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Engineering geological challenges in the development of hydropower in Nepal

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ABSTRACT

The major geological challenge for underground structures is to predict and confirm reliable rock mass condition due to its jointed, faulted, sheared, and folded natures. Constructing tunnels, dam and powerhouse through such rock mass is risky and are susceptible to very serious problems like cave-in, rock squeezing, water ingress, slope instability, structural failure, etc. These geological uncertainties can offset the construction schedule, causing substantial increase in the cost of the project. Therefore, a thorough and convincing geological investigation are very essential to identify and predict reliable rock mass condition prior to the actual construction that can minimise project cost significantly. Proper geological investigation is the key to planning, predicting rock and soil behaviour, economical design, and cost estimation of any hydropower project. The degree of accuracy in predicting geological conditions, evaluation, and interpreting the quality of rock mass during planning phase is a key measure for the successful completion of any hydropower project. Case studies of such geological challenges faced in the Khimti I hydropower project of Nepal is highlighted in this paper.

Keywords: Geological challenges, cave-in, squeezing

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INTRODUCTION

A challenge being posed by geological considerations, while developing hydropower in a country like Nepal, which is tectonically considered to be active, is an important issue today. Majority of the hydropower potential areas are located in the mountainous regions. The major power project structures are possible only by piercing through geologically sensitive tunnels and rearranging ecologically fragile slopes. Geological challenges are more pronounced in tunnels and underground structures due to difficulties in predicting geological condition without very high level of geo-scientific investigations. Investment in any tunnelling project without identifying geological challenges is economically and physically very risky. The major geological challenges during study and construction stages are (1) preparation of precise geological map and prediction of rock mass quality, (2) efficient investigation due to limited equipment, skilled manpower and laboratory test facilities, (3) cave-in and flowing ground due to extremely poor rock mass condition, excess ground water and poor tunnelling technology, (4) water leakage and ingress of water due to permeable ground, open joint and karst features (5) rock squeezing and rock burst due to stress overloading, and (6) slope instability due to huge slope cutting or reservoir rapid draw down etc. Geology, not only determines construction cost, it can also affect long-term maintenance challenges such as slope stability, sediment deposition, and arresting ground water leakage and seepage.

This paper describes geological challenges in the development of hydropower projects and suggests some of the options that can be considered for minimising the challenges. Some case studies of such geological challenges faced in the Khimti hydropower project of Nepal is highlighted.

FAULT SYSTEM OF NEPAL HIMALAYA

The Nepal Himalayas was formed as a result of collision of the Indian Plate and the Tibetan Plate about 50 million years ago (Kizaki 1994). Due to continuous thrusting of the Indian Plate, rocks of the Himalayas are faulted, sheared, and folded as a result of which, the geology became more complex. The Nepal Himalayas comprises majority of metamorphic and sedimentary rocks with few granite intrusions. Three major thrust fault systems, namely, Himalayan Frontal Thrust (HFT), Main Boundary Thrust (MBT) and Main Central Thrust (MCT), divide the rocks of the Himalayas into Siwaliks, Lesser Himalayas and Higher Himalayas from south to north (Fig.1). The MBT and HFT are active faults and are seismically most active thrusts that can create major problems during construction of any infrastructures. On the other hand, based on the field evidences, the MCT is no longer active and will not to create problems. Besides these major thrust faults, there are several minor faults and shear zones. Most of all faults and shear zones systems generally extend from east to west of Nepal, which are oriented parallel to foliation with north

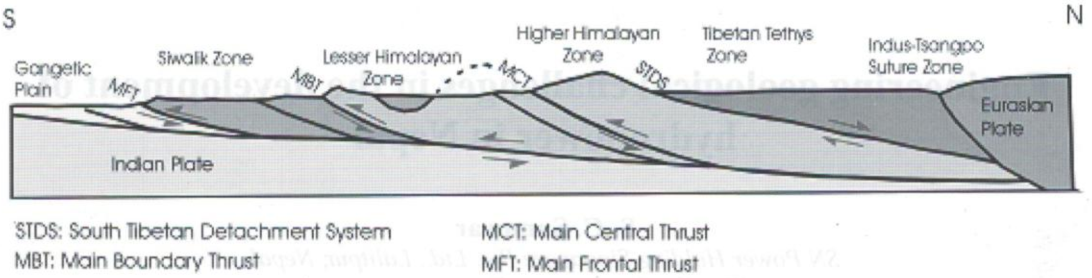

Fig.1: Generalised cross section of the Himalaya showing the major tectonic features of the Nepal Himalaya (Harris and Whalley 2001).

dipping. Most serious geological problems in development of infrastructures are generated by faults and shear zones (Sunuwar 2006). These faults and shear zones are responsible for posing geological challenges, and therefore, precise geological maps showing faults and shear zones are crucial for development of any hydropower project.

