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Rock support in hydropower projects of Nepal: case studies

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ABSTRACT

The principal objective of rock support is to assist the rock mass to support itself. One common example is where the rock support system (e.g. rock bolts and shotcrete) actually becomes integrated with the rock mass. Rock support strengthens the rock mass surrounding an excavation by creating a reinforced zone, which maintains the integrity of the excavated surface, possesses sufficient flexibility to allow for the redistribution of stresses around the excavation, and has enough stiffness to minimise the dilation (opening) of discontinuities. Rock mass classification systems are used worldwide as a basis for tunnel support design. The Q and Rock Mass Rating systems have been extensively applied in rock support design on most of the hydropower projects in Nepal. Generic design guidelines based on rock mass classification systems cannot provide suitable rock support for every site. Therefore some modifications are necessary to suite the site-specific ground conditions including local rock mass and geological hazards.

There are relatively few tunnels excavated in the tectonically active Nepal Himalaya. Large-diameter tunnels in Nepal are commonly lined with concrete whereas recently smaller-diameter tunnels are either shotcrete-lined or left unsupported. "Leaky" lining has been used in most of the projects to avoid the heavy reinforcement needed to withstand the occasional very high external water pressures.

INTRODUCTION

Tunnel excavation initiates changes in the stress field around an opening. If the stress is high enough and (or) the rock is weak enough, the surrounding rock mass will move slowly into the free space, i.e. inside the tunnel (Fig. 1). This inward radial deformation may continue until a fractured zone of rock collapses into the tunnel. Therefore a support is installed in an underground excavation to keep the rock in place and prevent subsequent failing (Fig. 1). In general, rock masses around an underground excavation support themselves by arching. The rock support strengthens the rock mass surrounding an excavation by preventing the detachment of loose blocks as well as interlocking individual blocks (i.e. by increasing the shear resistance of discontinuities). This process results in a reinforced zone (Fig. 1) within the rock mass, which maintains the integrity of the excavated surface, possesses sufficient flexibility to allow for the stress redistribution around the excavation, and has enough stiffness to minimise the dilation (opening) of discontinuities.

The main objective of rock support is to help the rock mass to support itself. This applies to any rock reinforcement system, e.g. rock bolts and shotcrete are the support systems that actually form part of the rock mass. Rock mass classification systems are commonly used for rock support design around the world and so is the case in Nepal.

TYPES OF FAILURE

Before designing rock supports it is necessary to understand the failure mechanisms. Generally there are two possible failure mechanisms in tunnels and caverns – structurally controlled and stress-induced ones. Both are dependent mainly on the rock mass and its in situ stress conditions.

Structurally controlled failure

This failure is dominated by gravity falls and sliding along inclined discontinuities that are generated by existing joints and weaknesses in the rock under the action of in situ stresses or external forces, particularly by gravity or porewater pressure. Examples are ravelling, loosening, and block falls (Fig. 2).

Stress-induced failure

Stress overloading results into different types of failure in low- and high-strength rocks and the failures can be classified as following.

Rock squeezing

It is a general shear failure without perceptible volume increase in the low-strength rock caused by overloading from the existing stress field. It has time-dependent deformation characteristics and leads to shear failures due to a gradual loss of strength. Rock squeezing (Fig. 3) generally

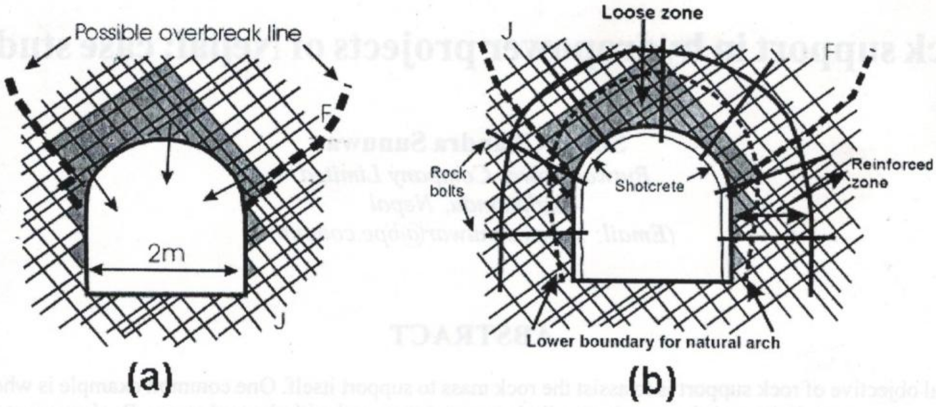

Fig. 1: Sketch of a tunnel section depicting: (a). Possible failure path after excavation due to changing of stress field around opening and; (b). Rock supports (i.e. rock bolts and shotcrete) helping the rock mass to support itself by an arching effect

