Journal of Nepal Geological Society, 2000, Vol. 22, pp. 227–236

Rock Support Design for the Khimti I Hydropower Project

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ABSTRACT

The Khimti I Hydropower Project (KHP) is the first project where the Norwegian Method of Tunnelling (NMT) was applied in Nepal. The method was found to be appropriate for drill-and-blast tunnels in jointed, fractured, and sheared rocks, which tend to overbreak. A combination of rock bolting with wet fibre reinforced shotcrete is the main rock support method in the NMT. In exceptionally poor rock (with squeezing conditions), reinforced ribs of shotcrete with a concrete invert were generally found to be a more efficient and cost effective. Cast concrete lining was also used occasionally at the KHP, mainly in overbreak areas with high groundwater discharge.
The main advantage of the NMT is that each stretch of tunnel is evaluated using the Q-System of rock classification and then only the required amount of support is applied. This procedure takes optimum advantage of the self-supporting capacity of the rock.

There were some problems encountered during application of the NMT at the KHP, but the adoption of 10 site-specific design principles made the method more practical and effective. It was generally experienced that the NMT was simple to use and gave appropriate guidelines that could be applied across a very broad range of rock conditions. At Khimti, the peak excavation rates reached 63 m/week and 72 m/week in 25 m^2 and 14 m^2 tunnels, respectively. The average excavation rates (including full support) were 10 m and 35 m per week in "Extremely Poor" and "Poor to Fair" rock conditions, respectively.

INTRODUCTION

The Khimti Hydropower Project (KHP) site is located in the Dolakha and Ramechhap Districts, approximately 175 km due east of Kathmandu (Fig. 1). It is a 'run of the river' type of hydroelectric power project designed for an installed generating capacity of 60 MW. The power plant utilises a drop from 1272 to 586 m above mean sea level in the Khimti Khola with the highest head of 686 m in Nepal. The total tunnel length is 12.8 km with diameter in the range of 3 to 10 m (including the adits). A concrete diversion weir diverts up to 10.75 cumecs of water from the river into a 7.9 km long headrace tunnel, and then through a 913 m long, steel-lined penstock inclined at 45° to an underground powerhouse (70 m long, 11 m wide, and 10 m high). The powerhouse is 420 m under the ground surface and 893 m inside. It contains five horizontal Pelton turbines.

GEOLOGY OF THE PROJECT AREA

The project area lies in the Midland Schuppen Zones of the Melung Augen Gneiss (Fig. 2). The rocks in this zone are represented mainly by grey, coarse- to very coarse-grained, porphyroblastic augen gneiss (63%), occasionally banded gneiss (12%), and granitic gneiss (7%) with bands of very weak, green chlorite and bright grey talcose schist (18%) parallel to the foliation at intervals of 5 to 15 m. Structurally, the zone is bounded by two major faults: the Midland Thrust and the Jiri Thrust to the south and north, respectively. The area is also influenced by several minor thrust faults characterised by very weak sheared schist with clay gouge (Fig. 3) running parallel to the foliation with 3 sets of clay-filled joint. The foliation at the tailrace to Adit 4 (the saddle of Pipal Danda) has steep dips (45° to 60°), whereas it is gently dipping (15° to 35°) between Pipal Danda and the headworks. The dip direction of foliation varies from N50°E to S80°E from the headworks to the tailrace tunnel.

BASIS FOR ROCK SUPPORT DESIGN

At Khimti, rock masses were divided into five main classes in order to ease and speed up the decision for correct tunnel support. The rock mass distribution in the entire tunnel is presented in Fig. 4. According to the distribution of rock mass, 'exceptionally poor rock' is 7–8%, 'extremely poor rock' is 21–22%, 'very poor rock' is 43–44%, and 'fair to poor rock' is 27–28%. It shows that the weak rock ($Q<1$) is about 72%.

A procedure for excavation and application of required rock support depending upon rock class was also worked out (Table 1). There were also additional special support recommendations (Table 2) for sub-horizontal alternating bands with significantly different rock quality (e.g., competent gneiss and incompetent schist), since almost 90% of the rock along the headrace tunnel dips sub-horizontally (<20°).

The methods were applied in 3 stages. Firstly, a tunnel log was prepared and the six parameters for quantifying rock mass quality were collected after each round of blast before

