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Construction phase engineering geological study in Modi Khola Hydroelectric Project, Parbat district, western Nepal

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ABSTRACT

Detailed engineering geological study is one of the basic requirement for the underground excavation and other infrastructural development works in the geotectonically active part of the Himalaya. Geological investigation directly relates with the sub-surface condition of rock and soil and their engineering properties. Underground excavation and construction of hydropower tunnels, dams, roads, irrigation canal and all foundation works without proper understanding of engineering geological parameters will not meet the requirements to design any safe, efficient and cost-effective infrastructure on and under the earth.

In the Modi Khola Hydroelectric Project, a sound approach on geological condition is considered during construction. This paper deals with the details of the construction phase engineering geological study. It also explains the various geotechnical and geological study done to decide the Tunnel alignments and foundation of various structures in the project. As a result of study, previously designed structures and location of the Surge Tank and Penstock fixed at loose terrace and conglomerate deposit were shifted on the bedrock by designing a 40 m high vertical shaft and about 445 m long pressure tunnel. Previously designed long and curved tunnel alignment from vertical shaft is changed by a straight and better alternative alignment to reduce the tunnel length by about 35m. Similarly, detail geotechnical study of rock in the tunnel have direct impact on support design. Using the maximum advantage of self supporting capacity of rocks, a big amount of cost in tunnel is also reduced by designing the tunnel support system based on Q-value of rocks. All such changes have provided safety to the project and also made more cost-effective.

INTRODUCTION

Modi Khola Hydroelectric Project

The Modi Khola Hydroelectric Project area lies in the Midland Zone of the Lesser Himalaya in western development region of Nepal. The construction site is located in Deupur Village Development Committee of the Parbat district along the right bank of the Modi Khola, between Betini and Patichaur villages at about 40-45 km stretch of Pokhara-Baglung road (Fig. 1). Modi Khola is one of the major tributaries of Kaligandaki River and is originated from the glacier cirque of the Annapurna Range. The catchment area upto the intake site of the project is about 510 km^2.

The main features of the Project includes a 7.5 m high, 33 m long diversion weir (a concrete gravity dam), which diverts the 27.5 cumecs of water of the Modi Khola into a 154.8 m long Desanding Basin through a 30 m intake and 250.29 m long underground Box Culvert. Then an open canal of 63 m length conveys the water to a Regulating Pondage of 26,640 m^3 capacity. Then the water passes into the 1507.01 m long Headrace Tunnel of 3.15 m diameter, 41.25 m long Horizontal Tunnel of 4.24 m diameter, a 37.96 m high Surge Tank of 5 m diameter, a 50.845 m long Vertical Shaft of 4 m diameter, 351 m long Pressure Tunnel of 4 m diameter and 90 m long Penstock Pipe of 3.2 to 2.77