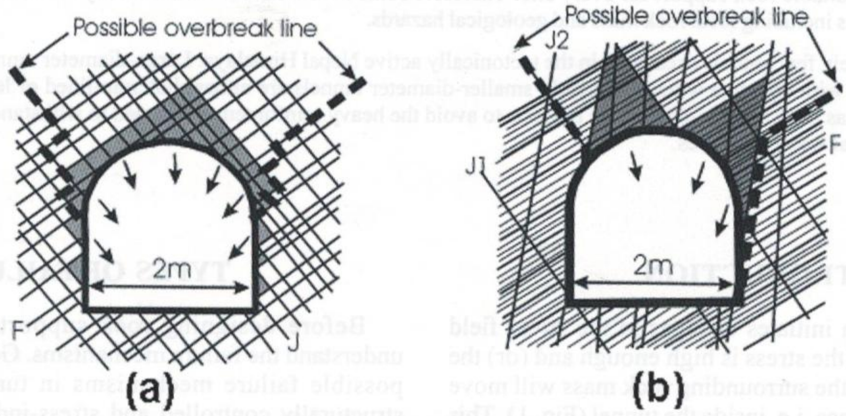

Fig. 2: Typical structurally controlled failures shown by a dotted line and grey shading: (a). Structurally controlled shape of tunnel in widely and closely spaced jointed quartzite; (b). Wedge failures at the crown and wall controlled by joints in slate

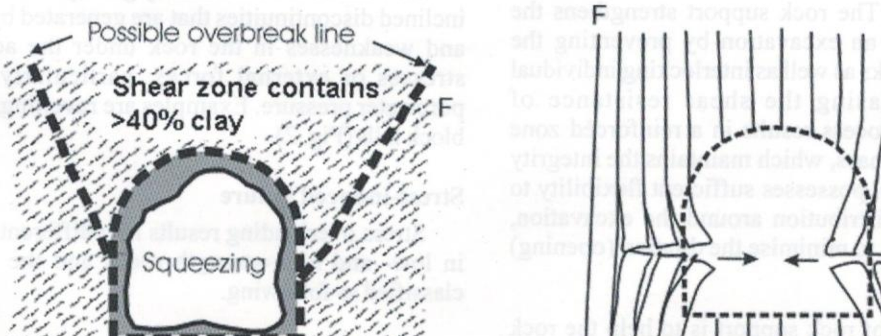

Fig. 3: Stress-induced failures: (a). Squeezing in a shear zone containing greater than 40% of clay; (b). Rock burst resulting into the buckling of walls due to a high horizontal stress in very strong and brittle quartzite. F= foliation

occurs in the low-strength rock containing a high percentage of non-swelling mica and clay minerals at shallow depths as well as in deeper levels.

Rock spalling or burst

This type of instability generally occurs at a great depth and in fjords due to high and anisotropic stresses in the strong, brittle, and massive rock. The intensity of instability may vary from splitting, spalling, and bending to buckling of slabs until an explosive failure or rock burst extends far into the rock (Fig. 3).

In addition, the instabilities caused by the pressure of soil containing swelling clay minerals (viz. montmorillonite, vermiculite, saponite, allophane, and halloysite) are common in shales, mudstones, anhydrites, and marls. The swelling pressure developed at the contact with water generates ground heaving, deformation, and cracking of concrete due to a time-dependent clay volume increase.

In Nepal tunnels are generally found at shallow depths. They exhibit a low in situ stress field with a high level of rock mass disintegration. In them the instability is related mainly to structurally controlled failures rather than to rock squeezing.

CASE STUDIES

In the tectonically active Nepal Himalaya, there are relatively few tunnel excavations. The Kulekhani, Marsyangdi, and Kali Gandaki hydropower projects are the examples that have been constructed with full concrete linings. Similarly the Andhi Khola and Jhimruk hydropower projects are the examples that used stone masonry linings with precast concrete arch. On the other hand, the newly built hydropower projects such as Khimti, Modi, Bhote Koshi, Chilime, and Puwa Khola applied mainly shotcrete and a pattern of rock bolting or the tunnel was left unsupported. The various types of rock support utilised in the hydropower projects of Nepal are summarised in Table 1.

