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Significance of grouting for controlling leakage in water tunnels – a case from Nepal

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ABSTRACT: The rock itself is a significant barrier against groundwater movement if it has low porosity and is unweathered. The existence of discontinuities in the rock mass however increases the permeability and it may vary widely. In the Himalayas, active tectonic movement and shearing have made rock masses weathered and fractured and increased the permeability. Having such rock masses has direct impact on stability caused by water inflow and leakage in tunnelling. If the tunnel is excavated for conveying water, the leakage problem is not limited only to the tunnelling phase, since there is also high risk of water loss through unlined tunnels during operation. The paper discusses the role that preinjection and postinjection grouting played for controlling leakage through the unlined headrace tunnel of the 60 MW Khimti I hydropower project in Nepal Himalaya.

1 INTRODUCTION

The rock mass is typically a jointed aquifer where water moves through the most permeable discontinuities or through open channels along them. In general, the rock masses close to the surface are more jointed and the joints are more open than deeper in the rock mass. Visual observation carried out in many ungrouted tunnels indicates that most water leakage in general occurs in the part of the tunnel which is closest to the surface and is confined in fractures, faults and weathered zones (Nilsen and Thidemann, 1993 and Karlsrud, 2002).

Water leakage problems in unlined or shotcrete lined water tunnels are not new issues in tunnelling. In many occasions severe water inflow as well as leakage problems have been faced that not only reduced the stability of the rock mass surrounding the tunnel but also valuable water has been lost from the tunnel causing huge economic loss to the projects. According to Kassana and Nilsen (2003), some of the notable projects, which have suffered excessive water leakage problems are Chivor II (Columbia), Whatshan (Canada), Askora and Bjerka (Norway) and Kihansi (Tanzania). Consequently, water leakage control in the tunnels plays a vital role not only in improving the rock mass quality, but also in saving economic loss caused by leakages. The innovative solutions that exist at present in tunnelling makes it easier to control and reduce the amount of water leakage to a target level by pre and even post-grouting technique. However, the main concerns are the cost and time, which must be accounted and optimized. Thus, the proverb "prevention is better than cure" fits very well when it comes to water leakage control in tunnels.

In the Himalaya, due to active tectonics in this region, the rock masses are highly fractured, folded, sheared and deeply weathered. Tunneling through numerous zones of weakness, fractures and faults is thus a matter of reality. Moreover, the majority of these zones are in general highly conductive, representating potential sources of ground water aquifer as well as possible sources of water leakage from the completed unlined/shotcrete lined tunnels. Accordingly, treating the rock mass with injection grouting against water leakage might be very cost effective and environmentally friendly solutions for tunnels in this region. As an example on the effectiveness of grouting, Karlsrud (2002) indicates that the hydraulic conductivity of the rock mass closest to the tunnel periphery can be reduced to approximately one tenth the conductivity of ungrouted rock if systematic pregrouting is carried out during tunnel construction.

This paper is thus focused on the significant role that grouting may play in controlling the water leakage. For this the headrace tunnel of the Khimti I Hydropower Project in Nepal Himalaya has been taken as a case. In this Project, the grouting contributed significantly in reducing permeability and meeting the contractual requirements concerning water leakage. The main focus of the paper is the effective use of pre-injection and post-injection cement grouting.

2 KHIMTI PROJECT

Khimti I Hydropower Project is located in the Himalayan region about 100 kilometers due east of Kathmandu, Nepal, see Figure 1.

The Project is owned by Himal Power Limited (HPL), Nepal and is among the first privately owned hydropower projects in the country. The Project started its commercial operation in June 2000. The Civil Construction Consortium (CCC) (a joint venture between Himal Hydro, Nepal and Statkraft AS, Norway) undertook the construction work on a turn key basis in 1996 and the construction work was completed in the summer 2001 after a one year defect liability period. The Project has an installed capacity of 60 MW and generates approximately 350 GWh electrical energy annually. To generate this energy the Project utilizes water from the very steep Khimti River, which has an average gradient of about 7 percent. Khimti I is a high head scheme, with a design discharge of $10.75\,m^3/s$ and a gross head of 684 meters. The total waterway length of the Project is approximately 10 kilometers (HPL, 2000).