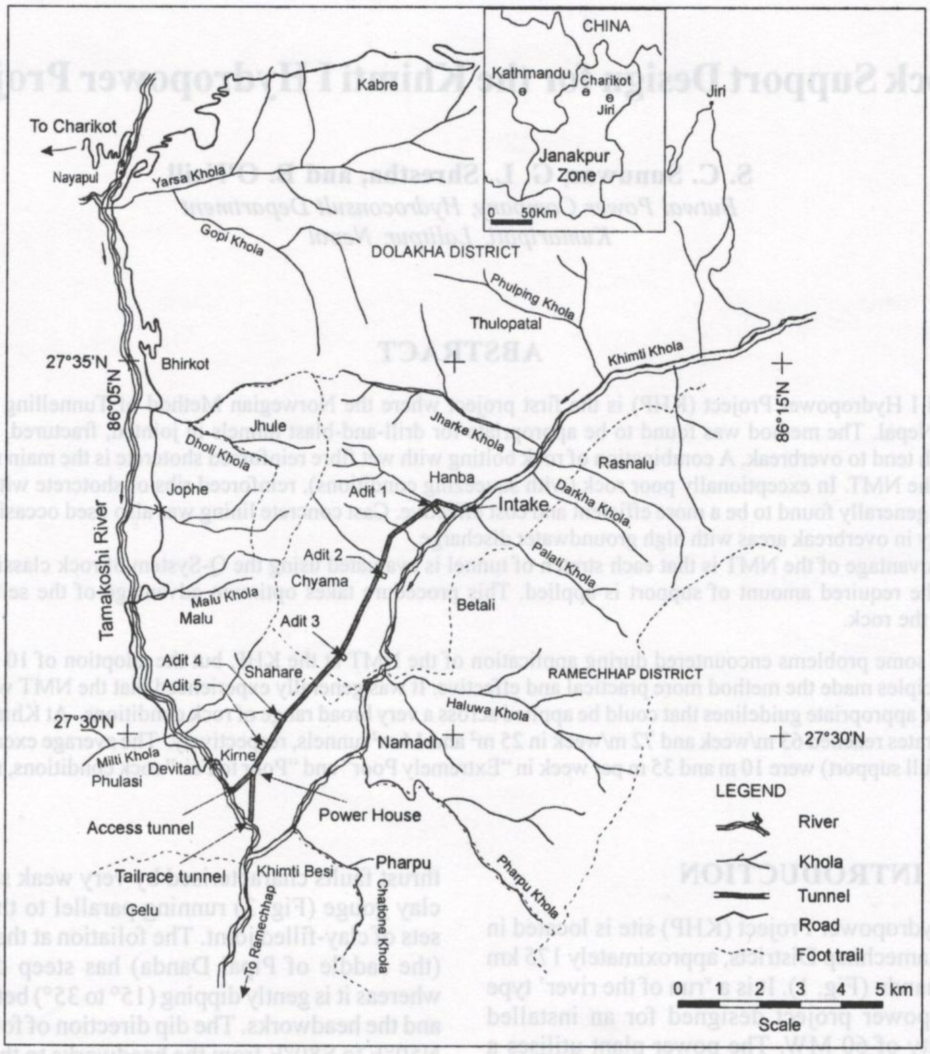


Fig. 1: Location and layout map of the Khimti Hydropower Project

installation of temporary support. Then, a required type of rock support was designed using the Norwegian Method of Tunnelling (NMT) and the recommended support was installed immediately or afterward depending on the stand-up time of rock. Finally, the applied support was monitored in the critical areas where deformations were noticed. Additional support, such as rock bolts, shotcrete, reinforced ribs of shotcrete, and concrete lining, was recommended according to the monitoring data and observations wherever necessary.

As short description of the procedure of underground excavation and support design at the Khimti Project is given below.

The NMT

The NMT (Barton, N. and Grimstad, E., 1994) is considered to be the most appropriate method for the drill-and-blast tunnels in jointed, fractured, and sheared rocks that tend to overbreak. Consequently, the NMT was the most suitable for the main basis of rock support design at the Khimti Project.

The NMT (also called 'design-as-you-drive') utilises a quantitative rock mass classification according to Q-system, an appropriate use of temporary reinforcement (such as bolting and wet fibre reinforced shotcrete) as well as a supplementary reinforcement and support. The main support is a combination of rock bolts in pattern and fibre-reinforced shotcrete. Shotcrete and rock bolts offer great flexibility with respect to the amount of support provided. The shotcrete thickness, rock bolt spacing, and the spacing and thickness of reinforced ribs of shotcrete on poor ground can vary with the greatest ease to suit rock conditions. The reinforced ribs of shotcrete are recommended mainly for 'Extremely poor' to 'Exceptionally poor' rock (with squeezing), but concrete lining is also recommended in a very small amount. The method is quick, has rapid advance rate, improves safety, and often the most economical tunnel support system. The tunnel span and the purpose of excavation were also considered in the selection of final support.