Generally rock mass classification systems are used for rock support design in Nepal. Among them the Rock Mass Rating (RMR) system initially developed by Bieniawski (1989) and the Q system developed by Barton et al. (1974) are widely applied with some modifications to suite local rock conditions. These rock mass classification systems are convenient to apply on a routine basis, provide good guidelines, and speed up tunnel progress. None of the classification systems ever provides the ability to design a correct support for excavation in different rock masses. Therefore for a proper rock support design some modifications are necessary to suite the individual project ground conditions on the basis of local rock mass conditions and possible geological hazards. Some cases of rock support in the hydropower tunnels of Nepal are discussed below.

Khimti I Hydropower Project

The Khimti I Hydropower Project is a run-of-the-river type plant designed for an installed capacity of 60 MW. It has the highest head of 686 m in Nepal. A concrete diversion weir diverts up to 10.75 cumecs of water from the Khimti River into a 7.9 km long headrace tunnel and then through a 913 m long steel-lined penstock to an underground powerhouse (70 m long, 11 m wide, and 10 m high). Himal Power Limited (HPL) is the owner of this project. The project started in 1996 and was completed in June 2000 with a total cost of US$ 140 million.

Geological conditions

The rocks of the project area are represented mainly by very coarse- to coarse-grained, grey porphyroblastic augen gneiss (63%), sporadically banded gneiss (12%), and granitic gneiss (7%) with bands of very weak, green chlorite and bright grey talcose schist (18%) at intervals of 5 to 15 m. The area is influenced by several minor thrust faults running parallel to the foliation and characterised by very weak sheared schist with clay gouge. In addition the rocks of the project area have two major orientations of dip due to the presence of a thrust fault. The rocks between Adit 4 and the tailrace tunnel dip steeply (45°–60°) whereas those in the headrace tunnel between Adit 4 and the headworks dip sub-horizontally (<20°). The tunnel axis runs essentially parallel to the strike of foliation which varies from N50°E to S80°E between the headworks and the tailrace tunnel. Major problems of overbreak and rock squeezing occurred in the sub-horizontally dipping schist. The schist contains a considerable amount of non-swelling clay and it is soft and weak. The thickness of schist varies from a few centimetres to a maximum of 60 m.

Design basis

The Norwegian Method of Tunnelling (NMT) was applied in the rock support design of this project (Sunuwar et al. 2000). This method was found to be appropriate for the drill-and-blast tunnels in jointed, fractured, and sheared rocks. In addition this method is also based on the modern rock support philosophy of taking the optimum advantage of the self-supporting capacity of the rock mass.

In the Khimti I Hydropower Project the rock mass was divided into main five classes (Table 2) in order to ease and speed up the decision for correct tunnel support (Sunuwar et al. 2000). There was also an additional special support recommendation (Table 3) for sub-horizontal layers (comprising almost 80% of the rock in the headrace tunnel) with significantly different rock quality (i.e. alternating competent gneiss and incompetent schist).

The tunnelling method proceeded in three stages. Firstly a tunnel log was prepared and the six parameters for quantifying rock mass quality were collected after each round of blasting before installation of a temporary support. Then

Table 1: Rock supports applied in the tunnels of various hydropower projects of Nepal

