled boreholes were tested again assessing the hydraulic conductivity after the grouting work.

The grouting holes were 20 m long. In most of the holes, cement Type 30 with a Blaine fineness of 5000 cm^2/g was used. Cement with a Blaine fineness of 15000 cm^2/g, combined with an additive, was also used in a few holes as an additional test. The classical North American grout mixes ratios were used. The maximum grouting pressure was set to 4,2 MPa. A total of 44 boreholes were grouted with a consumption of 60,5 Tons of cement.

Table 3: Hydrojacking events during grouting.

Borehole	Section [1]	Grout Mix w/c [2]	Pressure Drop (MPa) [3]
TF1	2	0,75	2,2-1,7
	2	0,75	3,2-2,2
TF3	1	3	1,5-1,1
	3	5	0,9-0,6
	4	3	1,4-0,9
TF18	1	5	2,6-2,1
	3	2	3,0-1,5
TF32	3	5	4,2-3,0
TF35	1	5	3,3-2,9
TF28	1	5	3,5-2,0
	3	5	2,5-1,8
	4	5	2,0-1,0
TF47	2	5	4,2-2,5
TF49	2	5	4,2-3,2
	3	5	4,2-3,1
TF50	4	5	4,2-2,5
	4	3	2,5-2,0
	4	3	2,0-1,5

[1] Grouting sections of 5 m. Section 1 at the bottom and 4 at the collar of the hole.
[2] w/c by volume.
[3] Pressure before and after claquage.

Table 3 presents the hydrojacking events, or "claquage", during the grouting test. Also presented on the table, the grouted sections for each borehole, the grouted w/c ratio at the moment of pressure drops, and the pressure before and after the drops. 18 cases of pressure drops were noticed in 9 boreholes. During the work it happened to be some communications between grouting holes, with the water tests holes and with joints located in different zones of the rock mass in the access gallery and the power tunnel.

The grouting produced water inflow into the underground excavations and into open holes ready to be grouted. The water was from the grout mix itself due to the high pressure (pressure bleeding). Some of the pressure drops took place in a same hole.

Table 4 presents the claquage cases that took place in the water pressure tests holes before and after the grouting. The indicated pressure is the one at which the claquage occurs.

Among the conclusions that the grouting test yielded, the most important were that the low stress area was influenced by the shear, that there was grout absorption and claquage only in the area affected by the low in situ stress. Finally, this confirmed that, as already known by many, it is impossible to build-up an adequate state of stress with the grouting methodology.

Table 4: Cases of Hydrojacking before and after Grouting in Water Pressure Tests Holes

Hole	Section	Depth (m)	Pressure (MPa)
Before Grouting			
TFA	5	4,5-1,5	2,0
TFC	1	20,6-16,5	3,5
TFC	4	9,5-6,0	4,2
	5	6,0-2,5	3,5
After Grouting			
TF7	3	13,0-9,5	4,2
	4	9,5-6,0	3,5
TFA2	5	5,0-1,5	0,5
TFC2	4	9,5-6,0	4,2
TFE	1	20,0-16,5	2,5
	2	16,5-9,5	3,5

4 LEAKAGE CALCULATIONS

Despite of the conclusive results of the grouting test, at the end of 1998, the project was then oriented to proceed with grouting as the solution. To assess the leakage into the openings, the hydraulic conductivities of each portion of the rock mass in its natural condition, after grouting, and of the hydrojacked grouted rock mass were calculated.

The water's paths towards the openings were identified. Thus, leakages from the power tunnel at full hydrostatic water head were calculated in function of the hydraulic conductivity and the geological assumptions. The leakage calculation did not take into account the intrinsic nature of the hydrojacking phenomenon, which is the pulsating discharge.

Table 5 presents the most relevant values obtained from the analysis including different hypothesis and many variants of the concrete plug locations and lengths. The calculations were useful to assess the magnitude of the problem, and for dimensioning the pumps to face the increasing amounts of water. Figure 5 presents the principal flow orientations and the water discharge areas.

Because of the huge number of possible hypotheses combinations, calculations for leakage values were done only for specific areas of discharge. As an example, taking the minimum and maximum values after claquage for the west wall of the powerhouse and the access gallery together, the total leakage reached up 4 and 30 m^3/min respectively. To put the calculated leakage volume into perspective, the drainage gallery was, prior to the problem, designed for 6 m^3/min.