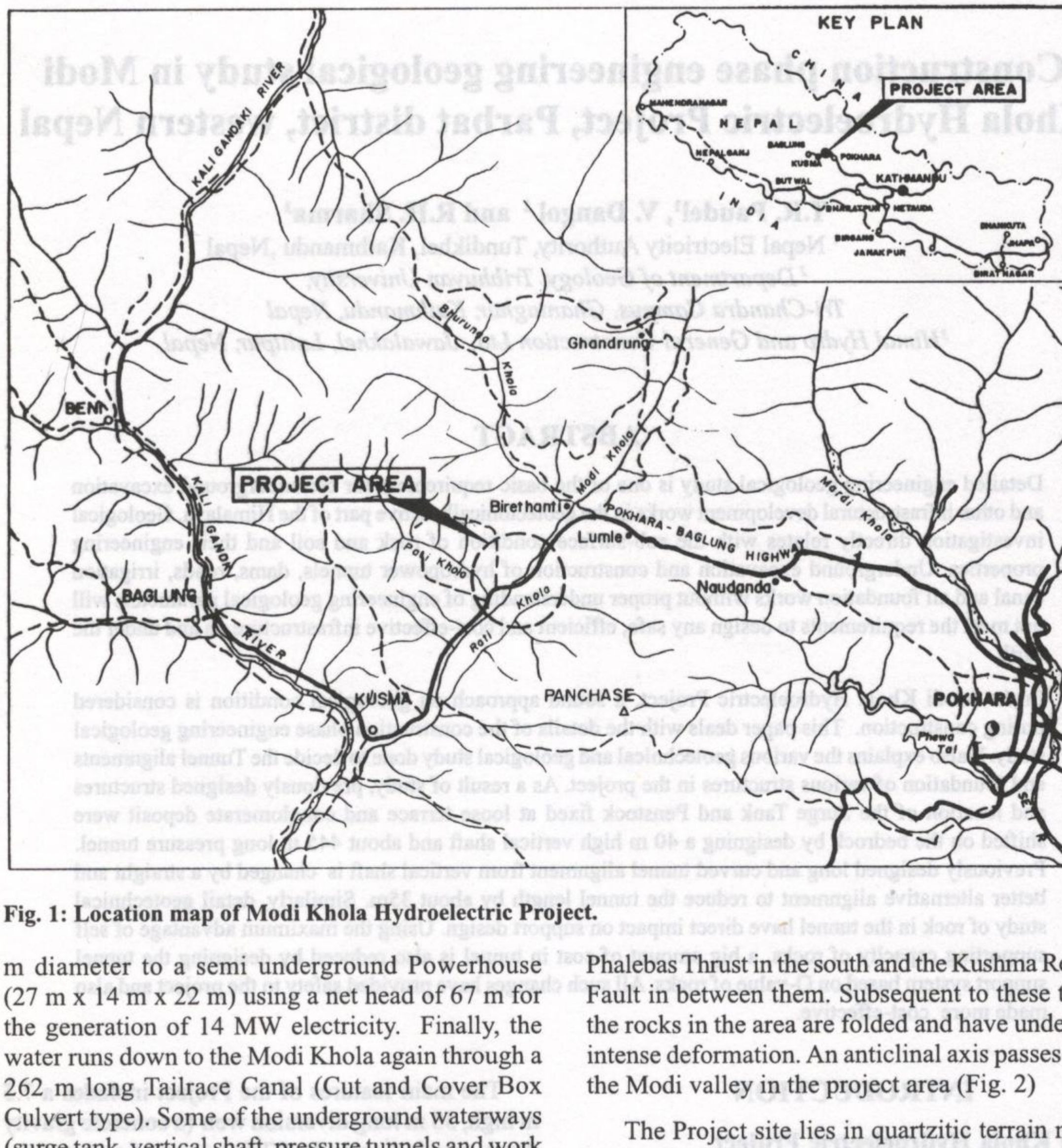

Fig. 1: Location map of Modi Khola Hydroelectric Project.

m diameter to a semi underground Powerhouse (27 m x 14 m x 22 m) using a net head of 67 m for the generation of 14 MW electricity. Finally, the water runs down to the Modi Khola again through a 262 m long Tailrace Canal (Cut and Cover Box Culvert type). Some of the underground waterways (surge tank, vertical shaft, pressure tunnels and work adits) are the modified structures from the original ones, which was required due to geological condition of the site observed during the construction period.

Geology

The area lies in the Lesser Himalayan Midland Zone, covered by thick, monotonous clastic sediments consisting of quartzite, phyllite, schist, metasandstones and phyllitic slates. Structurally, it is bounded by the Lumle Thrust in the north, Phalebas Thrust in the south and the Kushma Reverse Fault in between them. Subsequent to these thrust, the rocks in the area are folded and have undergone intense deformation. An anticlinal axis passes along the Modi valley in the project area (Fig. 2)

The Project site lies in quartzitic terrain named as Naudanda Quartzite (Hirayama et. al, 1981). It is exposed just to the north of the Phalebas Thrust and the Kushma Reverse Fault. It has a sharp contact with the underlying Seti Formation and gradational contact with the overlying Balewa Formation (Paudel and Dhital, 1996). The maximum thickness of the quartzite is about 1000 m in the Modi Khola section. In the lower part of the unit, 2 to 3 bands of medium to coarse grained white quartzite is found, while in middle part of the unit, medium to thick banded, green quartzite alternating with gray green phyllite, green schist, and metasandstone is

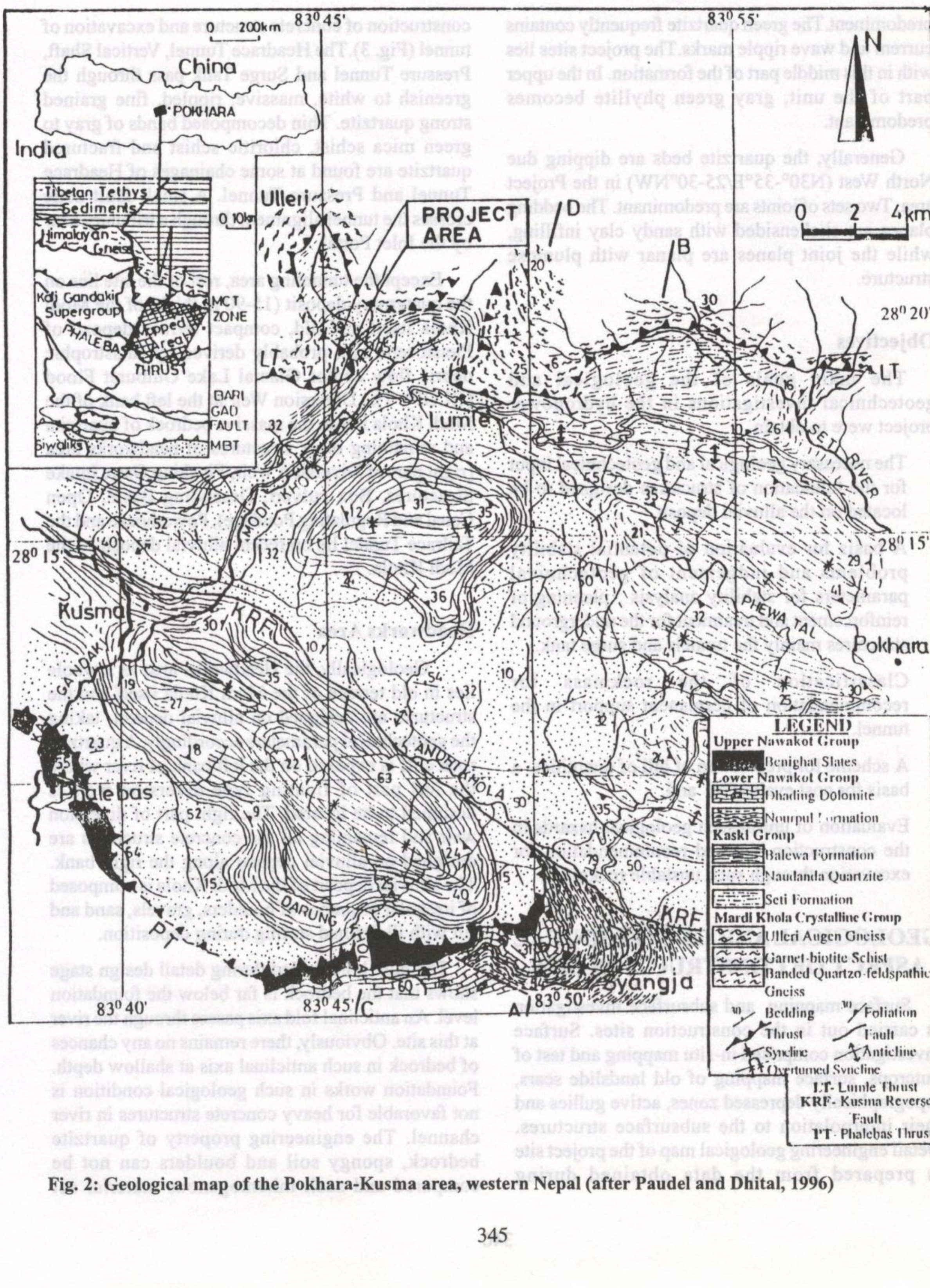

Fig. 2: Geological map of the Pokhara-Kusma area, western Nepal (after Paudel and Dhital, 1996)