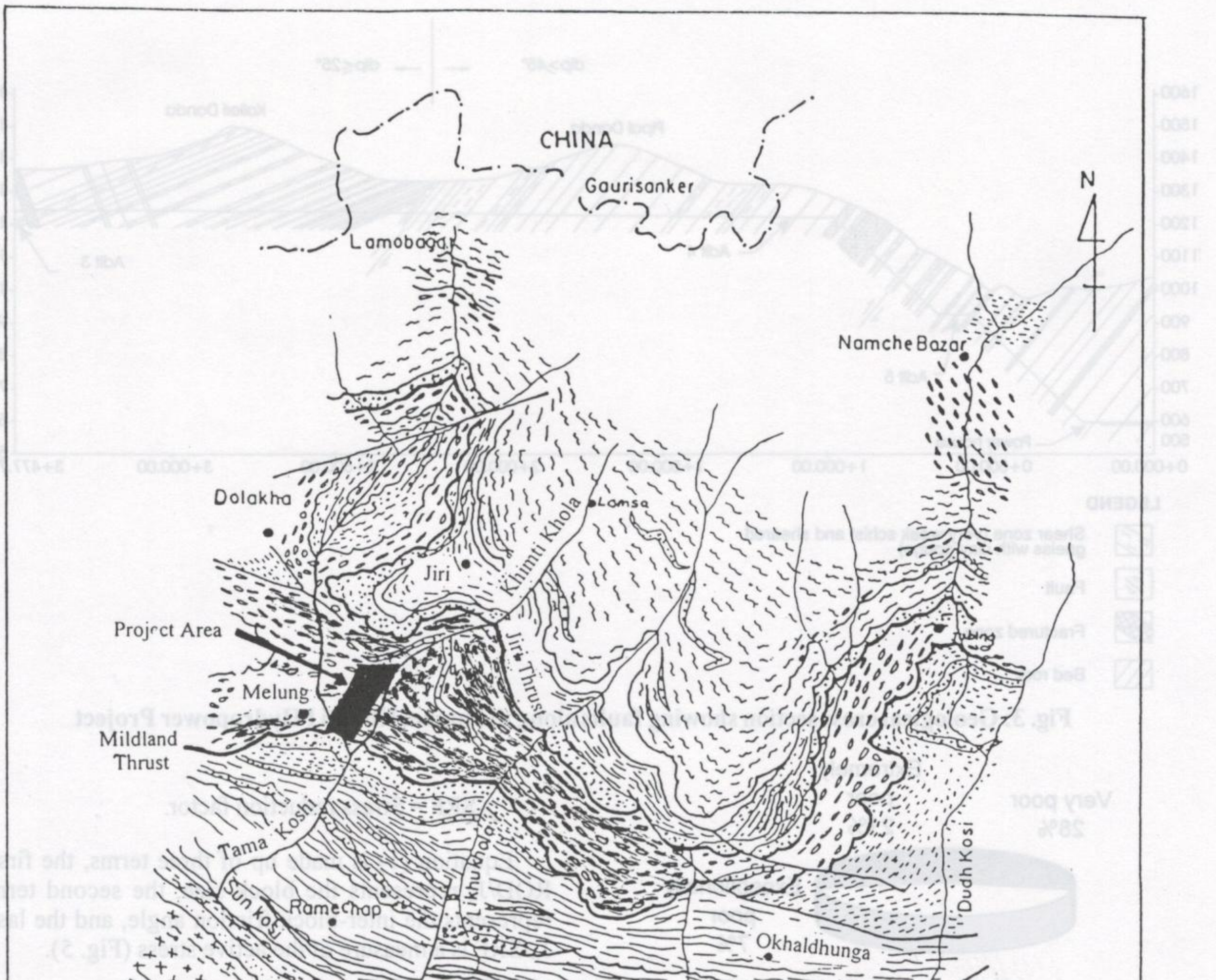


Fig. 2: Geological map and profile of the Ramechhap-Okhaldhunga Region (After Ishida and Ohta 1973)

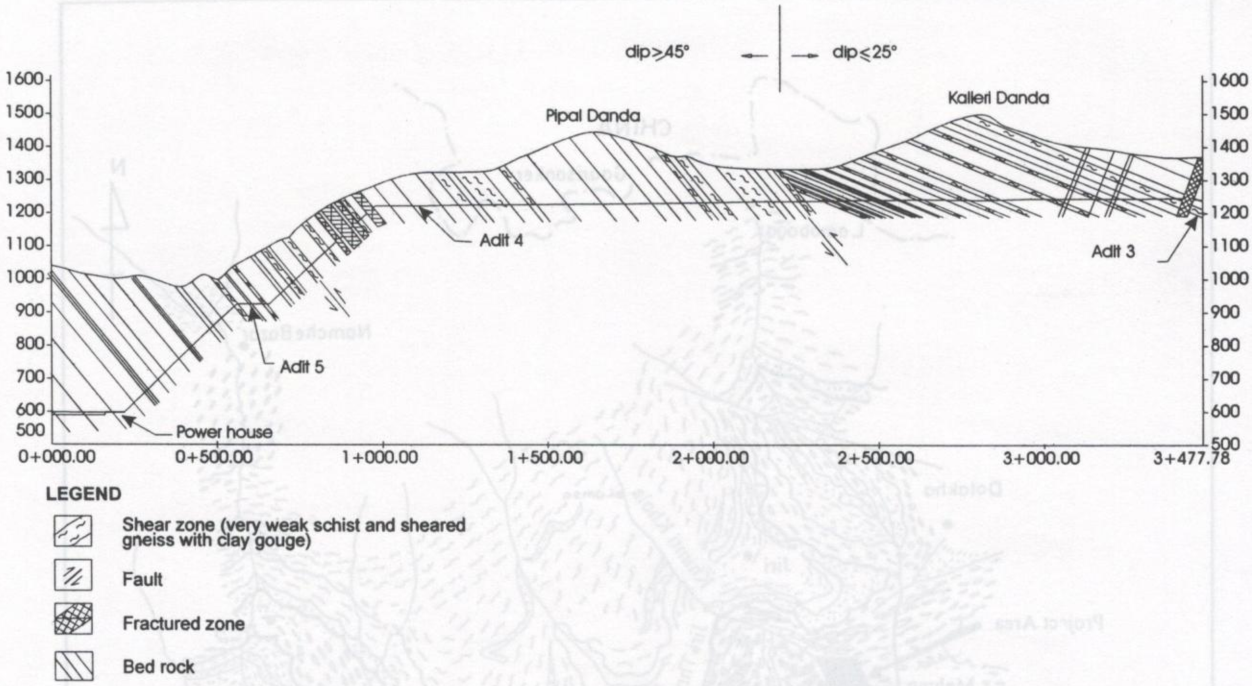


Fig. 3: Geological cross section showing faults along the tunnel Khimti I Hydropower Project

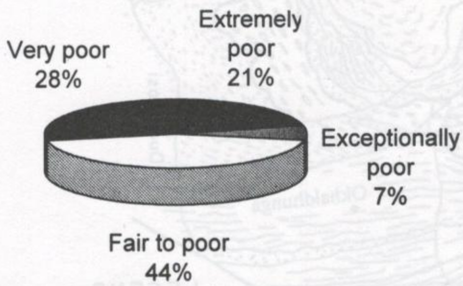


Fig. 4: Rockclass distribution based on Q-system for total tunnel length

The Q-system was developed by Barton et al. (1974) in Norway and updated by Grimstad and Barton (1993) based on more than 1000 new case records from main road tunnels. It includes sufficient information to provide a realistic assessment of the factors that influence the stability of an underground excavation, and is today being used more and more frequently as a quantitative measure of tunnelling conditions and support needs in an increasing number of countries around the world. The Q-System uses simple equations to numerically account for the most important size parameters affecting rock mass stability. It uses the following six different parameters.

$$Q = RQD/J_n \times J_r/J_a \times J_w/SRF \quad (1)$$

Where,

RQD = Rock Quality Designation,

J_n = Number of joint sets,

J_r = Joint roughness,

J_a = Joint alteration number,

J_w = Joint water leakage or pressure, and

SRF = Stress reduction factor.

