Engineering geological and geotechnical characteristics of the Kankai hydro-power tunnel in soft rock, Nepal

SUNIL KUMAR DWIVEDI[1], PRAKASH CHANDRA ADHIKARY[2] & JAYANDRA MAN TAMRAKAR[3]

[1] *Department of Physics and Earth Sciences, University of the Ryukyus, Okinaw, Japan. (e-mail: k048313@eve.u-ryukyu.ac.jp)*
[2] *Central Department of Geology, Tribhuvan University, Kathmandu, Nepal. (e-mail: tugeology@wlink.com.np)*
[3] *Nepal Electricity Authoriy, Kathmandu, Nepal. (e-mail: jmtamrakar@wlink.com.np)*

Abstract: Nepal, which is located in the Himalayan region of South Asia, is gifted with rich natural resources including huge rivers, and innumerable rivulets criss-crossing the country and, thus in recent years, has become a favourable site for the development of hydro-power. The construction of tunnels through soft rock is rare in Nepal and this paper gives details of the proposed Kankai hydro-power project, located in eastern Nepal. The scheme which will comprise a dam, intake structure, headrace tunnel and semi-underground powerhouse will be constructed in an area of soft sedimentary rocks in a sequence known as the Siwalik Group of Plio-Pleistocene age. The headrace tunnel, which will be located on the left bank of the Kankai River, will be driven through interbedded sandstones, mudstones and siltstones of the Lower Siwalik Group. The area is seismically active due to the presence of two major regional thrust faults, the Main Boundary Thrust (MBT) to the north and the Himalayan Frontal Thrust (HFT) to the south.

Detailed engineering geological and geotechnical investigations were undertaken as part of the feasibility stage of the project the results of which were used to develop a rock mass classification scheme for the project. Boreholes were sunk to obtain rock samples for uniaxial compression tests, triaxial tests, deformability potential tests and for the purpose of undertaking in situ permeability tests. The results indicated the rock formation along the tunnel section to have low shear strength and high deformability potential thus having a rock mass classification of very poor to fair. X-ray diffraction tests were also carried out and confirmed the presence of swelling clay minerals. The results of slope stability analysis of typical slope sections along the tunnel route are also presented in the paper. Finally a guide to excavation methods/techniques and the use of appropriate support systems for the headrace tunnel are presented.

Résumé: Le Népal, qui est situé dans la région de l'Himalaya de l'Asie Du sud, est doué avec les ressources naturelles riches comprenant les fleuves énormes, et les rivulets innombrables entrecroisant le pays et, en conséquence, est devenu un emplacement bien connu pour le développement d'hydro-électricité ces dernières années. L'excavation et la construction des tunnels par la roche molle est rare au Népal. Le projet proposé d'hydro-électricité de Kankai avec le potentiel de l'électricité de 60 MW est situé dans l'ordre de roche sédimentaire doux de Plio-Pléistocène, le groupe de Siwalik, du Népal oriental. Le tunnel de headrace dont est situé à la banque gauche du fleuve de Kankai et sera conduit par les grès, les schistes et les siltstones intercalés du Siwalik inférieur. Le secteur est séismicalement dû actif à la présence de deux défauts régionaux principaux de poussée, la poussée principale de frontière (MBT) dans le bandeau du nord et de l'Himalaya poussé (HFT) dans les sud. Machinant des observations géologiques, des investigations géotechniques et les arrangements de masse de classification de roche ont été employés dans la présente étude. Les essais de compressibilité uniaxiaux, les essais à trois axes, les essais potentiels de déformabilité et les essais de perméabilité étaient insitu effectué aussi bien que des échantillons de roche de forage. L'exposition de résultats la formation de roche le long de la section de tunnel a la basse résistance au cisaillement, le hauts potentiel de déformabilité et très pauvre, pauvre au type juste catégorie de roche. Des essais de diffraction de rayon X ont été effectués pour confirmer la présence du minerai d'argile de gonflement. En outre, l'analyse de stabilité de pente des sections typiques de pente le long de l'itinéraire de tunnel est également présentée dans le papier. Enfin les suggestions pour empêcher de futurs dommages dans le tunnel sont proposées.