S. No	Name of Hydropower Project	Permanent Rock Support Type	Support design basis	Rock type (Rock unit)	Remarks
1	Tinau (1 MW) Tunnel: 1.6 km long and 2 m span	Bricks with cement mortar and pre-cast concrete arc	Experience	Sandstone (Middle Siwalik)	Performing well since 1974
2	Kulekhani I and II (60 and 32 MW), Tunnel: 2.3 km long and 1.8 m span	Concrete lining with rock bolt and shotcrete	Experience	Quartzite/ schist (Lesser Himalaya)	Performing well since 1977 (Kulekhani I) and 1982 (Kulekhani II)
3	Marsyangdi (75 MW) Tunnel: 4 km long and 6 m span	Concrete lining with rock bolt and shotcrete	NATM[1] with experience	Phyllite/ quartzite (Lesser Himalaya)	Performing well since 1986
4	Adhi Khola (5.1 MW) Tunnel: 1.2 km long and 2.2 m span	Stone masonary with pre-cast concrete arc	Experience	Slate/phyllite (Lesser Himalaya)	Performing well since 1989
5	Jhimrik (12 MW), Tunnel: 1.1 km long and 3 m span	Stone masonary with pre-cast concrete arc	Experience	Slate/phyllite (Lesser Himalaya)	Performing well since 1993
6	Khimti I (60 MW) Tunnel: 7.9 km long and 4 m span	Rock bolts with fibre reinforced shotcrete, partly concrete lining	Q-System[2]	Gneiss/schist (Lesser Himalaya)	Performing well since 2000
7	Modi (14 MW), Tunnel: 1.5 km long and 3.15 m span	Rock bolts with mesh shotcrete and concrete lining	Q-system/ RMR[3]	Quartzite/ phyllite (Lesser Himalaya)	Performing well since 2000
8	Puwakhola (6.2 MW) Tunnel: 3.24 km long and 2.5 m span	Pattern of rock bolts with mesh shotcrete and concrete lining	Q-system/ RMR	Gneiss/schist (Lesser Himalaya)	Performing well since 2001
9	Kali Gandaki (144MW) Tunnel: 6 km long and 8 m span	Concrete lining with steel ribs, shotcrete and rock bolt	GSI[4]/RMR	Phyllite/slate /dolomite (Lesser Himalaya)	Performing well since 2002
10	Indrawati (7.5 MW) Tunnel: 3 km long and 2.5 m span	Concrete lining with steel ribs, shotcrete and rock bolt	RMR	Gneiss/schist/ Quartzite (Lesser Himalaya)	Performing well since 2002
11	Chilime (20 MW), Tunnel: 3 km long and 3.5 m span	Rock bolts with mesh shotcrete and concrete lining	Q-system/ RMR	Gneiss/ schist (Lesser Himalaya)	Performing well since 2003

[1]NATM = New Austrian Tunnel Method, [2]Q = Norwegian Geotechnical Institute (NGI) Tunnelling Quality Index; [3]RMR = Rock Mass Rating; [4]GSI = Geological Strength Index

Table 2: Recommended rock supports for the Khimti I headrace tunnel (width = 4 m, Excavation Support Ratio, ESR = 1.6) (Sunuwar et al. 2000)

Rock class	Q-Value/RMR	Support amount/Type	Description/Recommendation
I - Fair to good rock	>4 >50	Spot bolting or unsupported.	Mainly competent stable rock.
II - Poor rock	1 – 4 44 – 50	Bolts in pattern 1.5 x 1.5 m. 5 cm shotcrete at crown and fractured area.	Jointed and fractured strong rock with limited clay and water.
III - Extremely poor rock	0.1–1 23 – 44	Bolts in pattern 1.2 x 1.5 m. Shotcrete: 10 cm at crown and 5 cm at walls.	Heavily jointed or fractured medium-strong to weak rock. Normally reduce or do not exceed pull length.
IV - Extremely poor rock	0.01 – 0.1 3 – 23	Bolts in pattern 1 x 1.2 m. Shotcrete: 15 cm at crown and 10 cm at walls.	Weathered or weak rock, can be peeled with pocket knife. Support to be applied immediately.
V - Exceptionally poor rock	<0.01 <3	Bolts in pattern 1 x 1m. Fibre shotcrete = 20 cm. Ribs (6 nos. of T16 bars in 10 cm spacing. Spacing between each set is 1m) or cast concrete lining. Concrete lining at invert.	Very weak rock, normally containing >60% clay, easily separated by fingers. Support to be applied immediately. Apply quick-setting shotcrete at crown before mucking. Reduce pull length.

Table 3: Additional rock support for sub-horizontal layers of significantly different rock quality in Khimti I Hydropower Project (Sunuwar et al. 2000)

Revised Class	Description	General support classes	Modification to 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 fibre shotcrete and pattern bolting at 1.5 x 1.5 m at walls.
B	Extremely to exceptionally poor rock in crown (Class IV or V) and good to poor rock in the lower part (Class I or II).	Class IV or V	10 cm fibre shotcrete at walls.
C	Good to poor rock in crown and one of the wall (Class I or II), very poor to extremely poor rock n the other wall (Class III or IV).	Class I or II	7.5 cm fibre shotcrete and pattern bolting 1 x 1.5 m at wall with Rock Class III or IV.

Table 4: Rock mass distribution in headrace tunnel of Khimti I Hydropower Project based on the Q system

Rock class	Q-value	Percentage
Fair to poor rock	>1	28
Very poor rock	0.1 – 1	44
Extremely poor rock	0.01 – 0.1	21
Exceptionally poor rock	<0.01	7

the required amount of rock support was specified by the geologists from the consultant and contractor. Lastly the recommended support was installed either immediately or afterwards depending on the stand-up time of the rock.