Equation (1) is made up of three terms, the first term: RQD/J_n represents the block size, the second term: J_r/J_a represents the inter-block friction angle, and the last term: J_w/SRF is a measure of the active stress (Fig. 5).

Adopted rock support

None of the classification systems was suitable to design the correct support for excavations in different types of rock mass. Therefore, additional 10 design principles as compliments to the Q-system were introduced (Stille et al. 1998). The design principles were connected with the following four important hazard conditions in the Khimti area:

- Erosion and slaking,
- Squeezing ground,
- Ravelling ground, and
- Swelling ground.

The 10 design principles for the four hazard conditions are summarised below.

Erosion and Slaking

1. Shotcrete to be extended down to the floor in the rock class No. III to V.
2. If slaking is expected, like in very weak schist bands, extend the shotcrete down to the floor even in the rock class No. II or I.
3. Where slaking is expected in the floor in the rock class No. II to IV, provide erosion protection with a non-erosive layer (40 cm thick gravel invert).

Table 1: Recommended rock support in headrace tunnel (width of tunnel=4.0 m, ESR=1.6)

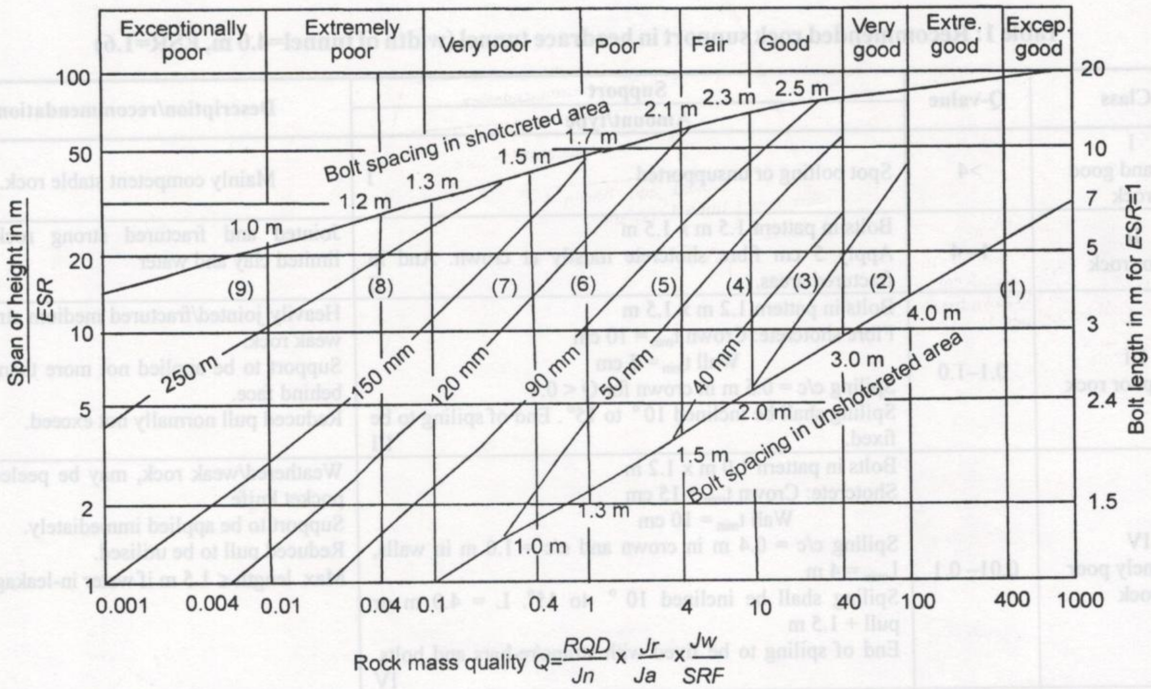
Rock Class	Q-value	Support Amount/type	Description/recommendation
I Fair and good rock	>4	Spot bolting or unsupported I	Mainly competent stable rock.
II Poor rock	1– 4	Bolts in pattern 1.5 m x 1.5 m Apply 5 cm fibre shotcrete mostly at crown. And in fractured areas. II	Jointed and fractured strong rock with limited clay and water
III Very poor rock	0.1–1.0	Bolts in pattern 1.2 m x 1.5 m Fibre shotcrete: Crown t_{min} = 10 cm Wall t_{min} = 5 cm Spiling c/c = 0.5 m in crown for Q < 0.4 Spiling shall be inclined 10° to 15°. End of spiling to be fixed. III	Heavily jointed/fractured medium strong to weak rock. Support to be applied not more than I pull behind face. Reduced pull normally not exceed.
IV Extremely poor rock	0.01– 0.1	Bolts in pattern 1.0 m x 1.2 m Shotcrete: Crown t_{min} = 15 cm Wall t_{min} = 10 cm Spiling c/c = 0.4 m in crown and c/c = 1.0 m in walls, L_{min} = 4 m Spiling shall be inclined 10° to 15°. L = 4.0 m or pull + 1.5 m End of spiling to be fixed with straps/re-bars and bolts. IV	Weathered/weak rock, may be peeled with pocket knife. Support to be applied immediately. Reduced pull to be utilised. Max. length ≤ 1.5 m if water in-leakage.
V Exceptionally poor rock	< 0.01	Bolts in pattern 1.0 m x 1.0 m i) Ribs (6 nos of T 16 bars in 10 cm spacing. Spacing between each set is 1 m) or cast concrete lining. ii) Concrete Slabs or concrete lining at invert. Spiling c/c = 0.3 m crown and c/c = 0.7 m in wall, L_{min} = 3 m or pull + 1.5 m. If necessary two layers. Spiling shall be inclined 10° to 15° for first layer, 30° to 45° for second layer. End of spiling to be fixed, add also shotcrete. According to scope of work if elaborated for actual face. V	Very weak rock normally containing > 60% clay, easily separated by fingers. Schist with water. Support to be applied immediately. Quick setting shotcrete at crown before mucking. Reduced pull to be utilised. Maximum length of pull ≤ 1 m.

