The Use of Self Supporting Capacity of Rock Mass for Sustainable Hydropower: An Analysis of the Middle Marsyangdi Headrace Tunnel, Nepal

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Abstract: The history of hydropower development in the Himalaya indicates that many tunnels have suffered from cost overruns and delays. These issues are directly dependent on the quality of rock mass and the permanent rock support applied in underground excavation. Right judgment and proper evaluation of the self supporting capability of the rock mass and the use of optimum rock support systems help considerably in reducing construction cost and delays. This paper examines such issues as geological conditions in the Himalayas and varying approaches and costs in tunnel construction. An assessment is made regarding the exclusion of permanent concrete lining in the headrace tunnel of the 72MW Middle Marsyangdi Hydroelectric Project in Nepal. The project has 5.2 km fully concrete lined headrace tunnel that passes through fair to poor rock mass. The evaluation is based on the use of actually recorded rock mass quality of the headrace tunnel during construction and rock support principle used at the comparable Khimti Hydro Project headrace tunnel. The evaluation includes calculation of equivalent tunnel section for similar headloss, stability analysis, assessment of possible water leakage, and required injection grouting measures. We conclude that the headrace tunnel without permanent concrete lining was possible and would have been equally stable, at considerable financial savings.

Key words: Equivalent tunnel section, squeezing, tunnel lining, stability analysis, leakage control, hydropower, Nepal

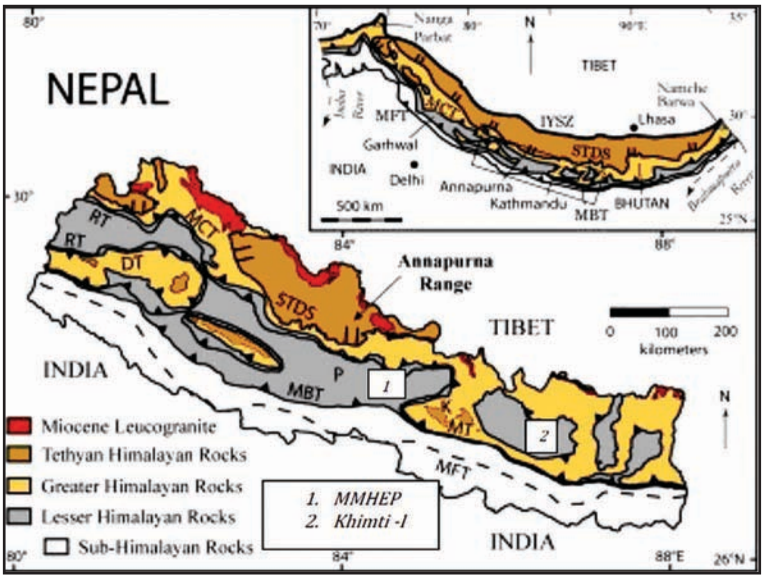
Introduction

A basic philosophy in tunnelling is that the extent of installed rock supports should reflect actual rock mass conditions. Wherever possible, the self supporting capacity of the rock mass should be fully utilized and the amount of rock support should be kept to a minimum. Selection of the tunnel rock support should be based on a sound understanding of the rock mass characteristics and stability problems of the tunnel in question (Nilsen and Thidemann 1993). When, for safety reasons, the hydropower industry started using underground waterways from the early 1950s they brought the steel pipes with them and later, since the 1960s, unlined pressure shafts have been successfully used. Experience with pressure tunnels and shafts over a long period of time has shown that if certain design rules are followed and certain geological and topographical conditions are avoided the rock mass is capable of containing water pressure up to 100 bars, or the equivalent of 1000 m of water head (Broch 1982).

The most challenging aspect of unlined water tunnels, however, is the control of leakage, which may be controlled by injection grouting. Injection grouting not only plays a vital role in improving the rock mass quality, but it also increases safety and reduces economic loss (Panthi and Nilsen 2008). As an example of the effectiveness of grouting, Karlsrud (2002) has indicated improvement in the hydraulic conductivity of rock mass closest to the tunnel periphery. Injection grouting can reduce leakage by approximately 1/25 to 1/100 times over un-grouted rock mass if systematic pre-injection grouting is carried out. Barton et al (2001) has pointed out that considerable improvement in the overall Q-value and Q-value parameters may be achieved by using pre-injection grouting. In dry conditions, pre-injection grouting may improve rock mass quality with one quality class and in wet conditions pre-injection grouting may improve the rock quality with two or even three quality classes. Overall, effective pre-injection grouting will reduce rock mass permeability considerably, and also help reduce tunnel convergence and rock support in the tunnel.

In the Himalaya, fully concrete lined tunnels as a final rock support has been the tradition even for low pressure headrace tunnels, but can be avoided if the self supporting capability of the rock mass is exploited. There are headrace tunnels in this region where the principle of self supporting capability of the rock mass is fully used. Concrete lining has been used only in very needy segments where a tunnel crosses, for example, weakness zones or fault zones. Headrace tunnels on the Khimti and Modi Khola Hydropower Projects are two examples of this endeavor and both tunnels are located in the lesser Himalayan region of Nepal. These two headrace tunnels were completed a decade ago and are functioning quite well.

This paper evaluates and analyzes the possibility of avoiding concrete lining in the headrace tunnel on the Middle Marsyangdi Hydroelectric Project (MMHEP) in Nepal, which came in operation in 2008. The analysis tries to exploit observed rock mass characteristics data collected during excavation, assesses possible leakage, and estimates the injection grouting requirement for controlling the leakage from headrace tunnel without concrete lining. The tunnelling principles used at Khimti-I headrace tunnel located close to similar geological setting (i.e., in the lesser Himalaya) is used as reference. See Map 1.