Table 5: Leakage values and Discharge Sectors.

Discharge Sector	Leakage (l/min)			
	After Grouting		After Claquage	
	Average	Max	Average[1]	Max[2]
Drainage Gallery	12	21	12	21
West wall PH. 1	150	4613	827	1504
West wall PH. 2	1408	8856	7749	14084
Access Gallery 1	648	8709	3620	6585
Access Gallery 2	538	7660	2951	5369
Access Gallery 3	345	5971	1892	3445
Access Gallery 4	259	4477	1419	2583
Access Gallery 5	1552	18681	8533	15518

[1]The average values after claquage were calculated increasing 5 folds the hydraulic conductivity.
[2] The maximum values after claquage were calculated with increase 10 folds the hydraulic conductivity.
[3] Two cases of Powerhouse West Wall for the concrete plug location. 1 is for the plug at a distance from the junction and 3 the plug was at the junction.
[4] The 5 cases of Access Gallery are due to different locations and length of the concrete plug.

5 GROUTING AND COMMISSIONING

5.1 Grouting Phase I

Although the conclusion and recommendations against the grouting program it was implemented as the solution in the low stress area of the power tunnel. The grouting was carried out from the end of 1999 to the first months of 2000. Together with the grouting program it was agreed that the concrete plug was to be displaced from its original position to a new one. The face of the plug was then aligned with the east wall of the power tunnel, to avert putting under pressure the very weak end of the access gallery.

The grouting pressure was 2,5 MPa. The work was done dividing the sector on 4 zones, rings at 4 m apart consisting of 6, 8 and 12 holes long of 21 and 26m. A total of 582 holes and 10898 m were drilled. A total of 242 Tons of cement was grouted. The grouting analysis is out of the scope of this paper.

5.2 Commissioning I

It was planned to increase the water pressure in the power tunnel at the reservoir's filling rate. On April 12^{th} of 2001, the power tunnel was under water pressure.

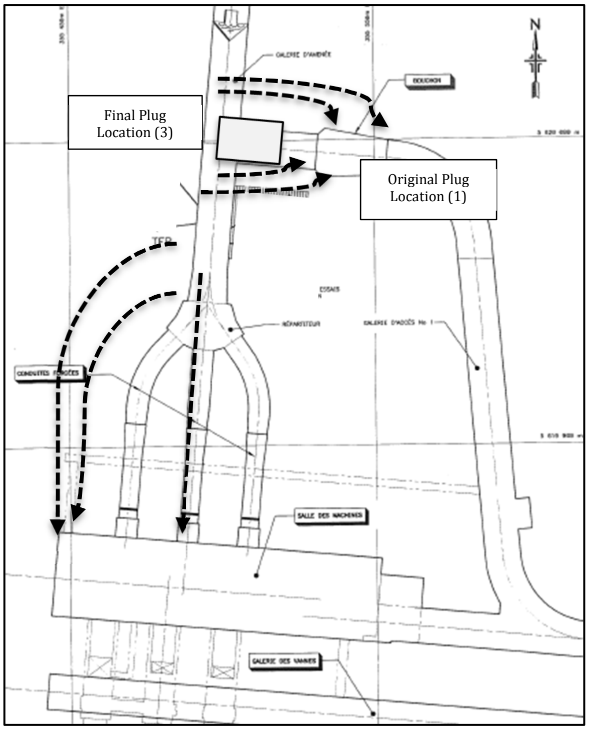

Figure 4: Principal flow paths and discharge areas.

With the increasing water pressure, the leaks at the access gallery behind the concrete plug, at the west and north walls of the powerhouse and at the drainage gallery, were not significant within the first few days. The water head was at that time lower than the maximum 3.2 MPa. Some days later, a leak of 29 litres/sec was estimated at the concrete plug.

The water inflow at the drainage gallery was increasing from the previous days. At the access gallery behind the concrete plug the water was reported to come out of the rock mass from the crown, the walls and the invert. Days later the leakage was estimated at 88 litres/sec at the access gallery.