The applied support was monitored in the critical areas with significant deformations. Additional supports such as rock bolts, shotcrete, reinforced ribs with shotcrete, and concrete lining were recommended according to the monitoring data and observations. The quality of rock mass in the headrace tunnel based on the Q system is presented in Table 4 (Sunuwar et al. 2000).

Adopted rock support design

In order to establish a proper and correct tunnel support design, the Rock Committee introduced additional 10 design principles (Sunuwar et al. 2000). The following four important hazards connected to the geological conditions were the main reasons for adding these design principles.

- Erosion and slaking of the tectonised or weathered rock mass,
- Squeezing ground in the tunnel with the poorest rock quality,
- Ravelling ground mostly connected to the poorer rock quality in the tunnel (unstable rock in general), and
- Swelling ground where swelling clay was present.

The 10 rock support design principles were the following.

a) Hazard type: slaking and erosion

1. Shotcrete to be extended down to the floor in class III to V.
2. If slaking is expected, like in very weak schist bands, the shotcrete to be extended down to the floor even in class II or I.
3. Where slaking is expected in the floor in class II to IV, erosion protection to be provided with a non-erosive layer (40 cm thick gravel invert).
4. All surfaces with the possibility of slaking to be covered with 5 cm thick shotcrete also in the class I and II rock.

b) Hazard type: squeezing or very ravelling ground

5. Concrete invert for class V rock.
6. Concrete invert for class IV rock if deformation measurements do not clearly indicate stable conditions.
7. If the shotcrete in squeezing areas is highly cracked and deformed, scaling and replacing of steel fibre reinforced shotcrete needs to be applied, if just minor cracks exist, an additional layer (30 mm thick) of shotcrete to be applied.

c) Hazard type: ravelling ground

8. Inspection of Classes I, II, and III rock masses is necessary to check if there are special conditions that can cause rock falls which lead to full collapse. Necessary actions (i.e. rock support) must be taken to deal with this problem.
9. Where the quality of material (bolting, shotcrete, concrete lining) and work does not meet the specifications, mitigation measures must be taken.

d) Hazard type: swelling ground

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

Instrumentation

The applied support in the tunnels was monitored by a tape extensometer in the critical areas where deformations were noticed. The deformation was continuously recorded with the changing conditions of the applied support. Deformations were noticed mainly in Classes III, IV, and V (generally in very weak schist bands). An additional final support was decided according to the monitoring data and field observations.

Final lining in this project (Tables 2 and 3) was carried out following the 10 rock support design principles stated earlier. Weep holes were provided in each 1 m^2 of tunnel area, mainly in shotcrete to release external porewater pressure following the design philosophy of "leaky" lining.

It was found that the NMT with some modifications (to suite local ground conditions) worked properly in this project. This method is less time-consuming, has a rapid advance rate, and produces better working conditions.

Modi Khola Hydroelectric Project

The run-of-the-river Modi Khola Hydroelectric Project includes a 7.5 m high and 33 m long diversion weir, 100 m long settling basin, 1500 m long headrace tunnel of 3.15 m diameter, 45 m long horizontal tunnel of 4.24 m diameter, 40 m high surge tank of 9 m diameter, 51 m deep and 4 m in diameter vertical shaft, 422 m long pressure tunnel of 3.2 to 4 m diameter, semi underground powerhouse (27 m x 14 m x 22 m), and 262 m long tailrace canal. Its net head is 67 m and install capacity is 14.7 MW. The Nepal Electricity Authority (NEA) is the owner of this project. The total cost of this project was US$ 30 million.

Geological conditions

The project area consists of quartzite (80%) and schist (20%). The quartzite is fresh to slightly weathered, white to green, fine- to coarse-grained, widely foliated, and strong with alternating bands of weak phyllite (Paudel et al. 1998). Similarly the schist is moderately to slightly weathered, fine- to medium-grained, and weak to moderately strong. Shear zones contain fully decomposed soft fault gouge and shattered fault breccia. The tunnel axis is almost parallel to the foliation which varies from N3°E to N40°E and dips 25° due NW. Overbreaks and rock squeezing occurred in the schist and fault gouge (Paudel et al. 1998). The gouge comprises more than 80% of non-swelling clay. A maximum deformation recorded in squeezing sections was 150 cm, i.e. 30% of the diameter of the tunnel (Sharma 2000). Consequently, the tunnel had to be reshaped in those sections and a concrete lining was provided for the final support.