Map 1. Location of Projects Concerned

Case Description

Reference Case: Khimti-I Hydroelectric Project

The Khimti-I Hydroelectric Project is located about 100 kilometers east of Kathmandu (Map 1). The project was developed by Himal Power Limited (HPL) and is the first privately invested and owned hydropower project in Nepal under the BOOT concept. The Civil Construction Consortium (CCC) of Statkraft Anlegg of Norway (now NCC) and Himal Hydro of Nepal carried out the construction work on a turnkey basis. This project has been successfully operating since 2000. See Figure 2 for tunnel alignment, layout and section.

The project is run-of-river project with an installed capacity of 60 MW, utilizes 682 m gross (667 m net) head, and has design discharge of 10.75 m^3/s. The project has fully pressurized headrace tunnel with a length of 7888 m (inverted D shaped with area around 14 m^2). Apart from 418 m downstream end, the headrace tunnel is shotcrete lined or unlined. The pressure in the headrace tunnel ranges from maximum and minimum static water head of 4 bars and 1.1 bars at its downstream and upstream ends, respectively. The tunnelling is based on Norwegian tunnelling principles and uses modern tunnel support means such as pre- and post-injection grouting, steel fiber shotcrete, spilling and rock-bolts. The contractual limit on leakage is 150 liters per second or 1.13 liters per minute per meter tunnel length (Panthi 2006).

Geologically, the project lies in the crystalline gneiss complex surrounded by major faults system in the Himalaya; the "Main Central Thrust (MCT)". As a result the rock mass are highly jointed, faulted and sheared. The mica gneiss and mica schist are frequently intercalated at the downstream, while upstream this intercalation interval is longer but the rock mass is more fractured and open jointed. During planning the rock mass along most of the headrace tunnel was expected to be of good quality except in some sections near the intake and the downstream end of the headrace tunnel and in sections with weakness zones (Panthi and Nilsen 2005). In contrast, large deviations were found over what was predicted (Figure 2a and 2b).

Middle Marsyangdi Hydroelectric Project (MMHEP)

MMHEP is located in the central Himalaya in Nepal (Map 1). The project is a run-of-river project with an installed capacity of 72 MW. The Marsyangdi river is a perennial snow fed river with a catchment of 2729 km^2. The project is designed with design discharge of 80 m^3/s and has gross head of 110 m (net head 98 m). The project has a 5114 m long fully concrete lined headrace tunnel with 5.45 m finished diameter and is circular in shape. The excavation shape of the headrace tunnel was a horse shoe and had a 6.4 m excavation diameter. See Figure 1 for project layout, geological section and observed and predicted rock mass quality.

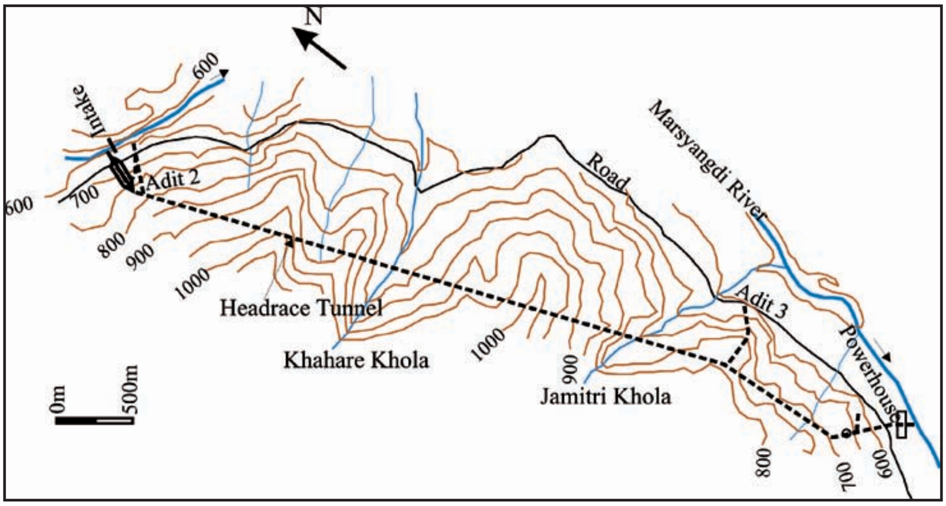


Figure 1. Project Layout (based on MMHEP, NEA)

In 1994 the Nepal Electricity Authority (NEA) carried out a feasibility study and in 1997 the project was upgraded by Lahmeyer International GmbH, Germany. The detailed design and construction supervision was carried out by Fitchner Joint Venture (Fichtner GmbH, Germany, along with Statkraft Engineering of Norway and Consulting Engineer and Salzgitter GmbH Germany). The civil contractor of the project is DDC JV (Dywidag, Dragados, and CWE China Water and Engineering). Construction commenced in 2001 and was planned to be completed in 2006. The project faced considerable delay, however, and finally came into operation in the summer 2009.

Geologically, the project is located in the same geological setting as Khimti in the Lesser Himalaya. Up to chainage 700 m from intake the headrace tunnel passes through massive quartzite, but there after fractured phyllitic quartzite and the intercalation of quartzitic phyllite and sheared phyllite dominates the middle segment of the headrace tunnel. At the downstream end the headrace tunnel mainly consists of the intercalation of phyllite and meta-sandstone (Figure 3). Along headrace tunnel (HRT), minimum Rock Mass Rating (RMR) is found to be 12 in weak sheared phyllite to maximum RMR to 64 in massive qaurtzite at the upstream segment of headrace tunnel near Adit-2. The average RMR in HRT is found to be 34.82.