WHY DOES GEOLOGY PLAY AN IMPORTANT ROLE?

Hydropower structures are either built in or are founded on geologically sensitive materials such as rock and soil. Rock mass are usually found to be faulted, sheared, jointed and folded and are structurally more complex, having anisotropic properties. Faulted, sheared and jointed rock mass pose serious problems like tunnel collapse, rock squeezing, water ingress, landslide, etc. during underground construction of underground structures. In addition, decisions taken on the finalisation of tunnel alignment, dam location, powerhouse location including construction technology, needs to be completely based on geological condition. Hence, geology strongly affects almost every major decision that is made during planning, design, and construction phases of a tunnel, dam and powerhouse. In addition, geology dictates the project cost by predicting geological problems and their behaviour and prescribing cost-effective solution and measures. The more accurately the geological condition of a proposed structure can be predicted, the more accurately the cost of a structure can be estimated. Consequently, full understanding of regional geology with fault systems and structures including hydrogeology is first step for recognizing geological challenges of a hydropower project.

Geological challenges

Some of the geological realities for understanding its challenges are briefly mentioned below:

- Underground structures have several uncertainties,
- Understanding of regional geology with faults system and hydrogeology,
- Precise geological mapping with identification of faults, shear zones, weak zones and ground water zones are a challenge in itself,
- Prediction of rock mass quality for tunnels and caverns requires thorough investigations,
- Prediction of ground water regime and its analysis are the most difficult task during and post construction phases,
- Every aspect of the geologic investigation for tunnels is more demanding than investigation for traditional foundation engineering projects,
- Limited facilities in site investigation technology and laboratory tests, and
- Limited experienced manpower.

The geological factors related to geological challenges are (1) extremely poor rock mass quality due to low strength, faulting, shearing, jointing and weathering (2) stress overloading in rock mass, and (3) influence of ground water.

Challenges during study stages

The challenges during study stage are summarised below:

Geological mapping and prediction of rock mass

Precise geological maps showing distribution of rock types, faults/shear zones, joints, folds and ground water condition are essential for planning and design of any project. Hence, in absence of reliable and precise geological maps, the project is subject to many uncertainties, that are likely to be very costly at a later stage of the project cycle. It has been experienced that occurrence of fault, shear zone and weak zone make great difficulties and can create complications during underground excavation. Therefore, more focus shall be given to identify faults/shear zones, rock mass distribution and ground water condition.

Some of the important parameters for estimating rock mass quality during geological mapping are summarized below:

Rock type

High strength and good quality rock is preferred for tunnelling and laying of foundation whereas low strength and poor quality rock mass will require heavy structures

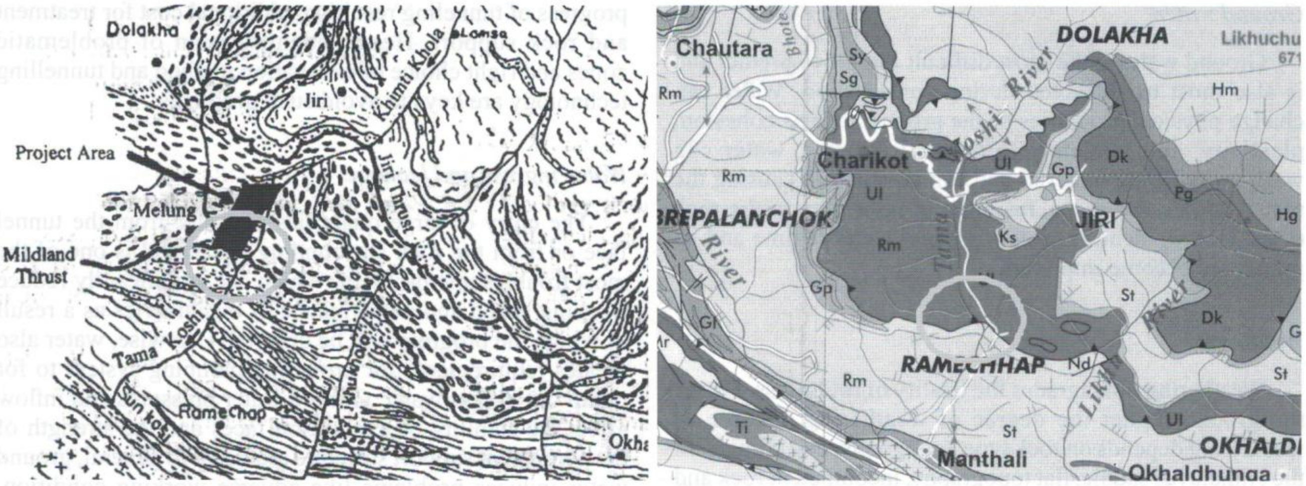

Fig 2: The Midland thrust showing in geological maps (published by Ishida and Ohta in 1973 and Map house).

and flexible rock support to prevent differential settlement in foundation and rock squeezing in tunnels. Hence, identification of low strength rocks like mica schist, phyllite, shale, chalk, evaporate, soap stone etc. are prerequisites to fix the alignment for tunnels and design related undertakings.