Keywords: soft rock, tunnels, geotechnical investigations, rock mechanics, slope stability.

INTRODUCTION

In the recent times, hydropower has become an important part of energy sources. Countries like Japan, United States and other developed countries are looking to rely more and more on hydroelectric energy as the world's fossil fuels are used up. As a result, there has been a huge increase in the number of hydroelectric projects around the world. One of the countries in the world with major water resources is Nepal, which is located in the Himalayan region of South Asia. Nepal is gifted with rich natural resources including huge rivers, and innumerable rivulets criss-crossing the country and thus has become a favourable location for hydropower development. Numerous engineering geological studies relating to roads, irrigation canals and bridges have been undertaken in Nepal (ref: Dhital et al. 1991; Fookes et al., 1985; Deoja, 2000) however, few studies have been carried out with respect to hydroelectric projects, and in particular in soft rock tunnelling (Thapa 1985; Kafle 1996; Paudel et al. 1998). Soft rock tunnelling is not only of interest in Nepal because of its rarity but is also generally of worldwide interest (Goel et al., 1995; Gurung and Iwao, 1998; Wang and Huang, 2002).

Nepal Electricity Authority's (NEA) Kankai hydro-power project, is just one of the underground power generating scheme projects in Nepal. The project area is situated 275km southeast of the capital city of Kathmandu in the Siwalik zone of eastern Nepal (Figure 1). The project is being developed as a dual-purpose scheme to provide both power (60 MW) and an irrigation water supply. The components of the scheme include a dam and intake structure, a headrace tunnel and a semi-underground powerhouse. The layout of the scheme is shown in Figure 2. The upper reservoir is to be constructed by cut and fill at the mountain top and the lower reservoir, which will also serve as an irrigation reservoir, is to be formed by constructing a 70m high clay-core rockfill dam. By 2003, the scheme had passed through the feasibility stage (NEA, 2002), and is awaiting final approval from the Government.

GEOLOGICAL SETTING

Regional geology

The geology of the area in which the scheme lies comprises strata belonging to the Siwalik Group, which in Nepal Himalaya has been divided into three lithostratigraphical units, namely the Lower, Middle and the Upper Siwalik (Schelling and Arita, 1991; Upreti, 1999). The area, which is underlain by thick beds of the molasse sediments of mudstone, sandstone, and conglomerate, has been well mapped by Schelling and Arita (1991). Structurally, the area is bordered to the north by the Main Boundary Thrust (MBT) and to the south by the Himalayan Frontal Thrust (HFT) (Figure 1). The MBT has been the source of very large earthquakes in the past and it is reported that the maximum potential for earthquakes is approximately 8.0 in magnitude. The scheme is located about 15km north of the MBT. The HFT, which lies approximately 1km south of the scheme is also active with the maximum recorded earthquake potential of this fault being around 6.5 in magnitude. It is thus evident that the scheme area is at significant risk from seismic activity.

Site geology

Geological investigation

Geological and geotechnical investigations of the scheme area commenced with a desk study of all existing geological data, followed by air-photo interpretation, surface geological mapping, ground investigation comprising boring and rock core sampling, water pressure tests, laboratory tests, x-ray diffraction tests, and shallow seismic refraction survey. The scheme area is situated in primitive forest, consisting of dense sub-tropical vegetation growing in a thick residual soil mantle. This made it particularly difficult to undertake surface geological mapping and thus collation of geological data was largely restricted to the mapping of a few rock outcrops exposed in the river and stream channels. Boreholes were sunk at various locations, but mostly concentrated in the areas of the major structures such as the dam and intake portal, the headrace tunnel, and the powerhouse (Figure 1). In all, seven boreholes (E/L 1, E/L 2, E/R 1, E/R 2, E/M 1, E/T 1 and E/T 2) totalling approximately 275m were completed. Seismic refraction survey was also undertaken to assist in identifying shallow lithological units and to detect the presence of weak geological features at shallow depth within the scheme area.