predominent.The green quartzite frequently contains current and wave ripple marks.The project sites lies with in this middle part of the formation. In the upper part of the unit, gray green phyllite becomes predominant.

Generally, the quartzite beds are dipping due North West (N30°-35°E/25-30°NW) in the Project area. Two sets of joints are predominant. The bedding planes are slickensided with sandy clay infilling, while the joint planes are planar with plumose structure.

Objectives

The main goals of the geological and geotechnical investigations in the hydropower project were to obtain.

- The necessary geological and geotechnical input for the foundation of structures designed to be located on the alluvial deposit.
- A basis for evaluation of potential stability problems and judgement of geotechnical parameters for stability analysis , planning of reinforcement and treatment for the underground structures mainly the tunnels and surge tank.
- Classification of the rockmass for recommendation of permanent support in the tunnel.
- A scheme for preparation of bill of quantities- a basis for cost evaluation and,
- Evaluation of unforeseen geological hazards in the construction site and recommendation for excavation through such unstable rockmass.

GEOLOGICAL AND GEOTECHNICAL ASPECT OF CONSTRUCTION SITE

Surface mapping and subsurface investigation is carried out in the construction sites. Surface investigation comprised in-situ mapping and test of outcrops, surface mapping of old landslide scars, topographically depressed zones, active gullies and their interpolation to the subsurface structures. Detail engineering geological map of the project site is prepared from the data obtained during construction of concrete structure and excavation of tunnel (Fig. 3).The Headrace Tunnel, Vertical Shaft, Pressure Tunnel and Surge Tank pass through the greenish to white, massive, rippled, fine grained strong quartzite. Thin decomposed bands of gray to green mica schist, chloritic schist and fractured quartzite are found at some chainages of Headrace Tunnel and Pressure Tunnel. A small fault zone crosses the tunnel alignment through a tributary near by the Inlet Portal.

Except the tunneling area, rest of the site lies on the quaternary deposit (15-90 m thick) of the Modi Khola itself and old, compact terrace deposit of Pleistocene age, probably derived by catastrophic debris flow and/or Glacial Lake Outburst Flood (GLOF). The Diversion Weir at the left bank of the Modi Khola lies in the massive bedrock of quartzite and remaining other structures of headworks area (right part of Diversion Weir, Flushing Gate, Intake Structures, Box culvert, Desanding Basin, Open Canal and Regulating Pondage), Powerhouse and the Tailrace Tunnel lie on recent alluvial deposit of the Modi Khola.

Headworks Area

Geologically, the Headworks area as a whole lies in old terrace of the Modi Khola itself and the structures are designed on alluvial deposit, taking the geotechnical parameters accordingly. A quartzite rock cliff is exposed on the left bank of river where the left part of floating type diversion weir is designed to be located. The right part of diversion weir and remaining all the concrete structures are designed on alluvial deposit along the right bank. The alluvial deposit of the Modi Khola is composed of irregular sequence of boulders, gravels, sand and silt with ill defined sorting during deposition.

The bore hole drilled during detail design stage shows that the bedrock is far below the foundation level. An anticlinal fold axis passes through the river at this site. Obviously, there remains no any chances of bedrock in such anticlinal axis at shallow depth. Foundation works in such geological condition is not favorable for heavy concrete structures in river channel. The engineering property of quartzite bedrock, spongy soil and boulders can not be compared and such inhomogenous material act