Notes: 1. Max. pull length based on use of Boomer. For hand held drilling use Class IV ≤ 1.5 m and Class V ≤ 1.0 m
2. For class II, III, IV and V thick clay layer covered with Shotcrete shall have wire mesh reinforcement.
3. For class IV and V steel, reinforced Shotcrete ribs or cast concrete may be used if potential squeeze, especially for schist.
4. Water inflow requires drainage. Preferably from already supported position.
5. All rock bolts are 20/25 mm dia, 2.3 m long. In special case this may be reduced for hand held equipment.
6. If the recommended amount of rock support are not adequate for long time stability according to observations, additional support will be provided.

Table 2: Revised rock support classes for sub-horizontal layers of significantly different rock quality (worked out according to recommendation of Rock Committee)

Revised class	Description	General support class	Modification of the support
A	Fair to poor rock in crown (class I or II), extremely poor to exceptionally poor in lower part (class IV or V)	Class I or II	10 cm shotcrete sfr and pattern bolting 1.5 m x 1.5 m *in the walls.*
B	Extremely to exceptionally poor rock in crown (class IV or V) and good to poor rock in lower part (class I or II)	Class IV or V	10 cm shotcrete sfr *in the walls.*
C	Good to poor rock in crown and one of the walls (class or II), very poor to extremely poor in the other wall (class III or IV)	Class I or II	7.5 cm shotcrete sfr and pattern bolting 1.0 m x 1.5 m *in the wall with rock class III or IV*

Note: The table to be read in conjunction with the recommended rock support in headrace tunnel agreed on 2nd March 98 by KSC and CCC.



REINFORCEMENT CATEGORIES

1) Unsupported
2) Spot bolting
3) Systematic bolting
4) Systematic bolting with 40–100 mm unreinforced shotcrete
5) Fibre reinforced shotcrete, 50–90 mm, and bolting
6) Fibre reinforced shotcrete, 90–120 mm, and bolting
7) Fibre reinforced shotcrete, 120–150 mm, and bolting
8) Fibre reinforced shotcrete, >150 mm, with reinforced ribs of shotcrete and bolting
9) Cast concrete lining

Fig. 5 : Estimated support categories based on the tunnelling quality index Q (After Grimstad and Barton 1993)

4. All surfaces with the possibility of slaking to be covered with 5 cm shotcrete also in the rock class Nos. I and II.

Squeezing ground

5. Concrete invert for the rock class No. V.
6. Concrete invert for the rock class No. IV if deformation measurements do not clearly indicate stable conditions.
7. If the shotcrete in squeezing areas is highly cracked and deformed, scaling and replacing with steel fibre reinforced shotcrete. If just minor cracks exist, add a layer (30 mm) of shotcrete.

Ravelling ground

8. Make inspections in areas with rock class Nos. I, II, and III to check if special conditions can cause rock fall leading to full collapse, and provide necessary measures (support) to deal with this.
9. Where the quality of material (bolting, shotcrete, concrete lining) and work do not meet the specifications, mitigation measures must be taken.

Swelling ground

10. If swelling ground is identified, special support measures have to be designed.

Monitoring and observation

Applied support in the tunnels is monitored by tape extensometer in critical areas where deformation is noticed, and continuous observations are made in changing conditions of applied support. Deformations are noticed mainly in the rock class Nos. III, IV, and V containing very weak schist (unconfined compressive strength between 1 to 15 MPa) bands greater than 20 cm thick. Additional final support is decided according to the monitoring data and field observations.

ROCK SQUEEZING

According to Terzaghi (1946), squeezing rock slowly advances into the tunnel without perceptible volume increase in weak rock containing a high percentage of micaceous or clay minerals with a low swelling capacity. In other words, when the strength of the rock mass is low compared to the rock stresses, a progressive, semi-plastic failure can develop in the rock mass around the cavity, resulting in an inward movement, and giving an active load on rock support structures installed in the tunnel to counteract the deformations.

Squeezing is normally considered to be the result of overloading of the rock mass. When the tangential elastic stress around an opening is of the same magnitude as the in situ strength of the weak rock mass or the stress is greater than the strength of the weak rock mass, squeezing may occur. Considering a simple example of applying load on table-like structure made of 2 different layers of soft and hard/brittle material. At failure, the soft layers will buckle and the hard ones will collapse. These are the examples of the two different phenomena of rock squeezing and rock bursting, respectively.

Squeezing always occurs in weak rock due to overloading of stress. The term 'weak rock' is considered below 'very poor rock' (Q value is less than 1, as defined by Bhasin and Grimstad 1996), but in practice it is difficult to define the weak rock, as the rock mass is inhomogeneous in nature due to faulting, shearing, jointing, weathering, altering, alternating competent and incompetent rocks etc.

Conditions for rock squeezing

It is not easy to accurately predict the squeezing behaviour of a rock mass. There are several theories developed to predict the squeezing behaviour. It is a long-term stability problem, which concentrates on the time-dependent behaviour of rock mass. If rock strength is less than field stress or overburden load in any direction, the rock squeezing will occur. Singh (1993) proposed that squeezing occurs when the Q-value and the stress level satisfy the relation:

$$H > 350 \times Q^{1/3} \qquad (2)$$

where

H = vertical rock cover (m) and
Q = Rock mass index according to the Q-system.