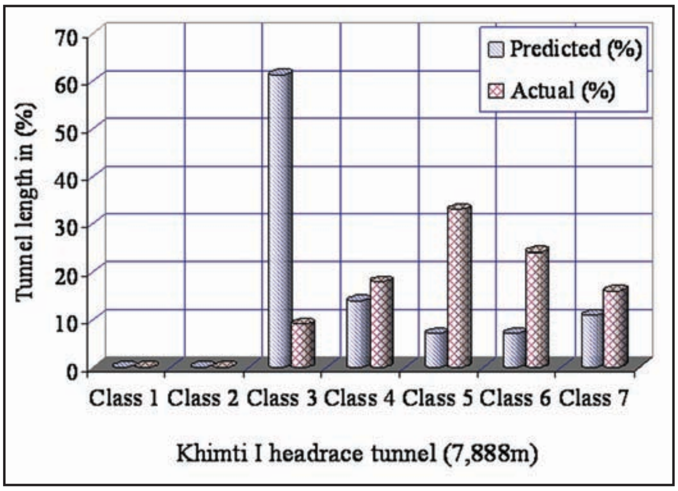


Figure 2b. Observed and Predicted Rock Mass Quality along the Headrace Tunnel in Khimti-I (Panthi 2006).

Analysis Methodology

Data on rock mass quality, rock support system actually applied, and data on convergence measurement along the headrace tunnel are used here for the assessment. Analytical and semi-empirical methods have been used for the stability analysis for both fully concrete lined and hydraulically equivalent tunnel section without concrete lining. The leakage analysis through headrace tunnel without concrete lining is carried out using the correlation established by Panthi (2006) and the analytical method proposed by Tokheim and Janbu in 1984. For leakage control measures, pre-injection grouting is suggested and the cost assessment is based on the grout consumption at Khimti headrace tunnel with correlation established by Panthi and Nilsen (2005, 2008) and Panthi (2006), assuming the correlation is also valid for MMHEP.

Stability analysis is carried out in critical sections of the headrace tunnel and the support requirement is assessed based on used rock support system of the completed headrace tunnel. Critical sections are those where the convergence was measured during excavation, since convergence measurement was carried out only in tunnel sections where tunnel deformation was observed. Estimation and comparison of cost effectiveness is done based on the Bill of Quantities (BOQ) quoted by the contractor in MMHEP; and some adjustment is made based on the current market rates.

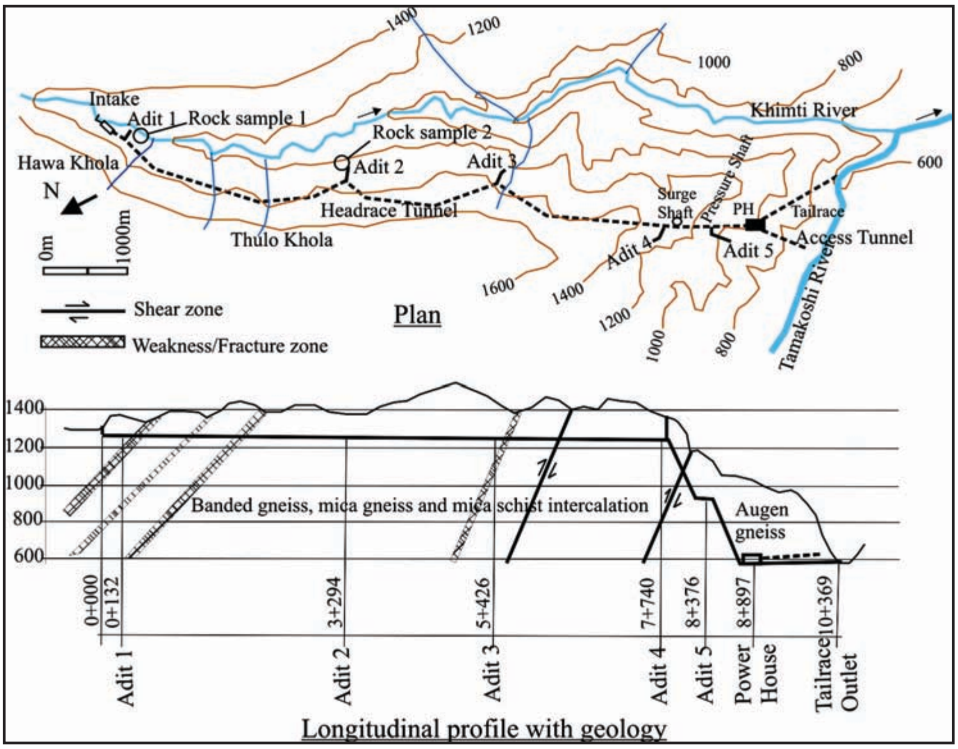


Figure 2a. Geological Section – Khimti I Hydropower Project (Panthi 2006).