Fault and shear/weak zones

The faults and shear zones are responsible for posing major geological challenges. Accurate identification of faults and shear/weak zones along the tunnel alignment are very essential for design, fixing alignment, selecting rock support, deciding construction technology, cost estimation etc. Orientation, extension, thickness, blocks and matrix natures, ground water conditions etc. of faults and shear zones are important parameters. However, identification of faults in Nepal Himalaya is challenging due to low angle dipping faults system along foliation plane, accessibility, steep topography, poor rock exposures and vegetation. Fault leaves some evidences at surfaces which are clues for identification. Remote sensing techniques are useful to identify faults/shear zones by using aerial photos and satellite images. Likewise field evidences such as gouge, slickensides, displacement etc. are physical way to identify faults/shear zones in good rock exposures. However, field evidences are not easily observe in the terrains where thick soil cover, vegetation, deep weathering etc. are prominent. In such terrains morphological features such as depression, saddle, deep gully, extensive vegetation etc. are more useful to identify faults/shear zones. In addition, drilling and geophysical methods (seismic, electric resistivity, ground penetrating radar, etc.) are useful to detect and identify faults and weak zones.

One misleading example of a major fault called Midland Fault can be considered for precise geological map. It is shown in regional geological map (Fig. 2) at Tamakoshi River area of Dolakha District which separates the augen gneiss and phyllite. In the field at near confluence of Khimti Khola and Tamakoshi River, there is a sharp contact between Augen Gneiss and Kuncha Phyllite and hence, there is no any physical evidence for presence of this fault (Fig. 3). Therefore, it is very essential to check physical evidences of faults in field during geological mapping.

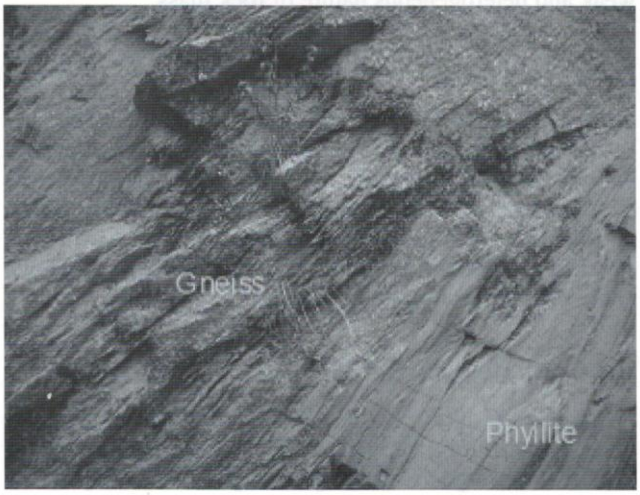

Fig. 3: The sharp boundary between Augen gneiss and Kuncha phyllite observed near confluence of Khimti Khola and Tamakoshi River which confirms no shearing evidence from engineering point of view.

Joints

Orientation, nature and numbers of joint sets control the stability of underground excavation and slope stability. More the number of closely spaced joint sets more likely that the quality of rock mass becomes poorer and increase instability. In addition, presence of ground water will reduce joint shear strength and facilitates failure condition. Therefore, collection and analysis of joint sets and their properties are important for designing of tunnel, slope and foundation.

Ground water

Ground water is the most difficult regime to predict and is also most troublesome during construction. Water can change physical properties of the ground such as cohesion, plasticity, and tendency to swell. Likewise, water can wash away filling material and can create void causing the matrix to become loose, resulting in instability in the rock mass. Hence, identification of ground water regime and its properties become important.

Weathering

Weathering downgrades the quality of rock mass. Flatter the ground higher the degree of weathering. Degree of weathering depends on rock type, topography and climate. In the Himalayas saddle, flat topography, incompetent rock and faulted area undergo high degree of weathering. Weathering creates problems like instability of tunnel and slope due to weak and heterogeneous nature of rock mass.