Rock support design

In the previous designs and drawings, 10 cm thick shotcrete with welded wire mesh and 2 m long systematically grouted rock bolts or steel ribs was recommended as an initial support (Sharma 2000). Similarly a 30 cm thick concrete lining was recommended for the permanent support in the whole length of the headrace tunnel (Sharma 2000). However these rock supports were found conservative according to the modern philosophy of rock support design. Therefore they were not considered and the rock support was carried out on the basis of the Q system (Paudel et al. 1998; Sharma 2000).

Some minor modifications to the recommended support were made to suite local ground conditions. Fibre-reinforced or wire-mesh shotcrete and a pattern of rock bolts are the main rock supports used in this project. In addition steel ribs and concrete linings were used in the stretch where extremely poor rock conditions prevailed. The RMR classification system was also used mainly for estimating

Table 5: Rock mass distribution in the headrace tunnel of the Modi Khola Hydroelectric Project based on the Q system (Sharma 2000)

Rock class	Q-value	Percentage
Good to poor rock	1 – 40	72
Very poor rock	0.1 – 1	15
Extremely to exceptionally poor rock	<0.1	13

Table 6: Recommended final supports for the Modi Khola Hydroelectric Project (Sharma 2000)

Q-value	Rock support type
< 1	20 to 30 cm thick concrete lining
>1	10 cm thick wire mesh and shotcrete with pattern of bolting or spot bolting

Table 7: Recommended tunnel supports for the Ilam Hydroelectric project (Singh et al. 2002, modified from Bieniawski 1989)

Q-value	RMR	Rock mass class no.	Description of rock mass class	Support required
<1	0–20	V	Very poor rock	i) Steel rib ii) 36 cm thick concrete at side walls and 20 cm thick concrete at bottom slab. iii) 36 cm thick shotcrete at crown
1 – 4	21–40	VI	Poor rock	Bolts in pattern with 1 x 1 m spacing 5 cm thick plain shotcrete ; 7.5 cm thick shotcrete with wire mesh
4 – 10	41–60	III	Fair rock	Bolts in pattern with 1.25 x 1.25 m spacing 5 cm thick, plain shotcrete; 5 cm thick shotcrete with wire mesh
10– 40	61–80	II	Good rock	Bolts in pattern with 1.5 x 1.5 m spacing 5 cm thick, plain shotcrete
>40	>81	I	Very good rock	Spot bolting and 5 cm thick plain shotcrete

the stand-up time and cross checking the designed support. Geological logging and rock mass classification were conducted throughout the underground works. The deformation in rock-squeezing zones (e.g. in the pressure tunnel) was recorded with a measuring tape.

The rock mass distribution in the headrace tunnel based on the Q system is presented in Table 5. For final lining, mainly two types of support were recommended (Table 6). As a result, about 60% of the tunnel section was supported by a combination of a pattern of rock bolts or spot bolting with a 10 cm thick layer of wire mesh shotcrete and the remaining sections were supported by a concrete lining (Sharma 2000). In this project the "leaky" shotcrete or concrete lining design was adopted.

Ilam (Puwa Khola) Hydropower Project

The Ilam (Puwa Khola) Hydropower Project is a run-of-the-river scheme and its installed capacity is 6.2 MW. The power plant utilises a 304 m high head where a 4.5 m concrete diversion weir diverts up to 2.5 cumecs of water from the Puwa Khola into a 3.24 km long headrace tunnel of 2.5 m diameter with a regulating pond of 2057 m^3 capacity. It has 40 m long and 5 m wide two underground desanding basins. The project's surface penstock is 1001 m long, steel lined and inclined at 45°. The NEA is the owner of this project. The project commenced in October 1995 and was completed in August 2001 with a total cost of US$ 15.5 million.

Geological conditions

The project area mainly comprises moderately to slightly weathered, grey, coarse- to very coarse-grained, thickly foliated, strong augen gneiss and banded gneiss with sporadic strong and fractured quartzite bands (Singh and Shrestha 2002). The tunnel axis is almost perpendicular to the gently (<10°) dipping foliation (Singh and Shrestha 2002). The gneisses contain from 1 to 20 m thick shear zones running parallel to the foliation where the major problems of overbreak and rock squeezing were encountered. The shear zones include frequent bands of clay gouge ranging in thickness from a few millimetres to 15 cm.