At this moment a power failure put the installed pumps out of service, producing a flood in the access gallery. On April 23rd, the leakage at the access gallery was estimated at 120 litres/sec, and a leakage of 80 litres/sec at the drainage gallery (2/3 of the its max. capacity of 120 litres/sec). The west wall of the powerhouse is reported to be "wet". The inflow volume (120+80-litres/sec) equals 720 m^3/hour, plus an unknown water volume travelling elsewhere into the rock mass.

The relationship between water pressure of that portion of the unlined tunnel and low rock mass stress has clearly a time-related behaviour. The power tunnel was under water pressure for approximately 3 weeks with increasing water head, but ultimately in the last 11 days the leakage increased more than 400% at the access gallery alone.

If the power tunnel would have been under pressure for a long period of time, after saturation of the rock mass, seepage could have reached all of the underground openings resulting in a much higher leakage, as shown by the calculations presented in Table 5.

On June 1st, the reservoir has still to rise about 20 more meters (0,2 MPa) to reach its maximum level. After the first commissioning, the upstream or north wall of the powerhouse was displaced only a few millimetres under the water pressure, but enough to disrupt the turbines test process.

To allow for the turbine testing, a cut of the concrete upper floor of the powerhouse, parallel to the upstream wall was carried out; the opening was of a few millimetres. When the tunnel was later under pressure for the second time, the wall reacted with a new displacement and the cut in the concrete was progressively closing up.

5.3 Grouting Phase II

A second intensive grouting program took place, from January to August 2001. The same grout mixes were used, and the pressure was set to 2 MPa. The grouting was carried out in two zones including one that was already grouted in the previous program. The rings were of 6 and 8 holes, with 26m long holes. A total of 162 holes and 2592 m were drilled. A total of 134 Tons of cement was grouted.

A quick comparison between grouting phases shows that in the second grouting phase, 24% less meters (8300m) were drilled but the quantity of cement used was 55% of the previous phase (134 Tons). The difference could be explained by the damage to the rock mass caused by the hydrojacking.

5.4 Commissioning II

The second commissioning of the power tunnel started on September 20th, 2001 and reached the hydrostatic pressure of 3,31 MPa on the 26th. The monitoring of the affected zones was continuous, with pumping flow, extensometers and load cells, 3 new points for flow monitoring were added. An electronic flowmeter was installed at the drainage gallery, which was to measure the water pumped from the access gallery.

Another v-notch was installed to measure the water from the drainage holes at the drainage gallery (DEV2). Another one (DEV1) was monitoring the flow from the drainage holes at EL 58m at the north wall of the powerhouse. The flow measured at DEV1 increased rapidly and stabilized around 11 l/sec between September 27th and 29th. At DEV2 the flow increased significantly between September 25th and October 11th and then stabilized at 70 l/sec.

A different situation was observed at the Access Gallery were, even if the water head was constant at 3,31 MPa on September 26th, the flow was increasing at a rate of 4,5 l/day and stabilized at 218 l/sec on October 30th. Of that value, 26 l/s was coming from the small gallery inside the concrete plug. Due to this second pressurization phase of the tunnel the consequence was a deeper disturbance of the rock mass. There was an obvious decrease of the time for the water to reach new volumes into the rock mass, because most of it was already disturbed and saturated. Thus the perturbation of the rock mass had reached the powerhouse.

With these flows and signal of instability such as rock in the wire mesh, opening of joints on crown and walls of the

Access Gallery and fissures in the concrete plug, it was decided to stop the commissioning on October 31st; releasing the internal pressure on the rock mass. After an evaluation of the situation, the intake gates were opened on November 6th to allow for a new pressurization of the power tunnel, 21 hours later the gates were closed again on November 7th.

During this short period of time, 30 measurements of flow were done for analyzing the rock mass response to a quick increase of the water pressure. At this moment the flow at the access gallery was of 230 l/sec, 12 l/sec more than on October 31st. This closing period extended up to the dewatering of the power tunnel starting on November 26th and ending on December 5th, 2001. The first 6 days after the gates closing, the infiltration flow at the access gallery decreased of 30 l/sec, and some days later the infiltration flow decreased of 60 l/sec. On December 6th the water infiltration stopped. After the analysis of the situation, the infiltration values, the damage to the concrete plug and to the rock mass, it was decided to proceed on January 2002 with a third grouting program; previously an underground inspection took place.