Karst

Karst features such as sinkholes, solution cavities and caves are generally present in limestone and dolomite terrain. This topography creates problems like collapsing of foundation, loss or ingress of water and requires filling or shielding of voids for engineering structures which is expensive. Hence investigation of karst features in limestone and dolomite terrains are necessary.

Insufficient site investigation

In general, the more investment is made on the site investigation during study phase, less problem is faced during construction stage. As of the general practice in Nepal, it is estimated that very less percent of the cost of the project is allocated for site investigation in hydropower projects. It must also be mentioned here that as of the present situation, in Nepal, there are limited site investigation equipment, skilled manpower and laboratory test facilities for effective site investigation. In addition site investigation is difficult due to lack of access and mountainous terrain. The situation, however, is changing fast and geology is receiving greater importance than before, albeit insufficient, compared to the prevailing international practice and standards.

Challenges during construction

The followings are challenges induced by lack of identification and construction technology during construction period.

Cave-in and flowing ground

Probable chances of occurrence of cave-in and flowing ground in tunneling are due to the weathered, decomposed, very jointed rock, fault and shear zone in seepage areas and poor tunneling excavation method. These problems delay the progress of tunneling resulting additional cost for treatment and rock support. Hence, identification of problematic zones and right choice of excavation method and tunnelling technology are keys to avoid such problems.

Water ingress and leakage

The entry of large quantities of water from the tunnel face or from the rock surrounding the tunnel is one of the most troublesome problems which can significantly reduce working speed and performance of machineries as a result of which the progress will be delayed. Likewise, water also creates requirements for providing pumping system to for pump out excess water. Ground water pressures and inflow affect the stability of excavation faces and the strength of the permanent support structure required. In general, ground water induces problems like adverse working condition, ground heaving/clay swelling, corrosion, slope instability and foundation settlement. In addition, water leakage through and into a finished tunnel severely affects the quality of the space, loss of generation energy and is difficult to correct. Hence, identification of ground water regime is essential for underground structures which help to select treatment technology and equipment for grouting and draining out.

Rock squeezing and swelling

Rock squeezing is a common problem in the Nepal Himalayas while tunnelling through low strength rock, fault and shear/weak zone (Sunuwar 2002). It can occur even in shallow depth where considerable amount of non-swelling clay is present. It reduces cross-section of a tunnel by time dependent deformation of rock. It may stop or continue for a long time. In the worst case, the tunnel itself can be a failure. The main challenge is to reshape and re-support the tunnel which is time consuming and expensive.

Presence of swelling clay (such as montmorillonite, smectite) in contact with water causes ground heaving, deformation and cracking of concrete due to time dependent increase in volume. The areas that contain shale, mudstone anhydrite and marl rocks are most likely to swell. If swelling clay area is not shielded properly from water contact, it can give long term instability problem.

Rock spalling/bursts

Hydropower tunnels, in general, are located at a shallow depth and therefore, are less likely to experience rock spalling/bursts. Rock spalling/bursts are likely whenever the overburden depth is greater than 700 m and uniaxial compressive strength is greater than 100 MPa. It becomes a serious problem below 900 m. Based on Norwegian experience, (Broch and Sorheim 1984) rock bursts are likely to occur if it is in a valley side with heights above the tunnel is 500 m or more and are aligned at an angle of 25° or steeper. In severe rock bursts condition, it becomes very dangerous for tunnel crew due to sudden and violent explosion of rock and failure of rock support system.

Slope stability

Slope failure problem is very common when changes are made in the original ground profile. In addition, rapid draw down in reservoir and earthquake also generate slope failure. It can cause serious structural damage or even loss of life. Soil/rock type, slope angle, discontinuity condition and pore water pressure are very important governing factors which have to be analysed properly during detailed study stage.

IDENTIFICATION AND POSSIBLE MEASURES OF GEOLOGICAL CHALLENGES

Proper planning, appropriate site investigation and interpretation of available data identify geological challenges whereas right choice of construction technology and methodology helps to tackle identified geological challenges. In addition, expert inputs are necessary to find the solutions. Some of the options that can be considered for minimizing the challenges are suggested below.

Appropriate site investigation

Site investigation helps to identify and estimate geological challenges. It can also save project cost by either advanced preparation for tackling the problems or by relocating major structures in safe locations. It has been proven that thoroughly investigated tunnels have fewer cost overruns and fewer problems. Site investigation helps in acquisition of critical data from site for the purpose of project planning, selection of sites, design, cost evaluation, construction and for preparation of tender documents. Therefore, proper planning and investment in site investigation will help to identify possible problems to reduce uncertainties. Reliable interpretation of available geological data is important part of the site investigation. Site investigation starts from pre-feasibility and can continue until construction phase to sort out geology.