The headrace tunnel, which is the major focus of this paper, is a pressurized tunnel with maximum and minimum static water head of 4 bars and 1.1 bars at its downstream and upstream end respectively. The tunnel is approximately 7.9 kilometers long with inverted D-shape and 14 square meters cross-section. Except first downstream end of 418 meters with full reinforced concrete lining, the tunnel is built based on Norwegian tunnelling principles and is unlined or shotcrete supported. Modern support means such as pre and post-grouting, steel fiber shotcrete, spiling and rock-bolts have been used.

2.1 *Project geology*

Geologically, the project lies in the crystalline Tamakoshi gneiss complex representating Kunchha Group of the lesser Himalaya. Structurally the area is bounded or surrounded by a major fault system of the Himalaya called "the Main Central Thrust (MCT)", see Figure 2. The rocks in the project area are mainly augen gneiss with inter-bedded bands of chlorite and talcose schist. This intercalation has been observed frequently at an interval of 5–10 meters at the downstream section of the headrace tunnel.

In contrast, at the upstream stretch the interval is longer and the rocks are more fractured and open jointed.

The foliation planes are generally striking towards northeast – southwest direction and dipping towards northwest. Since the project area is bounded with the Main Central Thrust (MCT) the rocks along the headrace tunnel are highly jointed, sheared, deeply weathered and deformed. The area is also influenced by several minor faults and weakness zones represented by very weak sheared schist and crushed zones.

Figure 1. Location Map of Khimti I Hydropower Project.

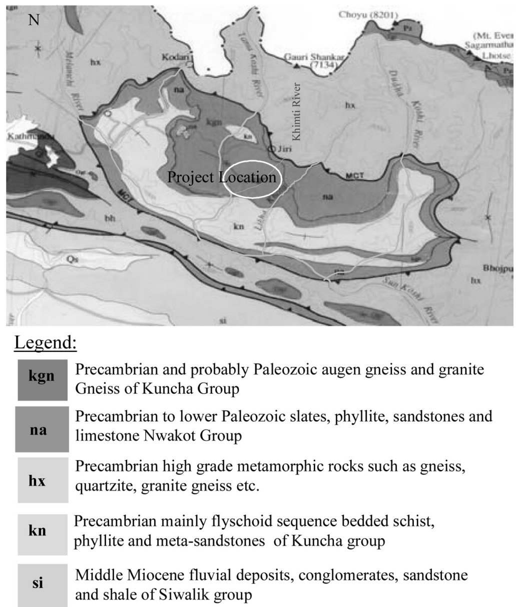

Figure 2. Geological map of the Project (source: Department of Mines and Geology, 1994).

2.2 *Actual rock mass conditions*

For a successful tunneling a method characterized by cost effectiveness and flexibility to adapt changing ground conditions is a must. In case of the Khimti headrace tunnel this concept is fully utilized with a maximum use of the self-supporting capacity of the rock mass. This is done by adopting a good system of water leakage control and by deciding the rock support at spot based on predefined rock mass classes and rock support procedures.

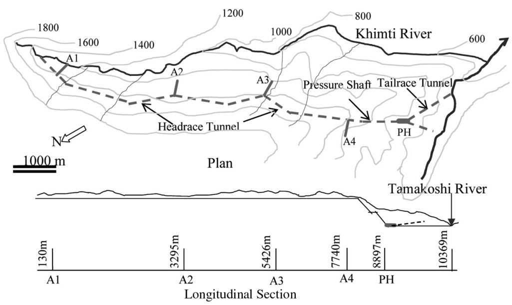

Figure 3. Plan and longitudinal profile of the Project.

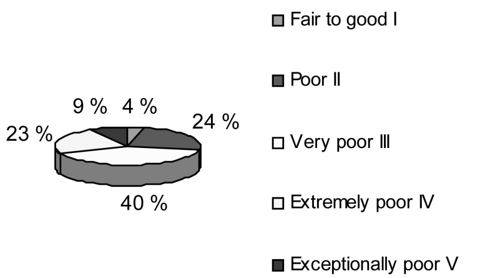

Figure 4. Headrace tunnel actual rock mass class distribution summarized after finish.