5.5 Underground Inspection

5.5.1 Access Gallery and Plug

Water inflows were still visible and even in spots far from the concrete plug that was never wet before. Other spots were dry but the seeping water stained the walls and crown of the gallery. Small rock blocks were displaced and some had fallen. At the plug's concrete-rock contact there was a water inflow of about 3 l/min. The shear visible in the rock mass now cuts diagonally through the concrete plug. The plug behaved as part of the rock mass; the displacements induced the rock mass' jointing into the plug. Two of the concrete pouring interfaces were opened. The rock-concrete contact was sound. While doing the grouting phase III, the subhorizontal grouting holes on the wall of the plug, communicated between them and had communicated also with the east wall of the power tunnel up to the debris trap. All the fissures in the concrete were connected.

5.5.2 Power Tunnel

Water flowed from the shear at the west wall, all the way down up to the invert at the debris trap. The total inflow was about 30 l/min. The joints at the trap were open up to 5 cm. The joints, in the invert between the debris trap and the manifold, were open. The west wall showed that the shear has remained open (it was covered with shotcrete during grouting phase I).

5.5.3 Manifold and Penstocks

The manifold showed small water inflows from several subhorizontal fissures on the concrete walls. Several subhorizontal fissures were observed in the concrete lining of penstock #1. One fissure carried water over 3 m in length up to the steel liner. The concrete-steel contact was open (1mm) with water inflow of 2 l/min. The contact shows a few mm of displacement. The penstock #2 shows less damage and no water inflow. The concrete-steel contact was open about 2 mm. Penstock #3 showed several subhorizontal fissures opened less than 1 mm. The contact was open about 2 mm.

5.5.4 Powerhouse and Aspirator

The powerhouse's upstream wall, at the elevation of the penstocks, was colored as well as the west wall, which still shows some seepage. Certain drains at the toe of the upstream wall were still pouring water into the ditch. The water colored the downstream wall, between units 2 and 3. The shotcrete located on the downstream wall of the powerhouse between units 1 and 2 and between units 2 and 3, presented vertical fissures. The bus bars galleries 1 to 3 presented subhorizontal and vertical fissures on the shotcrete. The concrete walls of the aspirator were fissured and open, showing displacements and loosening close to the junction between the cone and the aspirator. There were very long fissures extending from the gate to all the aspirator. The fissures were on the walls, crown and invert.

5.6 Grouting Phase III

On January 2002, the grouting phase III was on its way and it lasted up to February 2002. The grouting pressure was set at 3,5 MPa. 18 holes long of 21m per ring. Of the total planned, just 39 holes were drilled and grouted with 126 Tons of cement. The grouting was ended due to the impossibility to reach the refusal, the huge number of communications between holes and especially with the power tunnel (east wall and crown), and 10 events of claquage during grouting.

5.7 Steel Lining - Hydrojacking Tests

Once the grouting ended, the project decided to implement the construction of an upstream concrete plug and the installation of steel penstocks into the power tunnel (ø: 16,5x11,5 m), linking the new plug and the existing steel lining. Determining the location of the new concrete plug, took ten boreholes, distributed along the tunnel (walls and invert), tested for hydrojacking at the required level of in situ stress. The upstream face of the new plug is located at approximately 270 m upstream of the manifold. There was almost not consumption of cement grout during the consolidation grouting at the new plug location.

6 FINAL COMMENTS

The hydrojacking of the rock mass took place by opening the shear plane and displacing the hanging wall upward; in which is located the powerhouse and the other underground openings. Once the displacement initiated, the rock mass on the hanging wall was affected by a sudden decrease on the level of in situ stress. This generalized reduction of the stress allows to the joints to open up and the pulsating water discharge was on its way. If a new commissioning was to take place, or a sustained single one, it was suspected that the rock mass would have suffered even more severe damages. Failures could happen in the powerhouse and elsewhere into the openings. Water inflow into the powerhouse could increase drastically.

The displacement of the powerhouse and related underground structures was, obviously, not linear and monolithic. Rock mass recovering to its initial position after the final dewatering was a very slow-motion displacement that did not fully reach the original position. Since the beginning of the operations, the tunnel and the powerhouse performed very well and there are no specific issues related to the hydrojacking phenomenon that happened during commissioning.

7 ACKNOWLEDGMENTS

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8 REFERENCES

Author's personal implication, project files and observations (1997-2003).