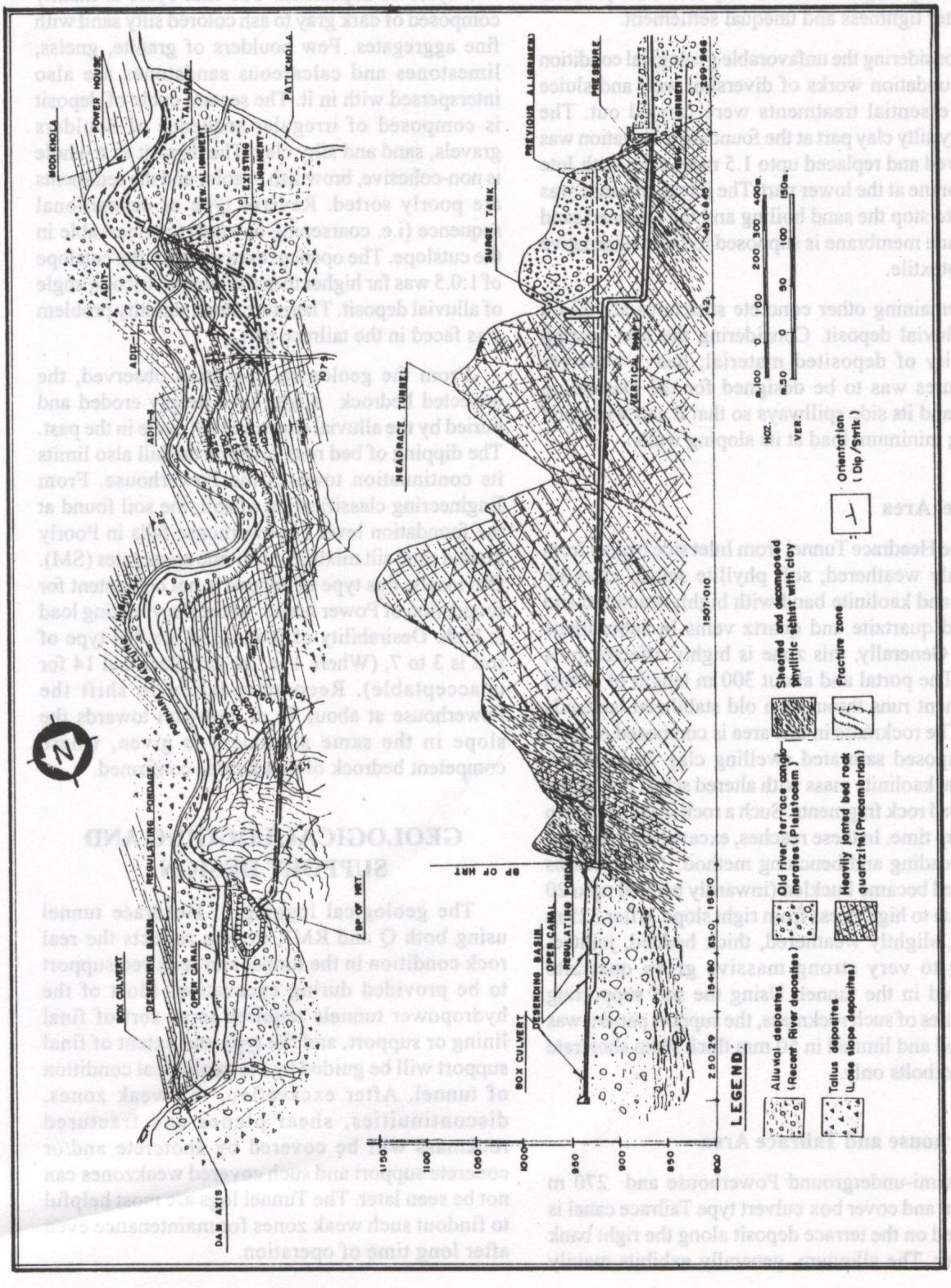

Fig. 3: Engineering geological map of the Project area

differently under stress and may create the problem of water tightness and unequal settlement.

Considering the unfavorable geological condition for foundation works of diversion weir and sluice gate, essential treatments were carried out. The spongy silty clay part at the foundation elevation was removed and replaced upto 1.5 m by sand with Jute membrane at the lower part The jute membrane was used, to stop the sand boiling and the replaced sand with jute membrane is supposed to be an alternative of geotextile.

Remaining other concrete structures are all on the alluvial deposit. Considering the low bearing capacity of deposited material, heavy concrete structures was to be designed for the Desanding basin and its side spillways so that it can withstand giving minimum load at its sloping walls.

Tunnel Area

The Headrace Tunnel from Inlet side begins from a highly weathered, soft phyllite schist, chloritic schist and kaolinite band with highly fractured and brittled quartzite and quartz veins in subordinate order. Generally, this zone is highly affected by a fault. The portal and about 300 m length of tunnel alignment runs through an old stabilized landslide area. The rockmass in this area is composed of fully decomposed saturated swelling clay fault gouge, breccia, kaolinite mass with altered schist layers and fractured rock fragments. Such a rock mass have zero stand up time. In these reaches, excavation was done with heading and benching method. The steel ribs installed became buckled (inwardly bended upto 20 cm) due to high stress from right slope. After 325 m length, slightly weathered, thick bedded, jointed, strong to very strong massive green quartzite appeared in the tunnel. Using the self supporting properties of such rockmass, the support pattern was changed and limited in 50 mm thick plain shotcrete and spotbolts only.

Powerhouse and Tailrace Area

A semi-underground Powerhouse and 270 m long cut and cover box culvert type Tailrace canal is designed on the terrace deposit along the right bank of river. The alluvium, generally exhibits mainly two cycle of deposition. The first cycle is mainly composed of dark gray to ash colored silty sand with fine aggregates. Few boulders of granite, gneiss, limestones and calcareous sandstones are also interspersed with in it. The second cycle of deposit is composed of irregular sequence of boulders gravels, sand and silty clay. The deposit as a whole is non-cohesive, brownish in color and the sediments are poorly sorted. Reverse type of depositional sequence (i.e. coarsening upward) is remarkable in the cutslope. The opencut excavation at the cutslope of 1:0.5 was far higher than the internal friction angle of alluvial deposit. Therefore slope stability problem was faced in the tailrace part.

From the geological condition observed, the expected bedrock might have deeply eroded and buried by the alluvial deposit of 1st cycle in the past. The dipping of bed rock towards the hill also limits its continuation towards the Powerhouse. From Engineering classification of soil, the soil found at the foundation level of powerhouse falls in Poorly graded sand silt mixtures with fine aggregates (SM). Obviously, this type of deposit is not competent for foundation of Power house, where the vibrating load is high. Desirability of foundation for this type of soil is 3 to 7, (Where 1 is for excellent and 14 for unacceptable). Recommendation to shift the Powerhouse at about 40 m upstream towards the slope in the same alignment is given, where competent bedrock of quartzite is confirmed.

GEOLOGICAL LOGGING AND SUPPORT DESIGN

The geological logging of Headrace tunnel using both Q and RMR system reflects the real rock condition in the tunnel and required support to be provided during excavation. Most of the hydropower tunnels requires some sort of final lining or support, and the type and extent of final support will be guided by the geological condition of tunnel. After excavation, the weak zones, discontinuities, shear planes and fractured rockmass will be covered by shotcrete and/or concrete support and such covered weakzones can not be seen later. The Tunnel logs are most helpful to findout such weak zones for maintenance even after long time of operation.