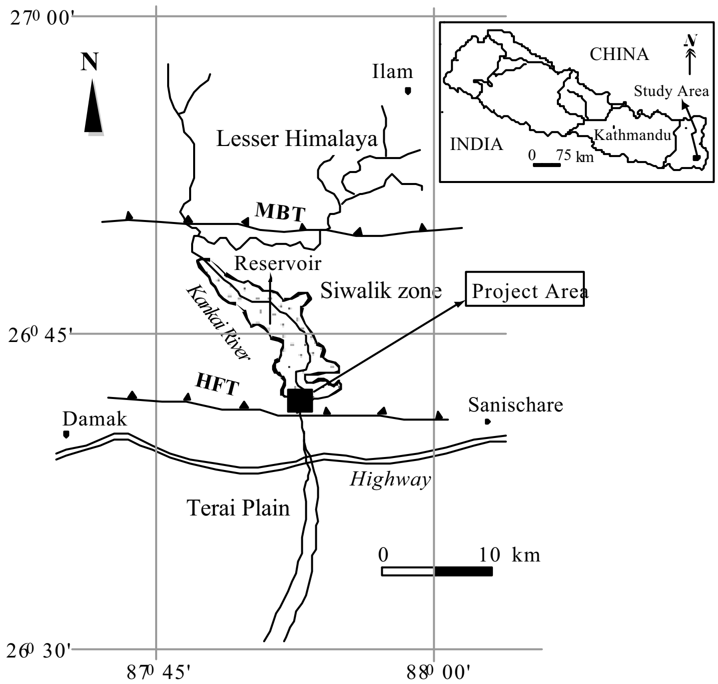


Figure 1. Location map showing tectonic features around the scheme area (modified after Schelling and Arita, 1991)

Ground Conditions

The subsurface in most parts of the scheme area consists of up to 10m of residual soil cover. Below this cover surface geological mapping and borehole data revealed the rock to be predominantly sandstone interbedded with thin mudstone bands and some siltstone intercalations. The data indicates, that the sandstone may be weathered to a depth of up to 30 m, is predominantly grey, fine to medium grained, soft, argillaceous, poorly indurated and highly jointed. A fining upward cycle and ripple marks were observed in the sandstone beds together with frequent intercalations of calcareous siltstones. The sandstone is of low to medium strength. Seismic refraction survey showed compressional wave velocity of the fresh sandstone in order of 2500– 5500 m/sec. Thinly laminated mudstone beds are frequent within the sandstone and show evidence of bio-turbation, variegation, and poor induration.

Discontinuity Data

The sandstone, which is generally massive but fissile, is seen throughout the area. The fissility has the same direction as that of the bedding, i.e., strike in NNW–SSE direction and dip up to 80^0 NW to NE direction. In the project area the strata strike NE-SW ($034^0 – 048^0$), i.e. parallel to the dam axis and dip 46^0 to 58^0 towards NW into the reservoir. Both, strike and dip of the beds are favorable for dam construction. In the intake area of the headrace tunnel, the strike changes to a NW-SE (100^0-130^0) direction. Approximately 400m south of the proposed dam axis the strata strike NW-SE and dip about 20^0 to 36^0 to the NE. At a few locations along the left riverbank however, the beds were recorded as dipping towards the SW, thereby indicating slight folding of the strata in this area. About 100 m upstream of the irrigation weir the trend of the strata is changing abruptly to NE-SW (about 045^0). The beds are oriented – perpendicular to the river and dip 40^0 to 60^0 towards NW. The predominant sets of discontinuities are as follows:

Left abutment of dam:

- Set 1: Strike $042^0 – 063^0$ / $47^0 – 58^0$ NW
- Set 2: Strike $129^0 – 142^0$ / $48^0 – 70^0$ SW
- Set 3: Strike $168^0 – 022^0$ / $27^0 – 43^0$ E
- Set 4: Strike $156^0 – 002^0$ / 80^0 E – 90^0 – 82^0 W
- Set 5: Strike $104^0 – 116^0$ / $55^0 – 83^0$ SW

Right abutment:

- Set 1: Strike $034^0 – 048^0$ / $46^0 – 58^0$ NW
- Set 2: Strike $142^0 – 160^0$ / $35^0 – 58^0$ NE
- Set 3: Strike $138^0 – 156^0$ /$53^0 – 66^0$ SW

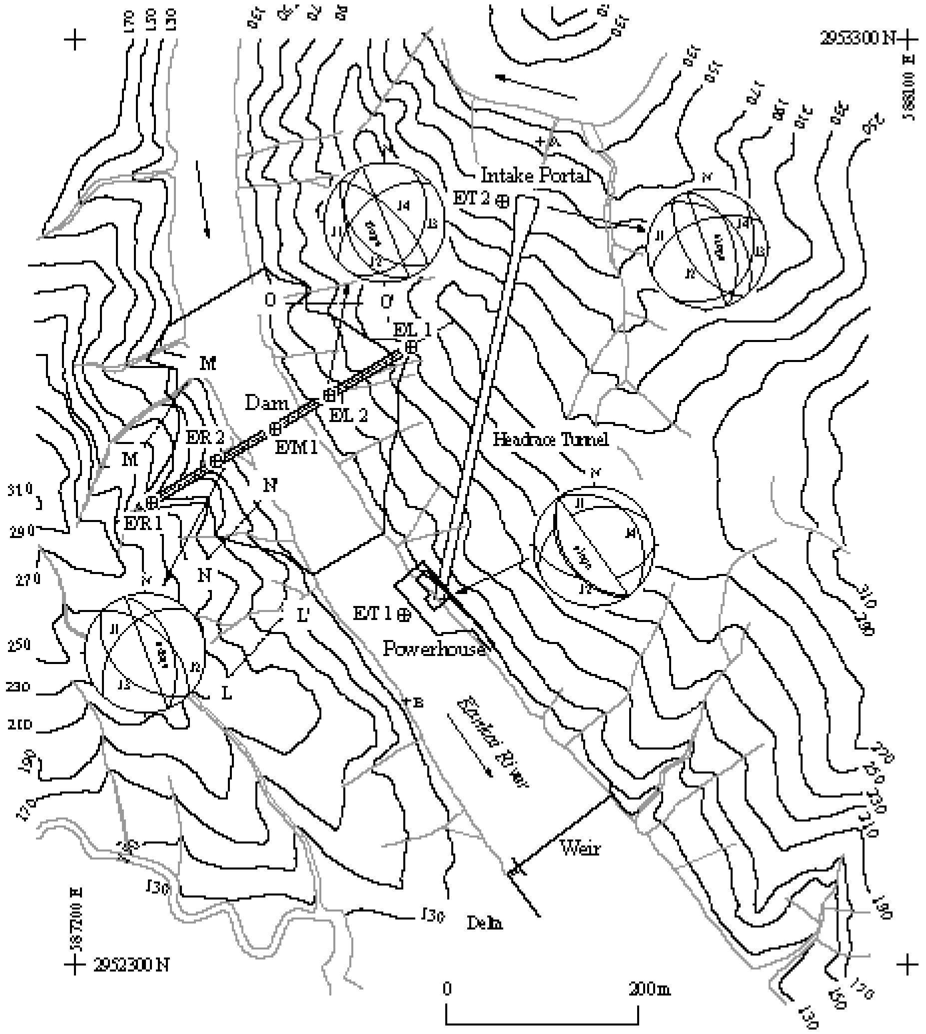


Figure 2. General layout of the Kankai hydro-power scheme showing discontinuity data at locations of structures