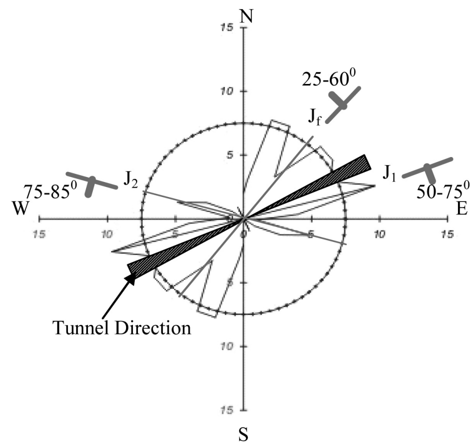

Figure 5. Joint rosette showing the orientation of the main joint sets and tunnel direction.

On the other hand, the planning phase investigation and predictions of the rock mass conditions along the tunnel in Khimti Project were rather poor and gave false impression to the contractor in planning the tunnelling work. The Design Basis Memorandum (HPL, 1995), which was the main guideline for the contractor in planning and detail design of the project, states that most of the tunnel length will be in sound rock. Exceptions were described to be the construction adits, the initial section close to intake and at the downstream end of the headrace tunnel, see Figure 3, where the tunnel was predicted to be in weathered rock where lining might be needed.

Accordingly, it was believed that the rock mass along the headrace tunnel would be of good quality and no measures such as water leakage control were considered at the initial phase of tunnelling. However, huge deviations have been found on the rock mass quality along the tunnel during construction. Less than 5 percent of the tunnel stretch was found to pass through sound rock, and the remaining 95 percents went through poor, very poor to extremely poor rock class (Bajracharya and Panthi, 2002), see Figure 4.

With respect to the discontinuity patterns, three sets of joints with occasional random joints have been observed along the tunnel alignment. The general strikes of the main foliation joints (J_f) have been found varying from N15 to 60°E. This is not very favorable relatively to the headrace tunnel alignment, which also is oriented in northeast/southwest direction, see Figure 3 and 4. The foliation joints are mostly dipping towards northwest with a varying dip angle of 50 to 60 degrees at the southern part of the tunnel (adit 4 area) and this trend changes gradually making the dip angle more flat with almost 25 degrees at its northern part. The joint set number one (J_1) is oriented with almost the same strike direction as the foliation joints and is very close to parallel to the tunnel alignment, but is dipping opposite to the foliation joints (dip angle 50 to 75 degrees towards southeast). Joint set number two (J_2) is oriented in northwest southeast direction with very steep dip angle (70 to 85 degrees) towards southwest, see Figure 5.

Relating to joint filling and alteration, most of the discontinuities at the southern section (downstream from Adit 3) of the headrace tunnel are filled with clay and bands of chlorite and talcose schist, and have been characterized as impermeable with respect to water leakage. In contrast, the discontinuities present at the northern section of the headrace tunnel are either open or filled with coarse grained, permeable silt materials. In this northern section several open joints with aperture of up to 10 cm have been observed. Moreover, the degree of weathering along the tunnel alignment varies greatly and classifies as medium to highly weathered according to ISRM (1978). In some sections the degree of weathering was so deep that decomposed organic soil was found in the tunnel. Especially the tunnel section 500 meters downstream from Adit 2 was deeply weathered (CCC, 2002). The valley side slope in this stretch of the tunnel is flatter (about 25 degrees) and the rock cover is approximately 100 meters.

Figure 5 indicates that the orientation and dip angle of two cross joints sets (J_1 and J_2) increase the possibility for large leakage from the headrace tunnel during operation as the valley side slope is oriented in northeast/southwest direction with a dip towards south, see also Figure 3. Moreover, most of the discontinuities upstream from Adit 3 are either open or filled with permeable materials that further increase the possibilities of large leakage during operation.