In general, the cost of the geotechnical investigation ranges from 0.5% to 3% of the total cost of the project and could be higher up to 8% also in complex underground projects (Parker 1996). The recommended 3% cost for site investigation is practical in comparison to the 12% average costs for payment of claims usually claimed against unexpected subsurface conditions (Parker 1996). Similarly the World Bank report concluded that actual construction costs for hydropower projects were on average 27% above estimated costs, whereas schedules were on average 28% longer than estimated (World Bank 1985). In addition, past experience in hydro preparation costs indicate that, on average, less than 1% of the total project cost is spent on feasibility, pre-feasibility, reconnaissance and hydrological studies before the engineering design is undertaken which is remarkably low compared to the potential cost overruns (Hoek and Palmeiri 1998). Hence, more investment with proper planning in site investigation is appropriate way to quantify geological challenges and make the project cost effective.

There is the Guideline for Study of Hydropower Projects prepared by Department of Electricity Development (DoED 2003), Ministry of Water Resources which provides good site investigation guidelines. However, there is not fixed site investigation rule, it varies from project to project depending on geology.

In practice, geological mapping can be considered as the first stage of the site investigation. It provides information on discontinuities, structures, rock and soil distribution and their properties. Identifying faults, shear/weak zones and problematic areas in field and projecting in the tunnel level are big challenges. The more accurately the geology of a proposed tunnel can be ascertained, the more accurate will be the cost estimates. Despite that, only the surface geological mapping is not sufficient to predict the properties of the subsurface rock and soil. Consequently, subsurface exploration by drilling, geophysical techniques, excavating test adit and trenching are necessary to collect subsurface properties of rock and soil. Carefully placed bore holes, exploration adit and geophysical survey provide an accurate 3 dimensional picture of the geological condition. Geophysical survey and trenching are second stage in the process to get subsurface information of rocks and soils. The geophysical survey provides information on rock head, rock mass quality, weak zones and groundwater regime. It also helps to plan and select location for drilling and to estimate the depth of drilling. Drilling is expensive in mountainous terrain and therefore, is normally confined to difficult and important locations only. Diamond core drilling is more applicable in site investigation of Nepal because, most of the drilling locations consist of boulder mix soils and bedrock. Drilling is the only reliable means to get at certain depth of tunnel for testing and collecting samples. In Nepal there is less site investigation practice along a tunnel alignment due to difficult topography and deep drilling and rely on geological mapping where more uncertainties. In situ tests in drill holes and test pits in addition to laboratory tests of collected samples are necessary to estimate rock and soil properties for design purpose. In most of the projects having major underground structures, test adit is necessary to understand behaviour of rock mass, to estimate rock mass quality, to measure in situ stress and deformation modulus, etc. Reliable laboratory tests results of rock and soil properties are important for design and predicting geological problems. Finally, interpretation of collected data, predicting geological challenges and proposing design are the most important tasks.

RIGHT CHOICE OF CONSTRUCTION TECHNOLOGY AND METHODOLOGY

Right choice of construction technology during construction will certainly reduce geological challenges and speed up progress. Prediction of geological challenges before construction stage is great advantage to minimize the risk. After predicting geological challenges, it can be controlled and prevented by selecting right choice of construction technology and methodology. In addition, selection of

Table 1: Rock mass distribution of headrace tunnel based on the Q-system (KSC 2000)

Rock class	Q-value	Percentage (%)
Fair to poor rock	>1	27%
Very poor rock	0.1 – 1	43%
Extremely poor rock	0.01 – 0.1	22%
Exceptionally	<0.01	8%

experienced contractor is another important prerequisite. For instance, in a squeezing ground, special techniques, such as pre-reinforcement, over-excavation, compression longitudinal slots in the shotcrete lining, yielding rock bolts, sliding steel sets and secondary shotcrete lining with sequential excavation and support are more effective. Likewise pilot hole drilling at face of the tunnel and pilot tunnel are very useful and informative to find out rock mass, ground water condition, opportunity to drain and reinforce rock mass ahead.

Panel of expert

One of the most effective means of overcoming the geological challenges is to establish a review panel of expert to assist in identifying geological challenges and solution quantifying in terms of cost and time from the early stage of the project study. In addition the expert suggests inadequacy of geological data, appropriate interpretation of the available data and appropriate solution to tackle the challenges. The preparation of or the review of specifications and contract documents from the experts are other important tasks to deal geological challenges during construction.