Hydraulic Equivalent Cross Section

Estimation of Manning M based on Khimti Headrace

Khimti-I is a high head project, which has a gross head of 682 meters. The net head was measured during the performance test, which amounts to 667 meters resulting the net head loss of 15 m (Panthi 2009). The total tunnel length of the waterway is approximately 10 km consisting of approximately 7.9 km headrace, about 1 km penstock

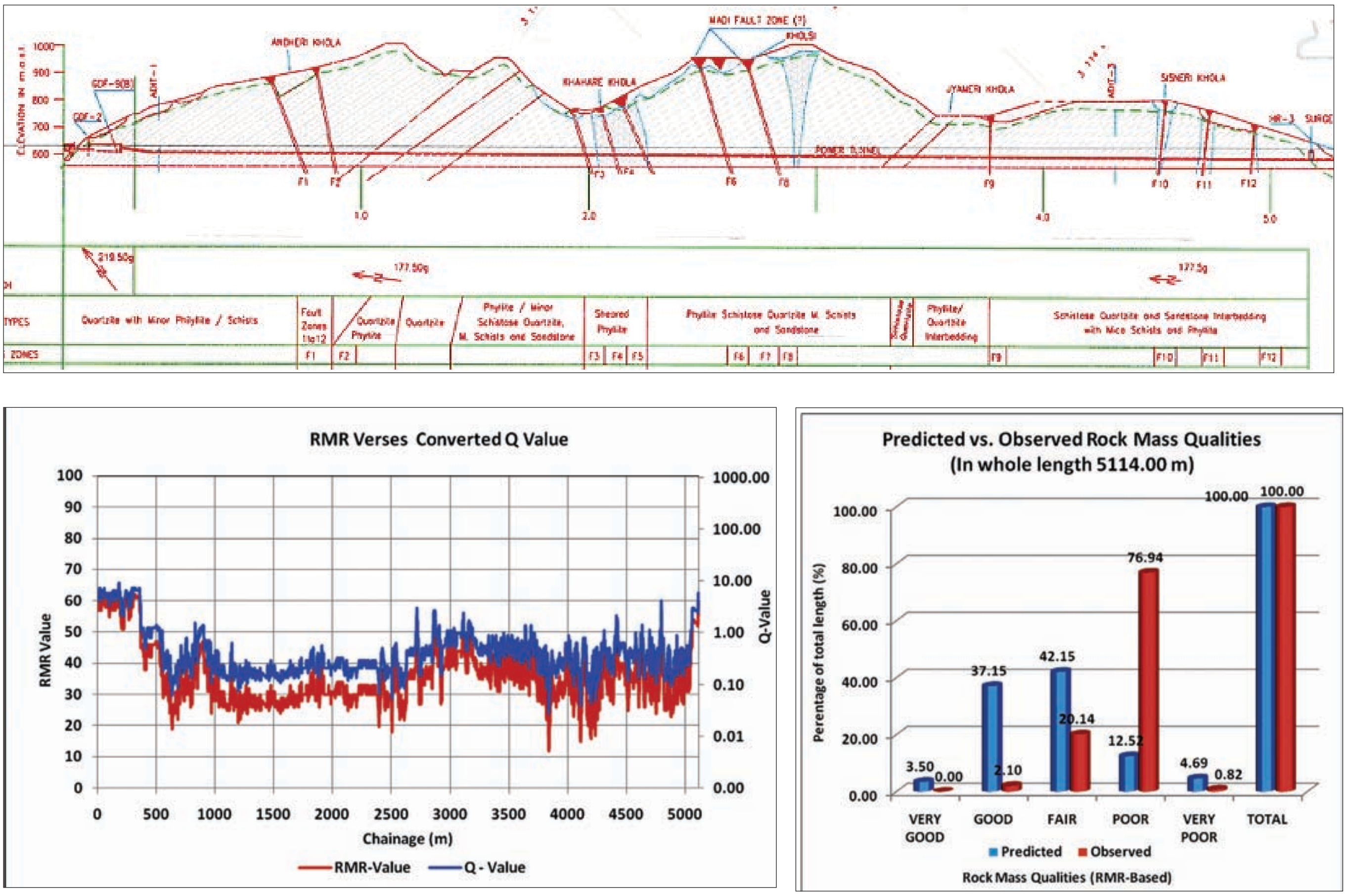


Figure 3. Geological Section and Observed and Predicted Rock Mass Quality along the Headrace Tunnel in MMHEP (Shrestha 2009)

shaft and about 1.5 km tailrace tunnel. The turbine system in Khimti is pelton wheel. The performance test carried out to measure the hydraulic efficiency of the waterways found satisfactory.

The length of the waterway from tunnel inlet to pelton turbine center is 1003.60 +7888.00+11.60 = 8903.20 m. Considering all possible minor headloss, the Manning's number (M) of the shotcrete lined tunnel of Khimti-I was back calculated and found to be M = 35.7.

The excavation methods in both headrace tunnels were conventional drilling and blast. The average round length of the drilling and blasting, however, was more in Khimti than in MMHEP. The average round depth (pull length) in MMHEP is 1.64 m (Shrestha 2009), while the average round depth (pull length) in Khimti is about 2.25m. From this we can say that the value of M obtained in Khimti could safely be used as reference M for MMHEP headrace tunnel without concrete lining. Hence, this M value is used to calculate the equivalent cross section with same headloss, as that is the case for fully concrete lined headrace tunnel at MMHEP.

Equivalent cross section for MMHEP headrace

Due to closeness of the sloping topography, the relatively higher water pressure and the surge shaft location, it is assumed that full concrete lining at the downstream end of headrace tunnel was needed. Therefore, out of the 5114 m headrace tunnel, 214 m downstream end near surge shaft is assumed to be fully concrete lined and the remaining 4900 m may be left without full concrete lining. The headrace tunnel shape will then be an inverted D shaped with a base width B as shown in the Figure 4 (left and middle). The permanent rock support will be systematic bolting, invert concrete lining, shotcreting and pre-injection grouting. For the calculation of

Table 1. Waterway System and Manning's Number at Khimti-I.