After tunnelling almost two and half kilometers of the headrace tunnel from different adits, the contractor realized this problem and had to change the tunnel construction methodology. He also incorporated a detail plan for preinjection grouting for the remaining tunnel stretch. However, there were only two alternatives left for water leakage control in the section of headrace tunnel where the excavation was already completed; full concrete lining or post-grouting. Even though it is well known that post-grouting is not as effective as pre-grouting in water leakage control, the contractor decided to go for post-grouting due to very tight construction schedule. Even if there were many uncertainties and risk connected to the postinjection grouting, success was achieved with tremendous amount of cost and time saving.

3 GROUTING CRITERIA AND PROCEDURE

The contractual limit for maximum water leakage from the 7,923 meters long headrace tunnel at the Khimti Project was defined as 150 liters per second, which gives specific leakage of 1.13 liters per minute per meter length of tunnel and is considered to be very strict with respect to water conveying tunnels. According to the Khimti civil works contract (KC2), failing to reach this criterion would have resulted in considerable economic penalties (CCC, 2002). In contrast, the traditional approach of full concrete lining in the tunnel would have been a very expensive and time consuming solution. In addition, concrete lining is not a fully watertight structure due to shrinkage cracks and there would be a need for contact grouting to make the tunnel water tight for full hydrostatic pressure.

Therefore, the decision was made to utilize the self supporting capability of the rock mass with modern means of support and to use mainly injection grouting in the headrace tunnel to reduce water leakage, see Figure 6.

As previously discussed, the best way to control the water leakage out of the tunnel would have been to carry out preinjection grouting ahead of the tunnel face during excavation. However, since almost 2.5 kilometres of the headrace tunnel was already excavated when the need of grouting was realized, both preinjection and post injection grouting was performed.

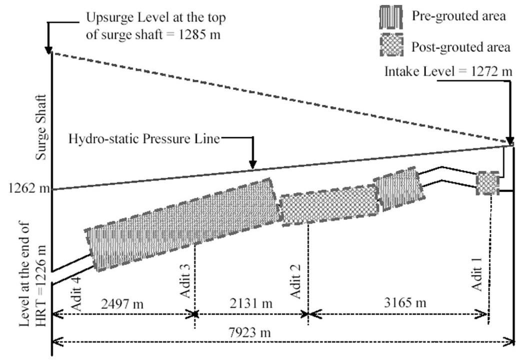

Figure 6. Headrace tunnel profile showing hydro-static line and areas with preinjection and postinjection grouting.

3.1 *Preinjection grouting*

Two criteria were mainly followed to decide whether there was a need for injection grouting or not. These were; (a) if the pressure of groundwater inflow to the tunnel is less than 1.5 times hydrostatic pressure during operation, (b) if the leakage through the rock mass exceeded a certain limit (water loss after pumping the water through exploratory drillhole with 1.5 times hydrostatic pressure). In general the criterion (b) is considered to be more reliable as criterion (a) is valid only in sections where groundwater inflow may occur during tunnel excavation and therefore (b) was used extensively for defining the preinjection grouting at the Khimti headrace tunnel.

First, one 21 meters long exploratory drillhole was drilled from the valley side wall of the tunnel face at an angle of 8 degrees relatively to the tunnel axis and slightly upwards with an angle of about 5 degrees. During the drilling regular observation was made to record possible water inflow as well as loss of flushing water, and the approximate depth of such incidents. Secondly, after drilling was finished, the water inflow (if any) into the tunnel was measured using packer and flow-meter. After that water leakage test was carried out by pumping water into the drillhole (through the packer) with 1,5 times the hydrostatic pressure. Finally, if the water leakage exceeded one litre per minute per meter length (lugeon value exceeding one) preinjection grouting was recommended. For preinjection grouting 12 drillholes with 21 meters length were drilled around the tunnel perimeter at an angle of 8 degrees relatively to the tunnel axis to establish a grouting cone, see Figure 7.