Case studies

Some of the geological challenges faced during implementation of Khimti Hydropower project is presented as practical examples:

Khimti Hydropower Project (60MW)

The Khimti Hydropower Project is a "run-of-the-river" project located in approximately 175 km due east of Kathmandu, Nepal. The power plant has the highest head of 686 m in Nepal. A concrete diversion weir diverts up to 10.75 cumecs of water from the river Khimti into 7.9 km long headrace tunnel and then through a 913 m long, steel lined penstock, to an underground powerhouse. The country rocks are augen gneiss with weak schist bands belonging to the Lesser Himalayan tectonic zone.

The geological challenges encountered in the project were summarised as below.

Insufficient geological study

The Khimti Hydropower Project was directly launched from pre-feasibility stage to construction stage. Therefore detailed geological investigation was not carried out. There was no precise geological map showing weak schist bands. Schist bands were few centimetres to tens of metres thick, deformed and gently dipping holding ground water above. Due to gently dipping nature of schist bands and low strength, tunnel excavation was difficult. Problems like tunnel collapse, water inflow and rock squeezing were common. The presence of schist bands and sheared rocks downgraded the quality of rock mass as a result estimated rock mass quality became just reverse i.e. worse rock mass quality was higher (Table 1). Finally rock support cost was very high.

Table 2: Major cave-ins records of the Khimti Hydropower Projects (KSC 2000)

Location	Date of occurrence	Size (lxbxh) m^3	Remedial measures
Adit 1			
Headrace D/S Ch.363-344m	29 Aug. '97	19x5x4	Shotcreting and rock bolting.
Ch.330-390m	18 Sept. '97	60x15x20	New alignment from ch. 328m.
Adit 2			
Headrace D/S Ch.230-260m	12 Aug. '97	30x12x15	New alignment from ch. 230m.
Adit 3 Ch.194-210m	19 Sept. '98	16x12x10	Shotcreting and pattern of bolting.
Headrace D/S Ch.12-26m	31 Oct. '98	14x12x16	New alignment from ch. 8m.
Adit 4 (Old) Ch.44-50m	9 Feb. '95	6x7x15	Abandoned and selected new alignment.
Adit 4 (New) Ch.44-50m	18 Aug. '96	6x7x9	Grouting, blasting, shotcreting and concrete lining.
Adit 5 Ch.110-115m	15 Dec. '94	5x6x8	Timber support, excavation and stone masonry.
Old upper pressure shaft	19 April. '97	??	Abandoned and changed 50m inside from the shaft.
Ch.186m onwards New upper pressure shaft Ch. 352-364m	14 July '98	12x15x10	Grouting, spiling, cutting and supported by reinforced ribs of shotcrete.
Tailrace Downstream Ch.129-135m	6 June. '97	6x7x6	Grouting, cutting and supported by in situ concrete lining.

Tunnel cave-in

Tunnel cave-ins were encountered in the tunnel section where weak schist bands, fault/shear zones, very jointed rock and considerable amount of seepage are noted (Table 2). There were 13 major cave-ins, all were in weak schist bands and shear zones. Main causes of caving are weak rocks with extremely poor rock condition having short stand- up time, ingress of water, and poor excavation method.

These cave-ins delayed tunnel excavation progress. Delay time of tunnels and adits were listed in the Table 3. For example, more than one year was delayed in upper pressure shaft due to 3 catastrophic caving. First caving delayed 319 days, which was before the financial closure. Therefore, this delay time had not hampered construction schedule. Stabilisation of caving in inclined pressure shaft is very difficult due to inclination and working space and it takes long time. Similarly, delay time of each major caving in horizontal tunnels was almost 4 weeks. Excavation and supporting work in loose caving debris is very difficult task, risky and sluggish process.

Rock squeezing

Rock squeezing problems encountered during excavation in different sections of the tunnels are summarised in Table 4.

Table 3: Record of delay time due to caving (KSC 2000)

Location	Delay time (Days)	Remarks
Adit 1	48	
Adit 2	68	
Adit 3	31	
Adit 4	70	
Adit 5	7	Excavated before financial closure
Pressure shaft	667	1st caving delayed 319 days before financial closure.
Access tunnel	11	
Powerhouse	1	
Tailrace tunnel	30	

Mild to moderate squeezing observed mainly in decomposed schist (Sunuwar et al. 2001). The maximum deformation recorded is 40 cm (deformation as large as 13% of the tunnel diameter) in Adit 5 tunnel at chainage 245 m (Table 4). According to the monitoring data, squeezing generally lasted for 5 to 12 months. In the squeezing section progress was very slow i.e. 7-10 m per week and heavy rock support of 25cm thick fibre reinforced shotcrete with grouted rock bolts or in situ concrete lining had to provide which increased the cost of the project. In some case re-profiling and re-supporting of the tunnel was required leading to increase of construction time and costs.