Waterways	Length	Mean Diameter / Area	Manning Number
Steel Lined Penstock and Bifurcation Section			
Upper pressure shaft	439.95 m	1.94 m	80
Intermediate horizontal section & lower pressure shaft	563.65 m	1.80 m	80
Bifurcation penstock pipe	11.60 m	1.53 m	80
Headrace tunnel concrete lined section (Inverted 'D' shaped)			
Downstream concrete lined section	418.00 m	9.1 m^2	60
Shotcrete lined or Unlined section (Inverted 'D' shaped)			
Headrace tunnel excavated from Adit-3 (Section-1)	1005.00 m	13.50 m^2	'M' to be calculated
Remaining shotcrete lined section (Section-2)	6465.00 m	11.60 m^2	

a hydraulically equivalent tunnel section of HRT, Manning's number 'M' is used as 35, which is equivalent to the Khimti headrace tunnel.

Some features of the existing tunnel system with concrete lining at MMHEP are as follows:

Gross head (H_g) = 111.00 m
Design discharge (Q) = 80 m^3/s
Net head (H_n) = 98.00 m (overall)
Total headrace tunnel length (L)= 5119.00 m
Diameter of the tunnel (D) = 5.45 m (circular shaped)
Manning roughness factor (n) = 0.0167 *(Design Document 1999)*
Manning Number (M) = 60
Head loss in headrace tunnel = 12.0 m

Assumptions for equivalent tunnel section:

- Base width is equal to height of the tunnel with radius half of base width (H=B and R=B/2).
- Average thickness of shotcrete (t) is 20 cm both in wall and roof and concrete lining thickness (t) is 30 cm in addition to shotcrete thickness.
- Manning number 'M' for concrete lined tunnel (214 m) is 60 and for shotcrete lined tunnel (4900 m) is 35, respectively.

With the above assumptions, the excavation width of the tunnel will be 6.70 m, which will give an excavation area of 40 m^2. The difference in some geometrical parameters for existing headrace tunnel with full concrete lining as final support (Figure 4 - right) and equivalent headrace tunnel without concrete lining as final support is presented in Table 2.

Assessment of Tunnel Convergence

The long term stability of the headrace tunnel is of great importance. Therefore, it is necessary to carry out stability analysis in this respect. In the following, the headrace tunnel is analyzed for three different main aspects of tunnel stability.

Table 2: Main Geometrical Parameters Between Section With and Without Concrete Lining.

Description	Existing horse shoe shape with concrete lining as final support	Inverted 'D' shaped tunnel with shotcrete lining as final support	Difference
Tunnel excavation area (m^2)	33.97	40.04	6.07
Total Perimeter (m)	21.35	23.92	2.57
Wall and crown perimeter (m) for shotcrete and steel ribs	15.65	17.22	1.57
Invert length (m) for invert concrete	5.70	6.70	1.00

The maximum convergence recorded at chainage 1671 m of the headrace tunnel is 169 mm; i.e., 2.73% of the total width. At this chainage the rock mass consists of sheared phyllite. The data record shows that mostly the headrace tunnel had less than 2.5 % tunnel convergence (Figure 5- left). Though the rock mass along the tunnel alignment varies from fair to poor with a maximum overburden of 420 m, the tunnel convergence is fairly low and is in an acceptable range in regards to long term stability of the tunnel. In a few sections (e.g., chainage 451, 775, 1671, 2806, 3450, 3850) total convergence is equal

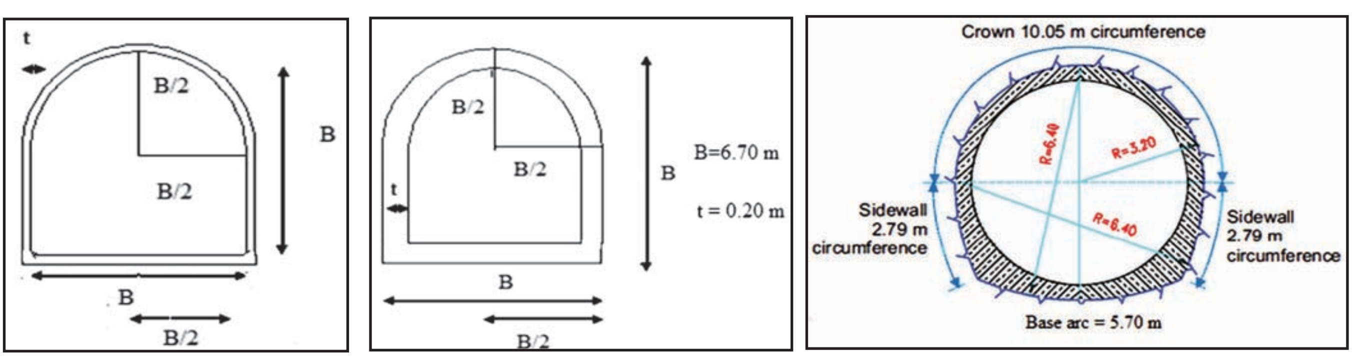


Figure 4. Proposed shortcrete lined and concrete line sections (left and middle) and existing concerete line section (right)