The preinjection grouting was performed using micro cement (Reocom) with some accelerator added to it. Three different mixes were used, starting with mix one having water/cement ratio 2. If mix one reached 1.5 times the operational hydrostatic pressure in the

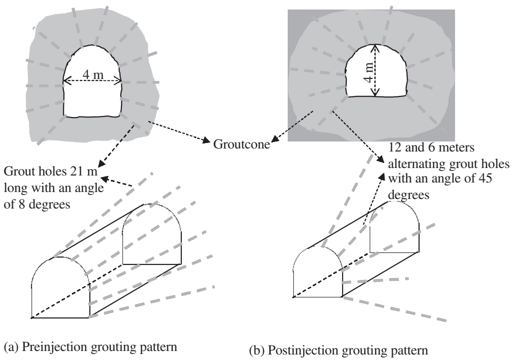

Figure 7. Used preinjection and postinjection grouting pattern.

tunnel, or the grout volume exceeded 200 litres in each hole, the grouting was stopped. However, if the grout pressure after 200 litres was still below 1.5 times the hydrostatic pressure, mix two with water cement ratio 1.5 was used with criteria as for mix one. In case of failing to reach the criteria, mix three with water cement ratio one was used. Finally, sawdust was added in case the desired pressure was not achieved.

3.2 *Postinjection grouting*

The orientation of joints shown in Figure 5 and the open character of the joints gave clear indication that there was a strong need for postinjection grouting at the section of tunnel where no preinjection grouting was performed during early stage of tunnel excavation, see Figure 6. Nevertheless, to be able to concentrate the postinjection grouting operation only to the required sections of the tunnel, it was very important to identify the areas from where maximum leakage could be expected. To verify that, early test water filling was performed in the headrace tunnel. A temporary plug was constructed at the downstream end of the headrace tunnel at chainage 7,505 meters, since the construction work at the pressure shaft and powerhouse were still not completed. The test water filling carried out in January 2000 indicated considerable leakage from the tunnel (approximately 700 liters per second), see Table 1.

As expected, the leakage was insignificant from Adit 3 and Adit 4, giving indication that the preinjection grouting was effective. On the other hand, the leakage from Adit 2 reached approximately 200 liters per second, giving indication that unacceptable leakage was occurring from the headrace tunnel near the Adit 2 area, where no preinjection grouting had been performed.

Based on the test water filling results and review of the geological tunnel log, a plan was made for the implementation of comprehensive postinjection

Table 1. Water leakage measured after test water filling.

Headrace tunnel locations	Leakage (Q) (liters/second)
Total leakage from headrace tunnel	700
From Adit 1 tunnel near concrete plug	2.6
From Adit 2 tunnel near concrete plug	200
From Adit 3 tunnel near concrete plug	0.4
From temporary plug at Adit 4	1.8
From remaining headrace tunnel	495.2

grouting. Main focus during grouting was on the ungrouted 910 meters upstream and 330 meters downstream stretch of the headrace tunnel from Adit 2 (chainage 2385 to 2965) where the joints were very open and the rock mass highly weathered. In addition, some length of the headrace tunnel near Adit 1 area where it was suspected that water was leaking, was also grouted, see Figure 6. Since the joints in these sections of the tunnel were more open, ordinary cement was used for the grouting. In addition, five percent bentonite clay was added in the grout mix to increase grouting effect. Four bars maximum pressure was used for postinjection grouting with a grouting pattern as shown in Figure 7-b. As indicated in the Figure, a 45 degrees angle relatively to the tunnel axis was used for drilling of the grout holes with alternating length of 12 and 6 meters.

4 GROUTING RESULTS

Both pre- and postinjection grouting were found to be very effective and economical solutions for controlling water leakage from the pressurized headrace tunnel. Compared to preinjection, the postinjection grouting was found somewhat difficult and challenging as there was a tendency of grout coming out in the tunnel wall, and due to very open joints it was often difficult to achieve the desired pressure. Consequently, the grout consumption was considerably higher than for preinjection grouting, and the grouting operations more costly. Nevertheless, the grouting costs were found much lower than those of concrete lining. The total consumption of grout material in the headrace tunnel is presented in Table 2.

The final water filling of the tunnel in March 2000, and leakage measured according to the performance guarantee in the civil works contract (KC2), gave a leakage of 120 liters per second in the 7,923 meters long headrace tunnel. This water leakage gives specific leakage (q) of 0.91 liters per minute per meter tunnel length, see Table 2, which is far below than the contractual leakage limit.