During excavation, geological logging of tunnel was carried out after each blast of Tunnel face; and the support system are based on rockmass quality (Q-value) of rock. The orientation, spacing and characteristics of discontinuities were analysed in detail. The quartzite bedrock is sufficiently strong but the major discontinuities are very prominent and intersecting with one another. Moreover, the Headrace tunnel alignment N 36o E and N 48o E is nearly parallel to the bedding orientation of N30°-35°E/24°-30°NW. Intersection of bedding plane by south dipping prominent joint (N10°E/58°SE) has resulted maximum possibility of block failures along the left wall and crown of Headrace tunnel. The analysis of stereographic projection of bedding plane, joints and shear planes (Fig. 4) shows many unstable condition on crown and walls (Kwon, 1998). The major discontinuities, and their infilling, opening, spacing, persistency, weathering, roughness and alteration of joint surface (Table 1) provide very important clues in making permanent support in the tunnel.

The support system (Fig. 5) primarily guided by rock mass quality (Q-value) of rock calculated during excavation. Further, subjective judgment of evaluating parameters also exist under consideration according to the field condition, purpose of tunnel and safety factor. Minor modification in the support as recommended by Q-system is carried out in the Headrace and Pressure Tunnel. Steel rib support at 1 m to 1.5 m spacing is provided at the major weakzones and in the remaining part the support pattern is primarily the combination of shotcrete and rockbolts where the rockmass appears in Poor to Fair class.The rockbolt spacing and length is determined from the formula,

$$L=2+0.15\ B/ESR,$$

where, B is Tunnel height. The spacing of bolt is 1/2 of the bolt length and the diameter of rockbolt is 20 to 25 mm.

The support pressure (P) is calculated from

$$P = \frac{100\text{-RMR}}{100}\gamma\beta$$

where γ = rock density and β = tunnel height

Table 1: Orientation and characterestics of discontinuities in the Headrace Tunnel.

Tunnel Location	Chainage	Bedding/Joint	Orientation	In filling	Opening	Spacing	Roughness
	99-120	Bedding	N 25°E/35°NW	Clay	-	2-3 m	Slickensided
	120-130	Bedding	N 27°E/37°NW	Clay	-	2-3 m	Planar
	330-400	Joint	125°/60°NE	Sandy clay	-	30-100 cm	Planar
HRT	400-440	Joint	12/70 SE	-	2-3 mm	50-100 cm	Smooth
from	400-440	Bedding	N 45°E/23°NW	Sandy clay	-	50-150 cm	Undulated
Inlet	440-500	Joint	N 45°E/85 SE	-	3-5 mm	100-200 cm	Smooth
	500-600	Joint	200/44 SE	Clay	-	40-100 cm	Planar
	600-660	Bedding	N 31°E/35NW	Clay	-	50-100 cm	Undulated
	660-690	Joint	N-S/62E	Sandy Clay	-	20-50 cm	Planar
	0-200	Bedding	N 30°E/24NW	Sandy Clay	-	100-200 cm	Slickensided
	0-200	Joint	N 20°E/65 SE	-	3-5 mm	40-100 cm	Smooth
Adit - 1	200-360	Bedding	N 27°E/28 NW	Sandy Clay	-	60-100 cm	Slicken sided
Upstream	200-360	Joint	78°E/74°SE	-	2-5 mm	20-60 cm	Rough/Planar
	360-390	Joint	N95/6SW	-	-	100-300 cm	Rough/tight
	390-530	Joint	220/66SE	-	5-1 0 mm	50-100 cm	Rough/Planar
	530-550	Joint	42/72 SE	Sandy Clay	-	30- 100 cm	Rough/Planar
	550-600	Joint	175/66 NE	-	5- 10 mm	50-100 cm	Smooth
	600-660	Joint	340/60NE	Silt + clay	-	40-100 cm	Smooth
	660-705	Joint	90/70°S	-	3- 5 mm	50-150 cm	Smooth
	705-795	Bedding	45°/35NW	Clay	-	200-300 cm	Planar
	705-795	Joint	330/65NE	-	2-6 mm	100-150 cm	Smooth/Planar