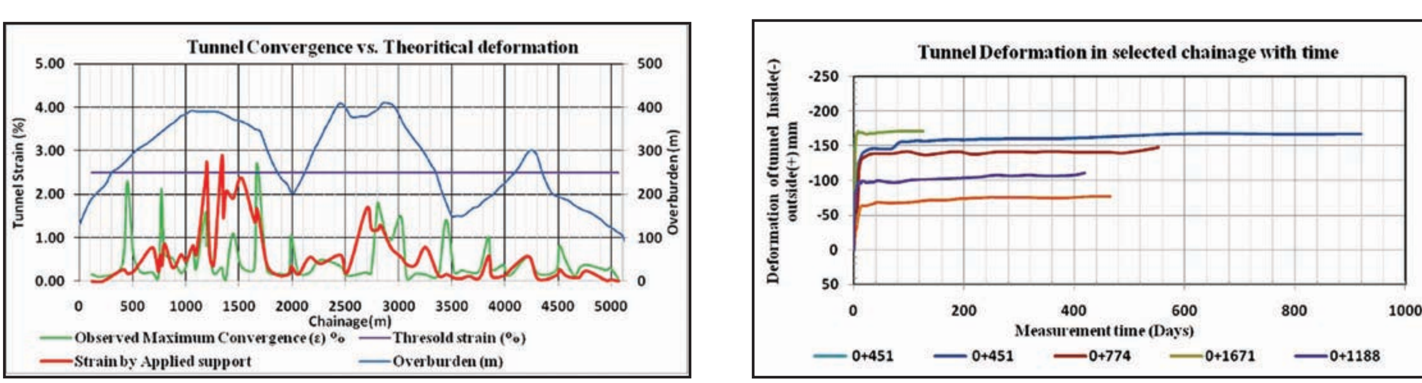


Figure 5a. Potential Sqeezing and Observed Convergence (Shrestha, 2009).

to or exceeds 2.5%. Similarly, the time dependent convergence shows that the overall movement of the tunnel wall stabilized after about two weeks of excavation and no significant movement occurred afterward (Figure 5- right).

A squeezing analysis is carried out with exactly the same rock support system in the existing tunnel for the recommended tunnel with concrete lining in the invert only. The analysis is carried out in the same sections where tunnel convergence was measured data during construction. The approach of Hoek and Marinos (2000) for tunnel squeezing is used. For the analysis the approximate support pressure is estimated based on Hoek (1998), Hoek and Brown (1980), and Brady and Brown (1985). The estimated support pressure used to assess the tunnel strain is shown in Figure 6a. As can be seen most of the tunnel sections indicate that the actual convergence is close to or less than calculated. More importantly, most of the tunnel gives total tunnel convergence (strain) below 2.5%, which represents minor squeezing and is within the acceptable limit.

It needs to be highlighted here that in the headrace tunnel no final concrete lining was carried out until 2006, meaning that the tunnel was in very stable condition for more than 3 years. This suggests that the applied support system is more than adequate in regards with long term stability. However, tunnel sections where applied shotcrete support is damaged or cracked and rock bolts are deformed need to be repaired before the tunnel is filled with water and comes in operation.

Potential Leakage Analysis

One of the most important aspects and challenges of the unlined or shotcrete lined water tunnel concept is how to control water leakage through the tunnel while in operation under full hydrostatic pressure. Panthi (2006) and Panthi and Nilsen (2008) suggest the threshold of 1 to 1.5 liters per minute per tunnel meter length for hydropower tunnels, depending upon their length and design discharge. The Khimti headrace tunnel, which is unlined/shotcrete lined, had the contractual limit for the leakage with 150 litres/second in the whole waterway, which gives about 1.1 liters per minutes per tunnel meter length.

A tunnel cannot be absolutely impermeable and it is likely that some degree of leakage through even fully concrete lined tunnels will occur. In fully concrete lined tunnels the observed leakage without any special measures other than water stops in the construction joints has ranged from about 0.1 to 0.4 liters per minute per meter tunnel (Benson 1989). Properly applied and controlled shotcrete can work the same as concrete lined in terms of strength and water tightness; and, in addition, it is cost effective in most cases (Benson 1989). Problems might occur; however, if there is considerable water inflow into the tunnel before the shotcrete is hardened and if there is a lack of good adhesion of shotcrete to the rock surfaces. This problem can be solved by means of advance injection grouting into the rock mass.

The leakage analysis along the Middle Marsyangdi headrace tunnel is carried out using both the Tokheim and Janbu (1984) method and a method suggested by Panthi (2006). The first method is based on the flow theory and consists of input parameters; i.e., rock mass permeability, geometric factor, hydraulic water head, etc. (Figure 6a). It is interesting to note that the estimation of hydraulic conductivity is crucial in this method since the hydraulic conductivity values are mostly estimated in power of $10^{(-x)}$. An increase in power value by one the leakage will increase by almost 10 times. Therefore, leakage analysis is very sensitive with the use of this method. Still, this approach of leakage estimate is used so that it is possible to see the sensitivity of the method. The hydraulic conductivity (k) of the rock mass is roughly estimated based on Bell (2007).

The second method suggested by Panthi (2006) is based on correlation between some Q-value parameters and leakage through probe hole drilled to check the injection grouting

$$Q_w = \frac{2\pi \times K \times L \times p}{\mu_w \times G}$$

$$G = \ln \frac{(2D - r) \times (L + 2r)}{r[L + 2(2D - r)]}$$

Where,
- Q_w = Inflow rate (m3/s)
- K = Specific permeability (m2)
- L = Length of tunnel (m)
- p = Potential active head (Pa)
- μ_w = Dynamic viscocity of water (kg/(m.s)
- G = Geometry factor
- D = Distance from tunnel invert to ground water table (m)
- r = Equivalent radius (m)

Figure 6a. Analytical Method (Tokheim and Janbu 1984).

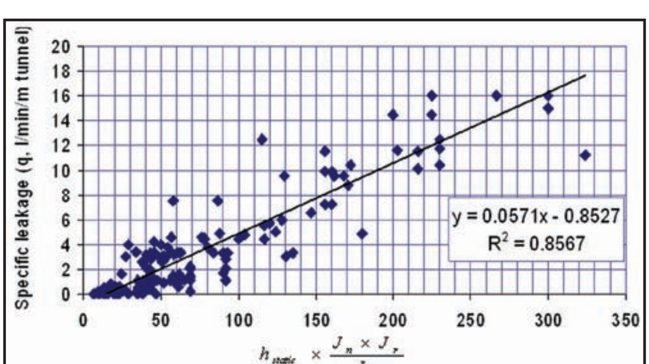


Figure 6b. Correlation of Specific Leakage to Joint Parameter and hstatic (Panthi 2006).