Rock support design

The Q and RMR systems were used for the classification of rock mass (Singh and Shrestha 2002). The support recommended by the Q system was the main design basis in this project. The recommended tunnel supports for this project are shown in Table 7.

Some modifications to the recommended support were made to suite local ground conditions and to speed up the progress. Wire mesh shotcrete and a pattern of rock bolts were the main rock supports used in this project. In addition steel ribs and concrete linings were also applied in stretches where very poor rock conditions were encountered. The RMR classification system was used to estimate the stand-up time of the excavated rock mass for the application of a rock support. Geological logging and rock mass classification were carried out throughout the underground works. The rock mass distribution in the headrace tunnel based on the Q system and RMR is shown in Table 8. Final supports were applied according to the recommendations given in Table 8. Weep holes were provided in the shotcrete and concrete linings to relieve the external porewater pressure. Monitoring was carried out only in the rock-squeezing section of the headrace tunnel with a measuring tape.

Table 8: Rock mass distribution in the headrace tunnel of the Ilam (Puwa Khola) Hydropower Project based on the Q system (Singh et al. 2002)

Rock mass class	Description of rock mass class	Percentage
I – II	Very good to good	38
III – IV	Poor to fair	32
V	Very poor	30

Kali Gandaki Hydroelectric Project

The Kali Gandaki A Hydroelectric Project is located about 180 km west of Kathmandu. It is a run-of-the-river scheme with 6 hours of daily peaking capability and 144 MW of installed capacity. The project includes a 100 m long and 43 m high concrete gravity diversion dam, a surface desander basin, a 6 km long tunnel of 7.4 m finished diameter, and a surface powerhouse. The tunnel shortcuts the 45 km long natural loop of the Kali Gandaki River to gain a net head of 115 m. A pressured flow of 141 m^3/s feeds three Francis turbines in the powerhouse. The NEA is the owner of this project with a total cost of US$ 380 million.

Geological conditions

The project area is occupied by the Darsing Dolomite and Andhi Khola Formation of the Kali Gandaki Supergroup of Late Precambrian to Early Palaeozoic (?) age (Sakai 1986). The phyllite is overlain by the dolomite. The dolomite is cherty in nature and is highly jointed and brecciated. Similarly the crenulated phyllite is weak to moderately strong and alternating with quartzite bands.

Table 9: Summary of recommended primary tunnel supports for the Middle Marsyangdi Hydroelectric project (NEA and Fichtner Joint Venture 2000)

Rock mass classes	Rock mass quality	RMR	Support required
I	Very Good	81–100	2 m long grouted rock bolt and 3–5 cm thick shotcrete where required
II	Good	61–80	3 m long grouted rock bolt and 10 cm thick shotcrete mainly at crown
III (a and b)	Fair	41– 60	III a = 3 m long grouted rock bolt and 5–10 cm thick shotcrete at crown and wall III b = 4 m long grouted rock bolt and 10–15 cm thick shotcrete at crown and wall
IV (a, b, and c)	Poor	21–40	IV a = 4 m long grouted rock bolt at crown and 15–20 cm thick shotcrete with deformation joints at crown, wall and Invert IV b = 4 m long grouted rock bolt at crown and 20–25 cm thick shotcrete with deformation joints at crown, wall and invert IV c = 4 m long grouted rock bolt at crown and wall and 25–30 cm thick shotcrete with deformation joints at crown, wall and invert
V (a, b, and c)	Very Poor	<20	V a = 4 m long grouted rock bolt at crown and wall with 20–25 cm thick shotcrete with steel arch at roof and wall V b = 4 m long grouted rock bolt at crown and wall with 25–30 cm thick shotcrete with steel arch at roof and wall V c = 4 m long grouted rock bolt at crown and wall with 25–30 cm thick shotcrete with steel arch at roof and wall; corrugated steel planks at roof and wall
VI (a and b)	Extremely Poor	Not measurable	VI a = Arch steel support at 0.5 to 0.75 m spacing with 20–25 cm thick shotcrete at roof, wall, and invert VI b = Arch steel support at 0.5 to 0.75 m spacing with 20–25 cm thick shotcrete at roof, wall, and invert; 4 m long grouted rock bolts

Rock support design

In this project the Q, RMR, and GSI classification systems were used for rock support design. They were applied to classify the rock mass and its possible extent of failure surrounding the tunnel. It was found that these classification systems were very difficult to implement in the field and that none of them could be relied upon to give an accurate description of the rock mass (Hoek 1999). However the GSI system was largely used to classify the continuous or monolithic rock (i.e. phyllite or slate). In addition the computer software Phase 2 was also applied for rock support design.