Water leakage and ingress

Water leakage and ingress problems were encountered particularly in headrace tunnel section between Adit 1 and Adit 3. Water leakage was favourable due to jointed and fractured nature of rock therefore injection grouting was carried out in headrace tunnel. Design water leakage was 150 litres per second in 7.9 km long. The headrace tunnel section at ch. 240 m from Adit 1 water ingress was very high (3000 litre/minute) which made very difficult in tunnel excavation. In this section fault was parallel to the tunnel axis, which acts like an aquifer. Similarly in the headrace tunnel upstream and downstream from Adit 2 water ingress problem hampered in excavation work. Water leakage problem was severe in the Adit 2 headrace upstream and downstream from ch. 0 to 150 m during water filling time. In this section tunnel was towards valley side and rock contained vertical open joint. The total measure leakage was 200 litres per second. This leakage was stabilised by injection grouting. It is experienced that water leakage problem is likely to occurred in jointed/ sheared strong rocks with open joint close to valley side.

Table 4: Some examples of squeezing in the KHP tunnels (Sunuwar and Fowell 2001)

Location	Q-value	Rock type	Deformation (mm)	Tunnel conditions
Adit 1 Headrace D/S Span 4m. Ch.465-620m.	0.003-0.006	Schist	50-170	Alignment parallel to foliation (<25º/295º). Badly cracked shotcrete (10cm wide cracks). Damp to wet seepage condition.
Adit 2 Headrace D/S Span 4m. Ch.1276-1280m.	0.03	Schist in wall	6	Alignment parallel to foliation (<25º/315º). Shotcrete badly cracked. Damp seepage condition.
Ch.1276-1280m.	0.03	Schist	60	Alignment parallel to foliation (<25º/315º). Shotcrete badly cracked. Dripping seepage condition.
Adit 3 Headrace U/S Span 5m. Ch.178-234m.	0.3	Schist in wall	40-63	Alignment parallel to foliation (<20º/262º). Shotcrete badly cracked. Damp seepage condition.
Adit 4 (Old) Span 3m. Ch.58-76m.	0.004	Schist	100-300	Alignment parallel to foliation (>50º/015º). Buckling of walls. Damp to wet seepage condition.
Adit 5 Span 3m. Ch.222-245m.	0.01	Schist & sheared gneiss	300-400	Alignment 50° oblique to foliation (<25º/020º). Shotcrete badly cracked. Wet seepage condition.
Tailrace D/S Span 5m. Ch.68-80m.	0.3-0.4	Schist bands & sheared gneiss	2.4	Alignment perpendicular to foliation (>50º/010º). Shotcrete badly cracked. Damp seepage condition.

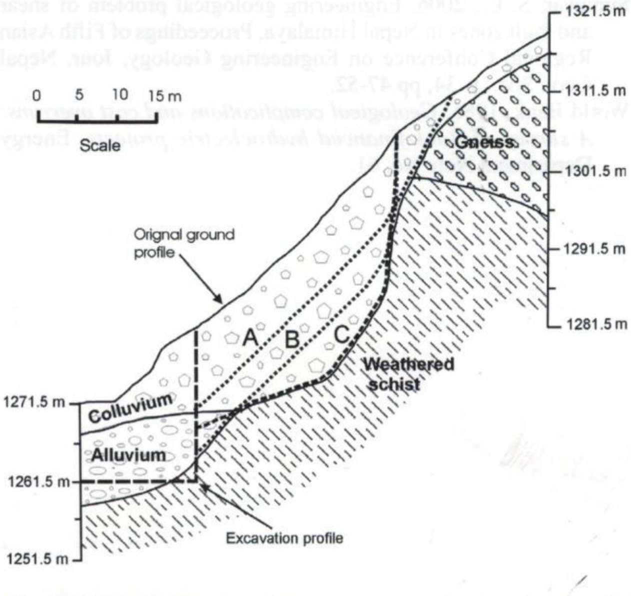

Fig. 4: Cross-section showing geometry, geology and potential failure paths for settling basin back slope (After Pant 1998)

Slope stability

Slope stability problem was encountered in the access tunnel portal and headworks area (Sunuwar 2005). The problem was generated by excavation of tunnel in weathered gneiss and huge cutting in colluvium deposit. Portal of Access tunnel excavated through weathered and fractured gneiss contained clay filling. During tunnelling all loose material was excavated and started in weathered and fractured gneiss. Weathered blocks of gneiss comprise stress relief joints containing clay-filling parallel to valley. Landslide was occurred in 14 July 1994 along joint planes contained clay when tunnel reached at ch. 25 m. Main causes are toe cutting, presence of weathered gneiss and joint containing clay and heavy rain. In addition portal of Adit 1 was blocked by landslide debris and cleared after 3 hours. Similarly landslide at Adit 5 portal destroyed transformer and electric workshop.