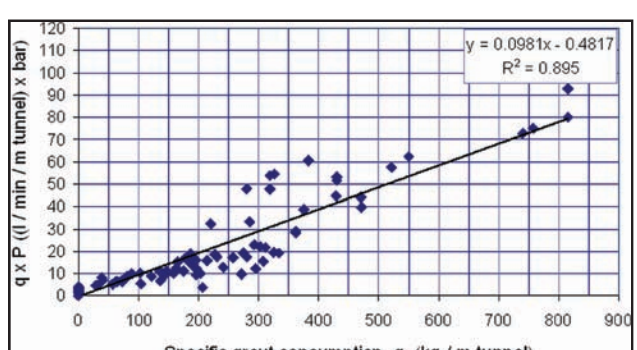


Figure 6c. Specific Grout Consumption at Khimti Headrace Tunnel (Panthi 2006).

at Khimti headrace tunnel (Figure 6b). For estimating required quantity of cement grout Figure 6c is used. The actually measured mean values of some Q-value parameters at Marsyangdi headrace tunnel that are required for this correlation to calculate the possible leakage are given Table 3.

Table 3. Representative Joint Parameters of Q-Values in HRT, MMHEP.

Chainage	Jn	Jr	Ja
	Representative Value	Representative Value	Representative Value
0.00 to 377.70	9	1.5	3
377.50 to 5114.00	6	2	8

The analysis results of the specific leakage and corresponding grout consumption along Marsyangdi headrace tunnel using these two approaches are presented in Table 4 and in Figures 7a and 7b.

Evaluation of Cost

Evaluation on costs is made based on the cost parameters for additional excavation and rock support due to change in tunnel cross section to achieve similar head loss as for fully concrete lined tunnel. The cost for injection grouting to control the leakage through the headrace tunnel in case of no final concrete lining is also included in this evaluation. Exchange rate used for the calculation is 1 USD= 80 NRs. Results of the calculation are presented in Tables 5, 6, 7 and 8.

The analysis indicates that there was a possibility to save the direct cost for tunnel construction by approximately 50 percent. Even if we consider the sensitive Tokheim and Janbu (1984) approach for leakage estimation the saving of about 30% would have been achieved.

This saving, too, is more on the safe side due to following reasons:

1. The value of M considered is only 35 but according to Benson (1989) it is possible to achieve its value close to 55 if shotcrete lining is smooth. In this case the difference in cross section would have been not that big, meaning insignificant extra cost for excavation and additional support.

2. The unit rate for concrete used is from BOQ and represents the unit rate of 1999. The today's rate is considerably inflated and the rate for injection grouting is of the year 2007.

3. It was possible to reduce the thickness of shotcrete to 15 cm in average if pre-injection grouting was adopted.

Conclusion

The self supporting capability of the rock mass should be utilized for sustainable development of the hydropower projects in the Himalayas. Innovation in rock support technology is recommended to be fully utilized to make hydropower projects economically attractive, sustainable and environmentally friendly. This paper demonstrates that similar quality tunnelling is possible to achieve without final concrete lining. The use of such approach, however, demands high level understanding of the rock mass leakage characteristics and topographic conditions of the project.

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Table 4. Specific Leakages Based on Tokheim and Janbu (1984) and Panthi (2006)

Methods	Specific Leakage (l/min/tm)				Specific grout consumption (kg/m)				Total Grout (tons)
	Average	Max	Min	St.dev	Average	Max	Min	St.dev	
Tokheim and Janbu (1984)	14.06	24.38	2.94	7.07	785.03	1718.53	146.33	502.15	3980.81
Panthi (2006)	5.98	13.95	3.16	2.23	324.58	572.94	121.20	134.37	1694.54

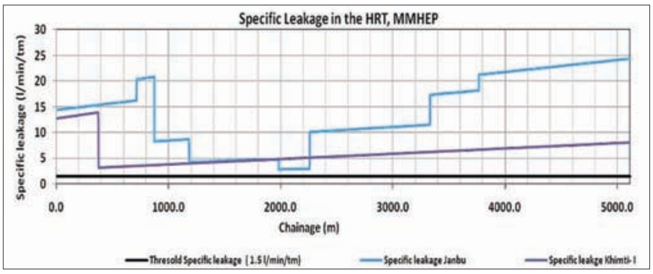

Figure 7a. Specific Leakage at HRT, MMHEP .

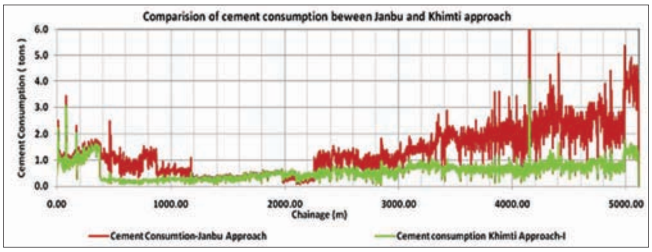

Figure 7b. Grout Intake in HRT, MMHEP (Shrestha 2009).

Table 5. Additional Costs of Excavation and Supports Between for Tunnel Without Concrete Lining.