An analysis has been done to find out whether there exists correlation between the specific leakage

Table 2. Grouting results and final water leakage measured.

Activity descriptions	Units	Quantity
A. Preinjection grouting		
Total grouted length	m	4612
Grout pressure used	bars	2.8–6
Total grout consumptions	kg	754,975
Specific grout consumption	kg/m tunnel	164
B. Postinjection grouting		
Total grouted length	m	1359
Grout pressure used	bars	4
Total grout consumptions	kg	941,260
Specific grout consumption	kg/m tunnel	693
C. Water leakage after final water filling		
Headrace tunnel length	m	7923
Total water leakage	liters/second	120
Specific water leakage	lt/min/m tunnel	0.91

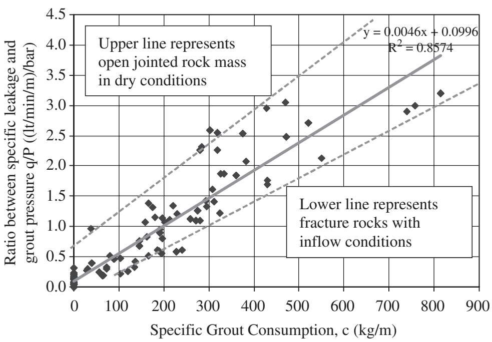

Figure 8. Correlation between specific leakage, grout pressure and specific grout consumption.

(q), the grouting pressure (P) (1.5 times static pressure during operation) and the specific grout consumption (c) for preinjection. As shown in Figure 8, a correlation has been found that might be used to estimate preinjection grout consumption for future tunnels in similar ground conditions.

Analysis has also been carried out of the specific grout consumption with respect to the rock mass quality of the tunnel section were both preinjection and postinjection grouting was performed. The results of this analysis are presented in Figure 9.

Figure 9 clearly indicates that the grout consumption for both preinjection and postinjection grouting is highest for rock class II and III. This is because the rock mass in these categories of rock class are typically open jointed. Poorer rock mass quality typically has clay filled joints, deformed rock mass and folding, which reduce the permeability. Figure 9 also indicates that the postinjection grouting has a considerably higher consumption than pregrouting.

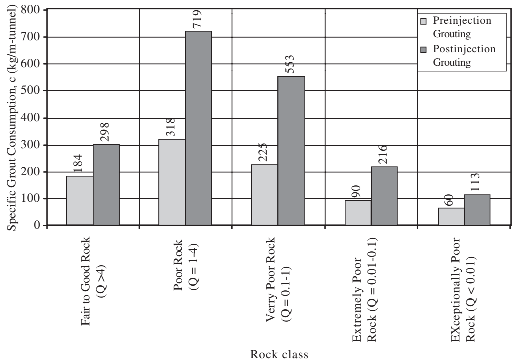

Figure 9. Relation between specific grout consumption and rock mass class.

5 CONCLUDING REMARKS

By applying systematic preinjection and postinjection grouting at the headrace tunnel of Khimti I Hydropower Project it was not only possible to control the water leakage to the contractual limit, but also to reduce the cost and time. The grouting carried out at this project clearly demonstrates that the use of systematic preinjection as well as postinjection grouting played a very significant role in the reduction of water leakage from the pressurized unlined / shotcrete lined water conveying headrace tunnel. Moreover, the grouting has played a major role in improving the rock mass quality by cementing the cracks and joints with grout material and reducing the need of excessive rock support.

The factual based analysis carried out has confirmed that there exists good correlation between the specific leakage (q), grout pressure (P) and specific grout consumption (c) that could be used for estimating grout consumption for future water conveying tunnels in similar rock conditions. The Khimti case also illustrates that the grouting operations are not easy tasks especially when it comes to postinjection grouting, which may reach a cost of many times that of preinjection grouting. Therefore, systematic preinjection grouting should be used as much as possible. Finally, to achieve a good grouting result at lowest possible cost, a good understanding of the ground conditions is of great importance.

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