Discontinuity set 1 is the most predominant. Its orientation is the same as that of the bedding. The strike of discontinuity set 2 is approximately perpendicular to the strata, i.e. parallel to the river. The sandstone mass was revealed by the boreholes to be generally massive with moderately to widely spaced discontinuities. The majority of discontinuities are orientated in a NW–SE direction. Discontinuity spacing is typically between 100 and 1200mm and varies between 1 and 3 m in length. The discontinuities are for the most steeply dipping typically between 40^0 and 75^0 and the surfaces were recorded as being typically planar and slightly rough to smooth. Discontinuity opening was noted as being generally tight at depth. In order to assess the feasibility and ultimately the stability of rock cut slopes that will be required as part of the scheme the discontinuity data has been analysed and processed utilizing a computer based program called "DIPS" (Diederichs & Hoek, 1989), which allows for the visualization and determination of the kinematic feasibility of rock slopes.

Groundwater

Investigations and testing were undertaken to assist in determining the nature of the groundwater regime and hydrogeological properties of the rock mass in the scheme area. Tests included water pressure tests in boreholes and water log tests. Unfortunately, however, the investigations and testing were inconclusive with respect to determining the presence of an established groundwater level within the rock mass. Drilling operations revealed considerable variations in the groundwater level, which is generally typical of a jointed rock aquifer. According to readings taken from the boreholes, the groundwater table varied from 8.8m, to 18.0m, to 33.50m, to 51.6m below the ground surface in boreholes E/L1, E/L2, E/R1 and E/R2 respectively. The measurements indicate that the groundwater at the right abutment is deeper below the ground surface than at the left abutment. Testing within the boreholes showed variability in the permeability from 1 to 48 Lugeons in the uppermost 30m of the boreholes. Below 30m, the test results indicated the rock mass was mostly impervious, with Lugeon values typically between 0–5 being measured (Figure 3). In a few places however, higher Lugeon values were recorded, believed to represent localised zones of fractured rock at depth. It should be noted however, that permeability measurements obtained from water pressure tests are strongly influenced by the occurrence of individual discontinuities, which may alter the degree of openness of the discontinuities and habit in the lateral extension.

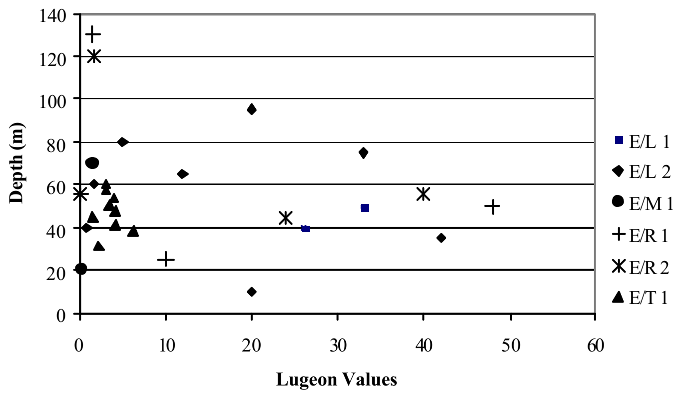


Figure 3. Lugeon values for the Kankai hydro-power project

INTACT ROCK PROPERTIES

In order to determine rock mass properties for the purpose of designing the various scheme elements the intact rock properties were firstly evaluated. A series of laboratory tests were conducted on surface rock samples as well as rock core samples obtained from boreholes drilled at various locations and to various depths. Rock cores 3cm in diameter and 3cm in length were prepared from the two types of rock encountered within the scheme area namely (a) sandstone, and (b) sandstone interlaminated with mudstone.