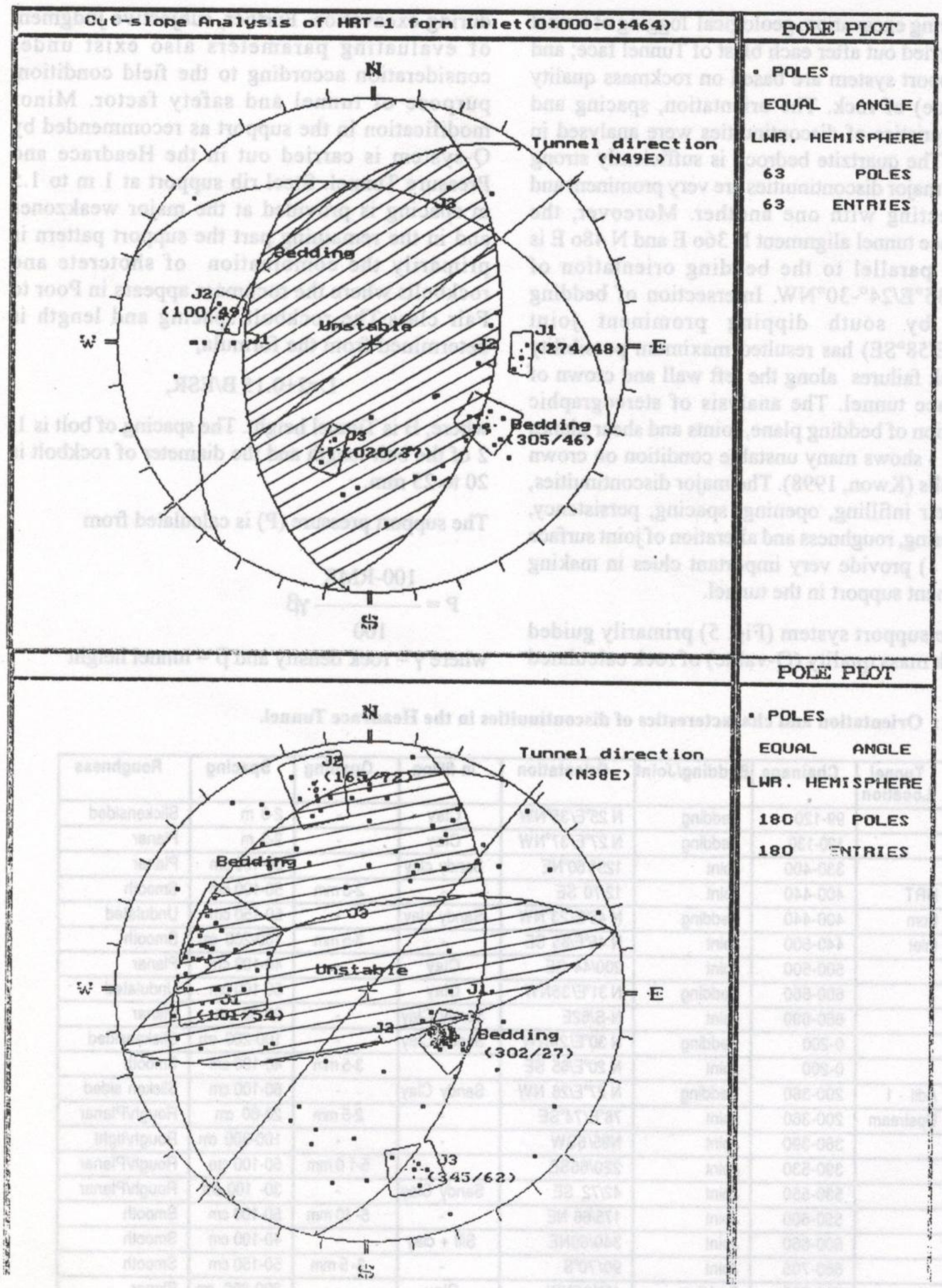

Fig. 4: Stereographic analysis of discontinuities of headrace tunnel (after Kwon, 1997).

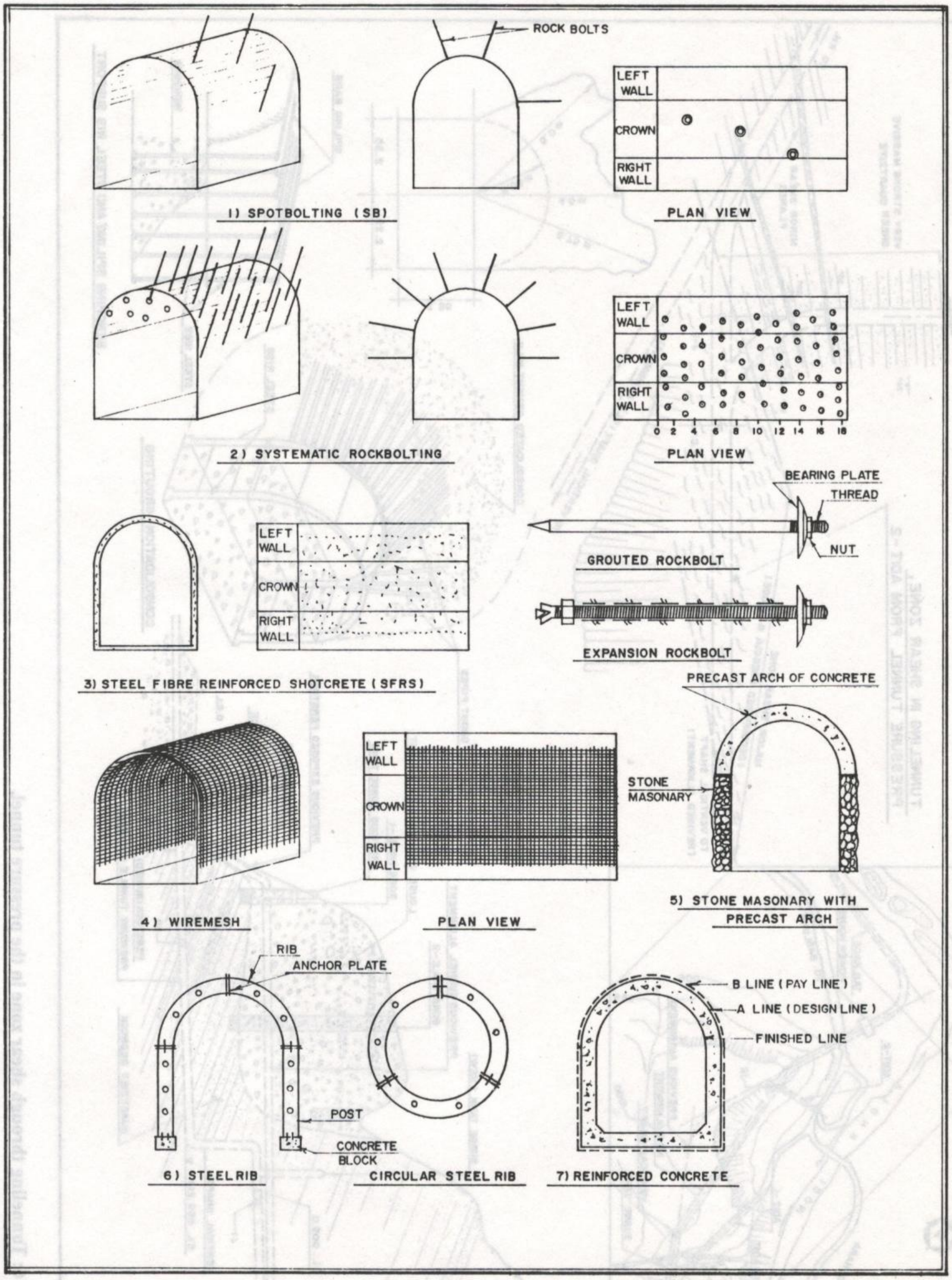

Fig. 5: Principal tunnel support patterns in Modi Khola Hydroelectic Project.