In the headworks area steeply standing (35º) colluvium deposit was excavated for settling basin. This steeply standing colluvium deposit was excavated vertically about 10 m down for foundation and 10 m vertical upslope (Fig. 2). Excavation of colluvium made steeper angle (>35º) which angle is not safe. Weathered schist and little seepage were present at toe of the colluvium. After excavation more than 20 m high vertical cliff was formed. The toe of the colluvium was day lighted which was favourable condition for slope failure. Tension cracks were observed on the crown of the colluvium deposit. These conditions collectively threatened the stability of the back slope of the settling basin. A 44 m tall, 6900 m^3 of gabion filling (KSC 2000) was erected for the protection of the settling basin. In addition, the settling basin was covered by reinforced concrete slab in front of this slope. This retaining and cover slab structure increased cost of the project. The overall cost of headworks landslides mitigation works was more than US$ 700,000 (Bisht and Dahal 2004).

CONCLUSIONS

The major engineering geological challenge for any hydropower tunnel is to predict and confirm reliable rock mass condition. The geological uncertainties create great difficulties and can delay the construction schedule which in turn can offset the estimated cost of the project negatively. Therefore, thorough and convincing geological investigation are very essential to identify and predict reliable rock mass condition prior to the actual construction that can minimize project cost significantly. In addition, right choice of construction technology and methodology will certainly reduce geological challenges during construction. The degree of accuracy in predicting geological conditions, evaluation, and interpreting the quality of rock mass during planning phase is a key measure for the successful completion of any hydropower project.

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REFFERENCES

Bisht, K. B and Dahal, G., 2004, Experience of Disaster Management at Khimti I Hydropower Project, Nepal. Abstract, Disaster Management in Nepal, published by Nepal Engineers' Association, pp. 115.

Broch, E. and Sorheim S., 1984, Experiences from the Planning, Construction and Supporting of a Road Tunnel Subjected to Heavy Rockbursting, Rock Mechanics and Rock Engineering, v.17, pp. 15-35.

DoED, 2003, *Guideline for Study of Hydropower Projects*, pp. 40-42, 63-64.

Harris, N. and Whalley, J., 2001, *Mountain Building*, Block 4, The Open University, U.K., pp. 165.

Hoek, E. and Palmeiri, A., 1998, Geotechnical risks on large civil engineering projects, Keynote address, Proceedings of International Association of Engineering Geologists Congress, Vancouver, Canada. pp. 1-11.

Kizaki, K., 1994, *A outline of the Himalayan upheaval: A case study of the Nepal Himalayas*. Published by Japan International Co-operation Agency (JICA), pp. 32-33.

KSC (Khimti Services Consortium), 2000, *Project Completion Report, Khimti I Hydropower Project*, Report submitted to Himal Power Limited, Unpublished report, pp. 11.

Pant, S. R., 1998, *Electrical Resistivity Survey for Headworks Retaining Wall, Khimti I Hydropower Project*, Report submitted to BPC Hydroconsult, Unpublished report, pp. 1-11.

Parker, H. W, 1996, *Geotechnical Investigation, Chapter 4, Tunnel Engineering Hand book*. Second Edition, Edited by Bickel J. O., Kuesel. T. R., and King E. H., Pub. Chapman and Hall, pp 46-77.

Sunuwar S. C., 2002, Squeezing tunnels in Nepal: Case studies, Proceedings of ISRM Regional Symposium on "Advancing Rock Mechanics Frontiers to Meet the Challenges of 21st Century", pp. 165-173.

Sunuwar, S. C., and Fowell, R.J, 2001, Rock squeezing problems experienced in the Khimti Hydropower Project, Nepal. Proceedings of the ISRM Regional Symposium Eurock 2001, Espoo, Finland, 4-7 June 2001, edited by P Sarkka & P. Eloranta. Pub. A.A Balkema, Lisse, Netherlands, pp. 711-716. ISBN 90 2651 821 8.

Sunuwar, S. C., 2005, Slope stability problem in hydropower projects of Nepal: Case studies" Proceedings of International Symposium on Landslide Hazard in Orogenic Zone from the Himalaya to Island Arc in Asia, pp. 361-368.

Sunuwar, S. C., 2006, Engineering geological problem of shear and fault zones in Nepal Himalaya, Proceedings of Fifth Asian Regional Conference on Engineering Geology, Jour. Nepal Geol. Soc., v. 34, pp 47-52.

World Bank, 1985, *Geological complications and cost overruns: A survey of Bank-financed hydroelectric projects*. Energy Department Note No. 61.