Description	Unit	Rate (NRs.)	Quantity	Total cost (NRs)	Remarks
Addtional Excavation (Total 5114 m)				46,200,859	
Additional fiber shotcreting throughout with length in wall and crown length 1.57 m-15 cm thick	m2	5,500	7,693	42,311,500	Market rate
Additional fibre shotcreting all around (5 cm thickness for final repair wherever required)	m2	2,000	21,095	42,190,000	Market rate only for around 25% of 4900 m
Additional invert concrete (20 cm thick)	m3	5,588	1,005	5,615,940	BOQ
Steel arch additional 1.57 m length (HEB 120 -1966 nos- 26.7 kg /m)	tons	120,000	83	9,960,000	
Additional cost on excavation and support				146,278,299	Equivalent US$ 1,828,478

*Out of 5114 m length , considering only 4900 m tunnel length assuming 214 m concrete lined tunnel **Price based on MMHEP BOQ and prevailing market rate. High quality steel = NRs. 120/kg while BOQ=NRs. 342.3 /kg

Table 6. Cost of Pre-Injection Grouting to Control Leakage Calculated for both Approaches

Description		Total Grout Consumption and Grouting Cost	
		Tokheim & Janbu Approach	Khimti-I Approach
Total Cement Consumption (tones)		3,981	1,695
Total Cost of pre-injection grouting(NRs. 35.00/ kg)	mill NRs.	139.3	59.3
	mill USD	1.74	0.74
	mill NOK	11.76	5.00
Injection grouting average cost per meter tunnel	NRs.	27245	11,597
	USD	341	145
	NOK	2299	1,718
Deduction for 214 m	mill NRs.	5.8	2.5
Total cost of grouting	mill NRs.	133.5	56.8

Table 7. Cost of Concrete Lining for MMHEP Headrace Tunnel.

Description and Quantity		Cost of concrete lining			
		Rate NRs.	Amount NRs.	Amount USD	Amount NOK
Concrete cost for 4900 m length considered	tm	11,431x6.9 = 78,874	386,482,600	4,831,033	32,609,469
Cost per m tunnel	tm	78,874	78,874	986	6,655

*tm = tunnel meter

Table 8. Cost Comparison Between Headrace Tunnel With and Without Final Concrete Lining.

Cost items		Tokheim and Janbu Approach (M. NRs)	Panthi Approach (M. NRs)
A	Cost of injection grouting	133.5	56.8
	Extra cost of excavation and support	146.3	146.3
	Total extra cost for tunnel without final concrete lining	279.8	203.1
B	Cost of concrete	386.5	386.5
C	Differences	106.7	183.4
D	% saving	27.6	47.5

to Tunnelling in Himalayan Rock Mass Conditions' in 2006 from NTNU. He completed his MSc in Hydropower Development in 1998 and MSc in Civil Engineering in 1992. He is the author of many scientific papers related to tunnelling, rock slope engineering and hydropower. He has over 15 years of experience in design, construction and planning of tunnelling and hydropower projects in the Himalaya (Nepal and India).

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CALENDAR IRRIGATION

1-5 March 2010: Workshop on: Capacity Development for Farm Management Strategies to Improve Crop-Water Productivity using AquaCrop, Location: Bloemfontein, South Africa More Info: http://www.fao.org/nr/water/events.html

25-26 March, 2010: Fifth International Seminar on "Dynamics of Farmer Managed Irrigation Systems: Socio-institutional, economic and technical Context". Location: Kathmandu, Nepal. Contact: subedee.s@gmail.com; pradhanp@mos.com.np

7-9 June 2010: Sustainable Irrigation 2010: Third International Conference on Sustainable Irrigation Management, Technologies and Policies. Location: Bucharest, Romania. More Info: www.wessex.ac.uk/10-conferences/sustainable-irrigation-2010.html

10-16 October 2010, 6th Asian Regional Conference, Theme: Improvement of irrigation and Drainage efficiency under the small land holding condition. Location: Yogyakarta, Indonesia, Contact : Fax : +62 21 726-1956, E-mail: secretariat@icid2010.org, Website : www.icid2010.org

CALENDAR ENERGY

20-21 January 2010: Solar Power Generation USA. Location: Las Vegas, USA. More Info: www.solarpowercongress.com/

27-28 January 2010: Virtual Energy Forum January 2010. Online. More Info: www.virtualenergyforum.com/

31 January -2 February 2010: India Energy Congress 2010, Location: New Delhi, India, Contact: info@indiaworldenergy.com

3-5 February 2010: Renewable Energy World Conference & Expo North America: Power Shift Location: Austin. More Info: http://www.renewableenergyworld-events.com/index.html

17-18 February 2010: ENERGY NOW EXPO, 2010. Location: United Kingdom. More Info: www.energynowexpo.co.uk/

23-25 February 2010: Renewable Energy World North America Conference & Exposition. Location: Austin, Texas, United States. More Info: http://www.renewableenergyworld-events.com/

24-26 February 2010: ENERGY INDABA 2010. Location: Johannesburg, South Africa. More Info: http://energyafricaexpo.com/

15-17 March 2010: World Biofuels Markets. Location: Amsterdam, Netherlands. More Info: www.worldbiofuelsmarkets.com/

18-21 March 2010 : New Energy. Location: Husum, Germany. More Info: www.new-energy.de/

21-23 April 2010: International Conference on Applied Energy (ICAE). Location: Singapore, Singapore. More Info: www.icae2010.org/

27 - 29 April 2010: International Small Wind Conference 2010, Location: Glasgow Science Centre, Glasgow, UK. More Info: www.ruralelec.org/238.0.html

8-10 June, 2010: Renewable Energy World Europe. Location: Amsterdam, The Netherlands More Info: www.renewableenergyworld-europe.com