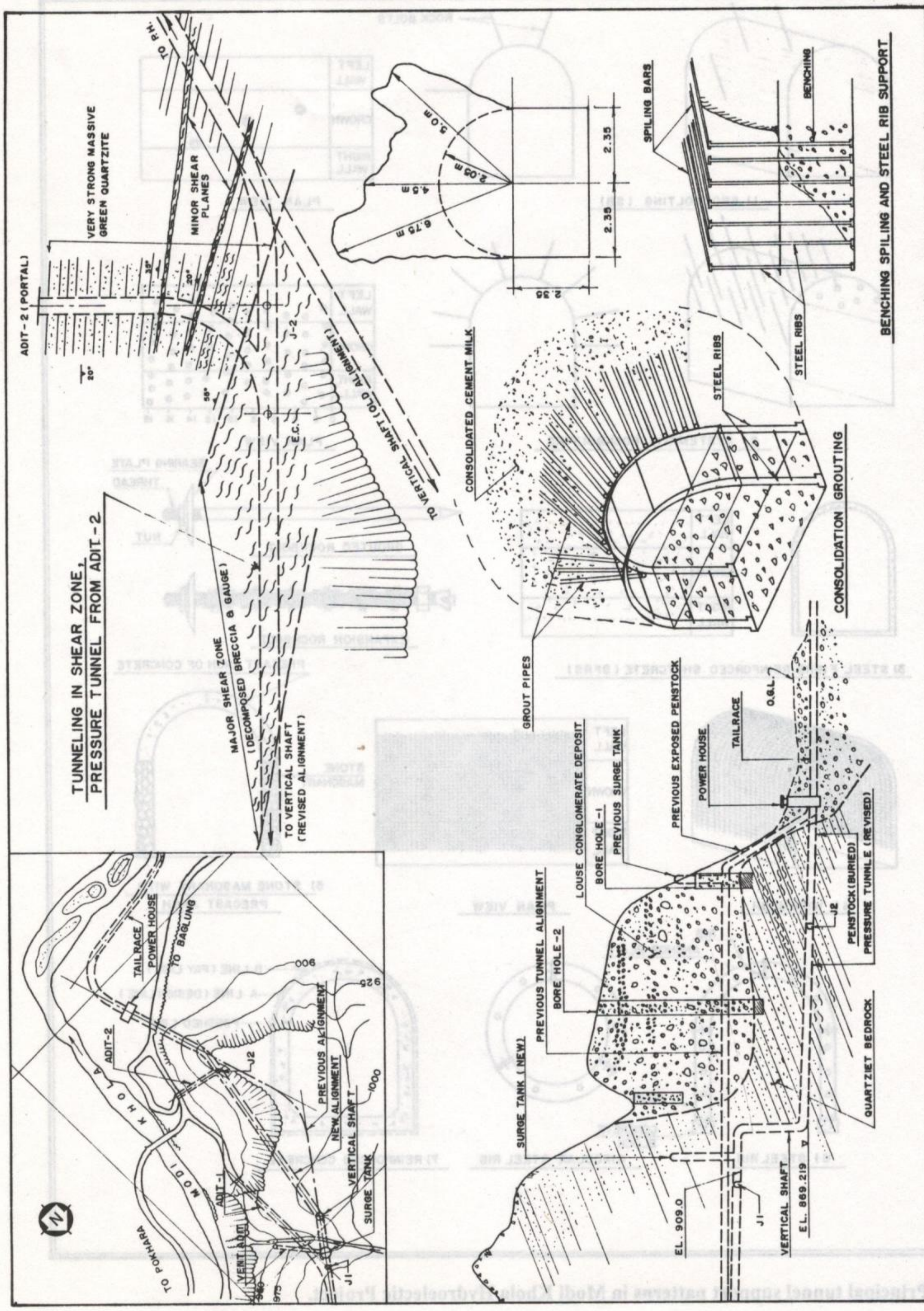

Fig. 6: Tunneling through shear zone in the pressure tunnel.

Permanent support pressure is calculated taking the average value of Joint set number (J_n), Joint roughness and Q value of rock from the following formula (Speers, 1992).

$$P = \frac{2J_n^{1/2} Q^{-1/3}}{3J\gamma} . 98.1 \text{ kPa}$$

Relocation of Surge Tank and Pressure Tunnel

In the original design, the Surge Tank was located at the slope surface just at the unconformity of the quartzite beds and the conglomerate deposit (Fig. 6). The conglomerate is mainly composed of gravel, boulders, pebbles and sand produced from granite, gneiss and quartzite, cemented by calcareous matrix and clay. Most of the part of this conglomerate is hard and compact, but it is loose at its top and often at its depth too. It is normally unsorted and inhomogenous. The bedrock shown in Fig. 6 is no more continuous in the tunnel excavation line due to NW dipping of rock. As a result, about 300 m long Headrace tunnel, upstream from portal was designed to be located through the conglomerate deposit.