Bhasin and Grimstad (1996) presented the conditions for squeezing in weak rock (Q value is less than 1) by calculating the ratio of tangential stress (σ_q) and compressive strength (σ_{cm}). According to it, the following three conditions are established:

- No squeezing if the ratio is less than 1,
- Mild and moderate squeezing if the ratio is 1 to 5, and
- Heavy squeezing if the ratio is greater than 5.

Method of rock support in squeezing ground

The philosophy of rock support method in squeezing ground at Khimti was the application of flexible support (shotcreting and rock bolting) with invert lining at first for providing room for deformation. The affected area was monitored in weekly or day-to-day basis depending on the squeezing rate. An additional support was recommended after deformation monitoring.

At Khimti, support was provided according to Tables 1 and 2, and the applied support was monitored in the rock class Nos. III, IV, and V (containing very weak schist), at first by visual inspection and then by tape extensometer. Especially, in the area with rock class No. V containing very weak schist, steel fibre was increased from 50 kg to 70 kg per cubic metre to make shotcrete more flexible to bear load. Similarly, concrete invert overlapping with shotcrete was provided in very weak schist area. According to monitored data after invert concreting, squeezing rates gradually/abruptly decreased. At some places, the shotcrete was badly damaged and there were up to 15 cm wide cracks. Grouted rock bolts worked efficiently in the squeezing area where the effect of squeezing was seen in twisting of end plates. Additional reinforced ribs of shotcrete with invert concrete were applied in critical areas of squeezing, whereas additional bolts with shotcrete were applied in minor squeezing areas.

Monitoring of squeezing ground

The critical sections of the tunnel were monitored by tape extensometer and observations for the applied rock supports were also recorded (Fig. 6, Adit 1 D/S monitoring data/graph). Generally, deformations were noticed after 2 to 3 weeks of excavation. The deformed sections were monitored immediately after the initiation of deformation. Based on the monitoring results, additional support such as reinforced ribs of shotcrete or rock bolts were added in areas with considerable deformation. At Khimti, the maximum deformation recorded was 40 cm (deformation as large as 5.5% of the tunnel diameter) in Adit 1 (between Chainage 500 m and 600 m, downstream of the headrace tunnel).

The effectiveness of rock bolting strongly depends on the bearing capacity of the rock mass under the end plate and on the ability of the nut plate blocking system to undergo high plastic deformation without failure. Due to mild squeezing at Khimti, only a few bolts (about 1% of rock bolt end plates or nuts) failed.

It was found that squeezing took place also in weak schist and decomposed gneiss (even when Q value was greater than 1). For example, squeezing was noticed only in a single band (10–20cm thick) of very weak schist intercalated with strong gneiss (Q value was from 1 to 6). Therefore, this example clearly indicates that the strength of rock is more important than the term 'weak rock' defined by the classification system.

Long-term mild rock squeezing problems were observed at the Khimti project. The squeezing problem was noticed by cracking of applied shotcrete and twisting of end plates of rock bolt 15m to 20m behind face or after 2 weeks.

At Khimti, mild squeezing was noticed at various locations (Table 3) after 2 weeks, mainly in the weak schist and decomposed gneiss with the value of uniaxial compressive strength ranging from 1 to 15 MPa and the depth of overburden between 80 and 420 m. The ratio of σ_θ and σ_{cm} at various squeezing locations is shown in Fig. 7. According to the graph, 40 locations satisfied the conditions and 2 were below squeezing limit but squeezing was noticed.

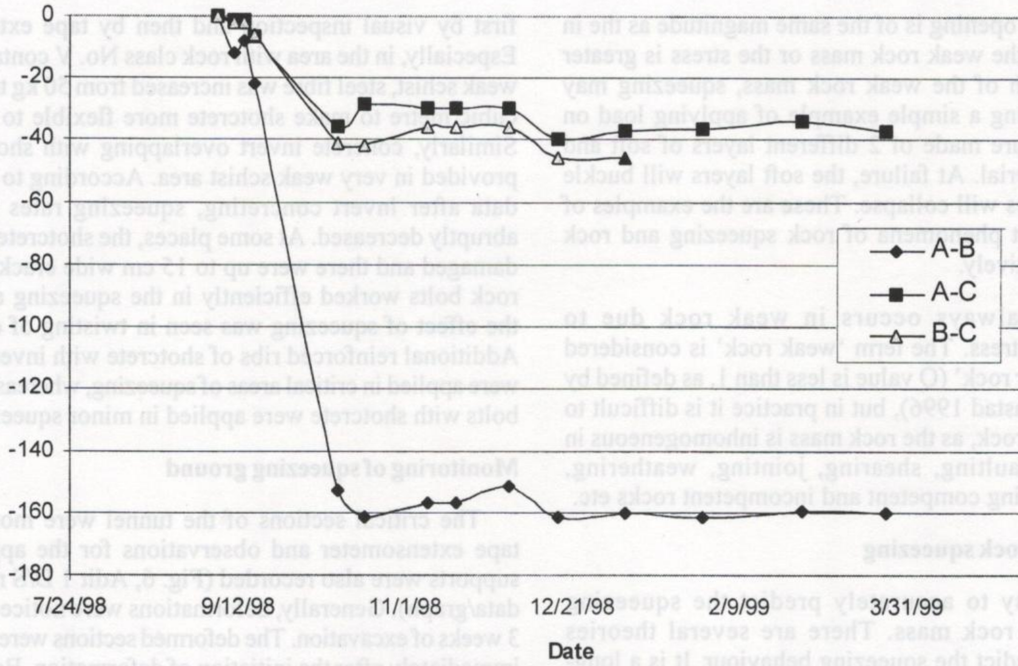


Fig. 6: Adit 1-downstream monitoring data/graph

Table 3: Condition of Mild squeezing in different tunnels of Khimti Project (60 MW)