The primary rock support was a combination of the pattern of 4 m long and 25 mm in diameter untensioned and fully grouted dowels and a 15 cm thick layer of steel-fibre-reinforced shotcrete (Hoek 1999). The dowels were installed radially and were spaced at approximately 1.2 m and 1.5 m axially. In addition the steel sets embedded in shotcrete were also applied in the stretches with poor rock quality. These systems were installed at a distance of 0.5 to 1.5 m behind the face. No support was provided for the floor at the face but a concrete invert slab, approximately 40 cm thick, was poured at 50 to 100 m behind the face. Convergence measurements and rock pressure measurements were carried out to monitor the deformation behaviour of rock mass for the design of a final concrete lining. The final concrete lining was placed in completely stable tunnel which carries no load other than its self-weight. The "leaky" final lining was adopted to avoid a very heavily reinforced concrete lining to release the surrounding water pressure.

The software Phase 2 was used in order to understand the behaviour of tunnel under various stress conditions and to design the primary support as well as final lining (Hoek 1999). The model includes a primary support in the form of fully grouted untensioned dowels and a layer of steel fibre reinforced shotcrete. Based on that model, excavation was carried out at a number of stages by providing appropriate support wherever necessary during the advancement of the tunnel face. Once the full tunnel profile had been excavated and the tunnel deformation had been stabilised by the installed primary support, the casting of the side beams, crown, and the invert of the final lining were simulated in the model. When the final concrete lining was placed in the completely stable tunnel, it carried no load other than its self-weight. The next stage was to simulate the loading of the tunnel lining by events that could occur after the lining was in place and the project is in operation.

The following assumptions were made (Hoek 1999) in modelling:

- The load carrying capacity of shotcrete was considered not to be zero.
- The load carrying capacity of rock bolts was considered to be zero assuming corrosion of the rock bolts in the long term.
- The long-term creep load was considered to be 10% of the in situ stresses.
- The water pressure, acting internally and externally, was also considered in the loading of the final lining.

Middle Marsyangdi Hydroelectric Project

It is located about 170 km west of Kathmandu in the Lamjung district. It is a daily pondage type run-of-the-river scheme with an installed capacity of 70 MW. The project utilises a gross head of 110 m with a discharge of 80 m^3/s in driving two Francis turbines of 36 MW capacity each. The major components of the project are: a 62 m high 95 m long combined concrete gravity and rockfill dam with a concrete spillway with a capacity of 4270 m^3/s, 3 numbers of 12 m x 19.5 m spillway gates, a peaking reservoir of 1.6 million m^3 capacity, three numbers of underground desanding caverns (15 m x 100 m x 25 m each) with two basins in each cavern to flush 95% of 0.2 mm particle size by a vertical flushing system, a 5.4 m diameter and 5210 m long concrete-lined circular headrace tunnel, a 20 m diameter and 45 m high surge tank, a 450 m long penstock, a surface powerhouse, and a 41 km long single circuit 132 kV transmission line. The NEA is the owner of this project whose estimated cost is about US$ 195 million.

Geology and rock support

The project area comprises quartzites, phyllites, and schists. In this project the RMR system has been used for designing the primary rock support during tunnelling. A combination of shotcrete and a pattern of rock bolts are the main primary supports. In addition a steel arch support with shotcrete is recommended in very poor to extremely poor rock. The primary supports are divided into six classes (NEA and Fichtner Joint Venture 2000) according to the RMR rating (Table 9). The rock support classes III, IV, V, and VI are further subdivided into a, b, and c types. About 20–25 cm thick "leaky" concrete lining throughout the headrace tunnel has been designed for final lining (NEA and Fichtner Joint Venture 2000) to reduce the head loss. There are altogether 12 recommended primary supports for various site conditions. However, it seems that there could be some difficulties in identifying precisely the various site conditions designated for each recommended support.

CONCLUSIONS

The type of rock support for any tunnel or cavern depends mainly on the failure mechanisms and factors influencing the stability of underground excavations. The Q and RMR systems (with some modifications) have been extensively applied for the rock support design in most of the hydropower projects of Nepal. Large-diameter tunnels are found to be fully concrete lined whereas small-diameter ones are either shotcrete lined or left unsupported according to the modern rock support design philosophy. A "Leaky" final shotcrete layer or concrete lining is applied in most tunnels to avoid very heavily reinforced concrete sections and to withstand the external hydraulic head.

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