Considering the geological problem, two exploratory drill holes were sunk to findout the subsurface condition of rock and thickness of overburden above the bedrock. From these two borehole interpretation, the bedrock line was traced. Consequently it became obvious that the original Surge Tank location and last 300 m of Headrace Tunnel was on the loose terrace conglomerate deposit lying unconformably over the bedrock, which is geologically unfavorable for hydropower tunnel. After many discussions and consultations among experts of Nepal Electricity Authority and foreign hydropower expert, the location of surge tank was shifted on the bedrock about 340 m upstream from its original location (Paudel, 1997). The last 450 m tunnel profile is lowered by about 45 m along the bedrock and designed as Pressure Tunnel introducing a 40 m high vertical shaft. A vent adit of 2.5 m diameter and 68 m length is added for the ventilation of surge tank. About 40 m length of vent adit was designed to be located on the conglomerate deposit. Since the diameter of adit was small, no problem was faced during excavation. Since this adit is to use only for ventilation purpose, there remains no geological problem in future also.

Pressure Tunnel Excavation through the Shear Zone

A 76.5 m long work Adit No. 2 for the excavation of Pressure Tunnel was designed just below the Pokhara-Baglung road. The massive bedrock outcropped at the Portal and the orientation of bedding and joints shows a very favorable condition for tunnel excavation. But, at the end of Adit and further in the Pressure Tunnel, a major shearzone was to cross along the upstream part from junction (Fig. 6). At the junction area, thinly foliated, soft mica schist layers were appeared.The orientation of these weak layer is almost parallel to the tunnel alignment. As a result the weakzone continued for a long distance. On further excavation, a major shear zone composed of fully decomposed soft fault gouge and shattered fault breccia collapsed at upstream face chainage of 26 and 41 m. Since the Pressure Tunnel alignment is 7-10 m below the river bed level, high rate of ground water inflow further made the condition worse.

The slid material was piled up at the face and consolidation grouting was started to stabilize the collapse. The grouting was done by injecting the cement milk at the ratio of 1:1 (cement:water) through 3 to 8 m long perforated GI pipes. Cubes from the collapsed material was made and tested in the laboratory.

After consolidation grouting in the perimeter and above the crown, heading and benching method is followed for further excavation (Fig. 6). Heading by steel arch installation above SPL at 30 to 50 cm spacing and lagging behind with steel bars was done in the first stage. In the second stage full section excavation was done by erecting the post and struts with concrete foundation at 30 to 50 cm spacing.

After excavating 14 m in the shear zone following the heading and benching method; horizontal Probe Hole Drilling was started to find out the width of the shear zone. Core samples of fractured quartzites were obtained from about 18 m onwards from the drilling face so that the width of the shearzone was found to be about 32 m. About 300 bags of cement grouting in the perimeter and spiling bars at 20 cm spacing was required per meter length of tunnel in average. Steel ribs support with 75 to 100 mm thick fibre reinforced shotcrete was used throughout the weak zone.

On the basis of experience from the Modi hydropower, tunneling methods in unstable sqeezing rockmass can be suggested to overcome the problems encountered at other projects in similar situations. Provisions of compressible packing between rock and the support to allow deformation under controlled condition and final lining of the tunnel after sufficient time has elapsed through completion of excavation so that all the movements have died down, is an important consideration in the context of tectonically active zones.

Assessment for Permanent Support in the Tunnel

An assessment considering the rockmass condition for the final lining of Headrace Tunnel is attempted. Recommendation of permanent support by Q-system needs certain modification because of following points which are taken as general principle for modification.

- The Q-system of rockmass classification was developed primarily based on Norwegian experience in their rock type. It is not yet proved in the context of Himalayan rocks so far.
- Most of the parameters of Q-system are to be determined on visual inspection and experience only; so the ultimate results are in wide intervals.
- One of the important limitation of the Q-system is that , it does not consider the tunneling direction relative to the direction of the main discontinuities. The tunnel stability will generally be reduced and the overbreaks increase gradually when the angle between the tunnel axis and predominent joint set becomes smaller than 25-30°.
- The bond of shotcrete with rock surface and in between the two layers is weaker and there will be chances of detachment of shotcrete layers. As a result. regular maintenance of support increases the actual cost even in operation phase.

Considering these situation, the permanent support system for the underground waterways are recommended after modification on the basis of subjective judgement of actual field condition. The Pressure tunnel, Vertical Shaft, Surge Tank and Upper Pressure Tunnel are concrete lined taking the design & hydraulics elements in consideration. In the remaining underground waterways the final support system is recommended as follows.

In the concrete lining sections, the rock bolts and consolidation grouting will be reduced and instead, backfill grouting will be required; while in the shotcrete lining sections, rockbolt number and consolidation grouting will be increased. The rockbolts and shotcrete applied as an initial support will be the part of final support and required number of rockbolts and shotcrete layer should be added in shotcrete lining sections. High water pressure test was carried to decide the grouting pattern and locations

CONCLUSIONS

From the geotechnical investigation during the construction phase of the Modi Khola Hydroelectric Project, few major structures designed to be located in geologically incompetent sites are relocated in safe sites of competent bedrock It also helped to control the cost in support measures by applying the Norwegian technology of tunneling in Himalayan rocks.

Construction phase geological and geotechnical study in hydropower project incorporates the geotechnical parameters taken during design and their deviation in particular construction site. The geological problems in construction site sometime requires certain modification of structures for which a geologist works in close contact with design engineer. The role of geologist in making decisions is very important.

Hydropower development in Himalayan regions requires a high level expertise in geological and geotechnical fields. The geological study in coming years should be confined to particular specific purpose rather than in generalized forms.

In the engineering geological study of a Project area, it is more important to divide the type of rocks