Location	Chainage (m)/Q value	Overburden (Z) Horizontal stress (σ_h)* Vertical stress (σ_v)	Rock type/ Strength (MPa)	Deformation (mm)	Tangential stress (σ_θ)/Compressive strength (σ_{cm}) (MPa)	Remarks
Adit 1 headrace downstream (span = 3.6 cm)	470–620/Q = 0.003– 0.006 = 0.004	Z = 100–130 m σ_h = 3.9–5 MPa σ_v = 2.6–3.4 MPa	Sheared schist/ 0.5–2 MPa	50–170	5–6.5/1.7–3 = 2.9–2.1	Alignment is parallel to foliation (N25°E/<25°NW). Badly cracked shotcrete (10 cm wide cracks) and twisted end plates of bolts.
	450–457/Q = 0.01	Z = 112 m σ_h = 4.35 MPa σ_v = 2.9 MPa	Sheared schist/ 0.5–1 MPa		5.6/3.7 = 1.5	Alignment is parallel to foliation (N10°E/<35°NW).
Adit 2 headrace upstream (span = 4 m)	1280–1295/Q = 0.03	Z = 212 m σ_h = 8.25 MPa σ_v = 5.5 MPa	Schist at walls/ 1–5 MPa	6	10.6/5.4 = 1.96	Alignment is parallel to foliation (N10°E/<25°NW).
	1315–1380/Q = 0.01	Z = 250 m σ_h = 9.75 MPa σ_v = 6.5 MPa	Schist/1–2 MPa	60	12.5/3.7 = 3.4	Alignment is parallel to foliation.
	1620–1630/Q = 0.01	Z = 112 m σ_h = 4.35 MPa σ_v = 2.9 MPa	Schist at crown/ 1–5 MPa		5.6/3.6 = 1.55	Alignment is parallel to foliation.
	1725–1740/Q = 0.04–0.08	Z = 90 m σ_h = 3.51 MPa σ_v = 2.34 MPa	30 cm thick clay/ 0.25–1 MPa	12.4	6.8/4.5 = 1.5	Alignment is parallel to foliation (N55°E/<35°NW).
Adit 2 headrace downstream (span = 4 m)	126–140/Q = 0.01–0.02	Z = 80 m σ_v = 3 MPa σ_v = 2 MPa	Schist/1–5 MPa		4/3.7 = 1.08	Alignment is parallel to foliation (N50–55°E/<25° NW). Squeezing noticed by twisting of end plate and badly cracked shotcrete at crown.
	434–440/Q = 0.2–0.4	Z = 125 m σ_h = 5.85 MPa σ_v = 3.9 MPa	Multiple schist bands at left wall/1–5 MPa	4	6.25/11.7 = 0.5	Alignment is 40° oblique to foliation (N10°E/<40°NW). Shotcrete cracked at left wall.

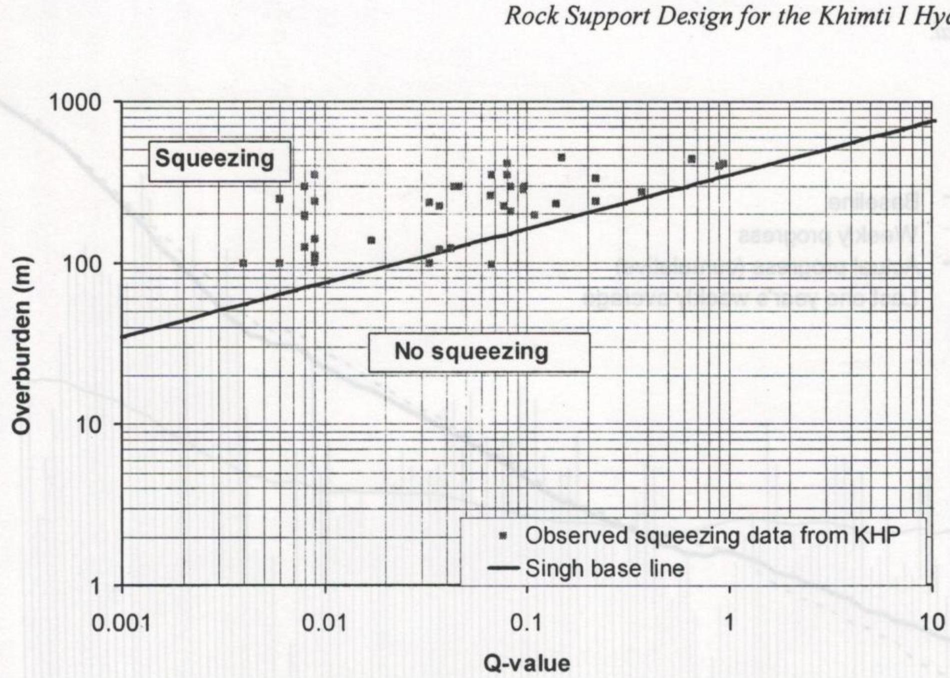


Fig. 7: Singh's Criterion for rock squeezing

DIFFICULTIES FACED

It is not easy to define rock mass quantitatively due to its non-homogeneity by faulting, jointing, folding, weathering, alteration, stress distribution, textural variation etc. In the Khimti area, the rock mass classification and design faced with the following difficulties:

- Q-system does not take into consideration the orientation of the tunnel relative to the orientation of the main discontinuities even though this may have a major influence on stability.
- There is a general problem of defining the joint set that dictates the stability. According to Barton, the most unfavourable joint set, which has an influence on the stability of the tunnel, should be critical. Selection of 'the most unfavourable joint set' may differ considerably from geologist to geologist. The range of possible values for J_a and J_r (based on the most unfavourable joint set) is comparatively large.
- The value for the joint water reduction factor, J_w, (especially for high flow conditions) is difficult to estimate as the rating may be based on inflow and pressure. The inflow resulting from low pressure and high conductivity may be similar to the one resulting from high pressure and low conductivity. The two cases may result in the similar rating even if they are connected with completely different rock conditions.
- SRF is the most difficult factor to evaluate. For the rock considered 'competent', the SRF should be selected with reference to the prevailing stresses in relation to the uniaxial compressive strength or tensile strength. Since it is difficult to measure stresses, the rating for this factor has to be based on a rough estimation. If the rock mass contains zones of weakness of any kind, then the number of zones and the depth of excavation defines the value. However, the thickness of the zone has no influence on the SRF value. Moreover, if a weak section does not intersect the tunnel alignment but influences the stability, the evaluation of SRF will be difficult.
- Though the Q-system support requirements cover a very wide range of rock quality, it was developed primarily based on the Norwegian experience in relatively good rock.
- Because of the effects of surface weathering, it is difficult to estimate the Q-value of the underlying rock during surface mapping. Similarly, some of the weak zones are not detected during surface mapping, but are encountered during excavation.

At the beginning, the progress rate of the project was slow because of poor geological conditions than the expected from the feasibility report, inefficient equipment, and semi-mechanised method of excavation. Consequently, after 1.5 years, the system was completely changed into mechanised one using boomer for drilling, robot machine for shotcreting, and scooptram, wheel loader, and dump truck for mucking. The mechanisation speeded up the progress so as to complete the excavation work on time. The progress rate is shown in Fig. 8. At Khimti, the recorded progress rates were 63 m/week, 46 m/week, and 73 m/week (in both headings) in 25 m², 14 m² and 14 m² cross-sectional area of tunnels, respectively.

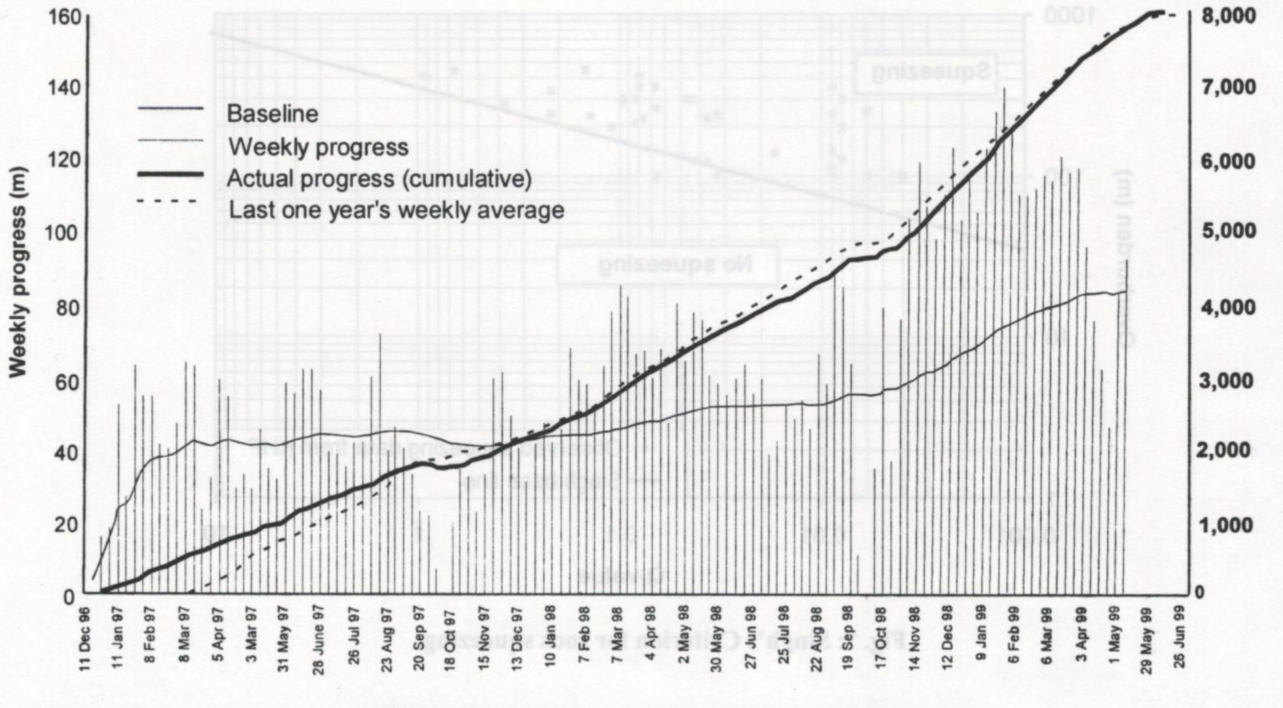


Fig. 8: Headrace tunnel construction progress rate

CONCLUSIONS

The NMT was applied for the first time in Nepal at the Khimti I Hydropower Project, and was found to be the most appropriate for drill-and-blast tunnels in jointed, fractured, and sheared rocks, which tend to overbreak. It provided appropriate rock support as required by rock mass quality and is found efficient and effective, because after completion of construction only additional cosmetic rock support is needed.

The NMT needed adjustments with the recommendations based on the experiences gained in some of the poor-quality rocks of Nepal. Therefore, additional 10 design principles were developed for the Khimti I Hydropower Project. Those were found to be very useful and cost-effective.

Mild squeezing was noticed in very weak schist and decomposed gneiss having unconfined compressive strength of 1 – 15 MPa and the depth of overburden between 80 and 450 m. Reinforced ribs of shotcrete with invert concrete worked effectively in the mild rock-squeezing area. Mild squeezing was observed even when the ratio of tangential stress and compressive strength was slightly less than 1.

ACKNOWLEDGEMENT

We are grateful to the Butwal Power Company for granting permission to publish this paper.

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