Physical properties

In order to determine the mineral composition, the texture and the fabric of the rocks, thin sections were initially prepared and x-ray analysis undertaken (Table 1). Mineralogically, both rock types consist primarily of feldspar (35–45%) and quartz (25–35%) with a lesser content of mica, chlorite, calcite, kaolinite and some opaque minerals. The sandstone interlaminated with mudstone contains about 5% clay minerals (chiefly montmorillonite). The rocks have medium density, high porosity and high absorption. An impregnation of the rock by iron-hydroxide could be observed in the thin sections and porosity of 10-20% has been determined. The discontinuities were often filled by calcite. Classification wise the rocks can be classified as a silty to clayey, sometimes calcareous fine to medium grained sandstone with a montmorillonite content generally up to 5%. Both rock types have similar density, i.e., 24.4 kN/m^3 and 24.3 kN/m^3 for the sandstone and the sandstone interlaminated with mudstone, respectively. The absorption of the former is between 28.8% and for the latter is 42.7%. The former has low porosity, typically in the order of 2.0, compared with 2.7 of the latter. Due to the maximum water absorption it is assumed that the swelling montmorillonite or mixed- layer minerals content is lower than 5%. This assumption is proved by the results of x-ray analysis.

Table 1. Results of x-ray analysis

Sample	Major components > 20%	Minor components 5 - 20%	Traces < 5%	Classification
E/L1-1	Q	F, K, Mu, Il	Py, Mo, Chl, Si, Ok	Sandstone, fine grained, silty
E/L1-2	Q	Ca, Mu, F, K, Chl	Do, Si, Mo, Py, Gi	Siltstone calcareous
E/L1-3	Q	Ca, Mu, F, Chl, K	Si, Do, Mo, Gi, Py	Sandstone, calcareous
E/L1-4	Q	Ca, Il, F, Chl, K	Mx, Si	Siltstone, clayey
E/M1-5	Q, Ca	K, F, Chl	Chl, Do	Sandstone, fine-grained silty
E/M1-8	Q, F	K, Mu	Chl, Do	Sandstone
E/M1-18	Q, F	Mu, Mo	K, Ca, Chl	Sandstone, calcareous
E/R1-25	Q, F	Mu, K	Chl, Ca, Do	Sandstone, silty
E/R1-19	Q, F	Mu, K, Ca	Chl, Mx	Sandstone, calcareous
E/R1-29	Q, F	K, F	Pyr, He, Mx, Mo	Mudstone
E/R1-1	Q, Mu	Chl, K, F	Goe, Mx	Mudstone
E/R1-9	Q, Mu	F, Chl, Mo	Ca	Mudstone, calcareous
E/R2-1	Q, Mu	F, Mu, Chl, K	Mo	Mudstone, calcareous
E/R2-2	Ca	Ca, Mu, F, K	Mo, Do	Mudstone, calcareous

Abbreviations: Q-Quartz, Mu-Muscovite, F-Feldspar, Chl-Chlorite, Mo-Montmorillonite, K-Kaolinite, Mx-Mixed layer mineral, Ca-Calcite, Mi-Mica phase (probably Muscovite-Illite), Py-Pyroxene, Si-Siderite, Ok-Okenite, Gi-Gibbsite, Do-Dolomite, Il-illite, Pyr-Pyrite, He-Hematite, Goe-Goethite

Rock strength

Uniaxial Compression Tests

The field observations were confirmed by laboratory test results. Uniaxial compression tests were performed on 21 rock core samples. The unconfined compression strength of the dry sandstone from the scheme area ranges between 6.8 and 38.9 MPa although these values were noted to reduce considerably in the wet condition. For the sandstone interlaminated with mudstone the unconfined compression strength was measured as varying between 11.4 MPa and 53 MPa, but more typically varied between 25 and 35 MPa. From calcareous sandstone samples unconfined compression strength values between 22.1 and 76.2 MPa were obtained. In summary, the average compressive strength of the sandstone (including calcareous samples) is approximately 30.6 MPa, whilst for the sandstone interlaminated with mudstone the average is 25.2 MPa (Figure 4). Accordingly, the sandstone and sandstone interlaminated with mudstone in the scheme area can be classified as medium and low to very low strength rocks, respectively. Failure modes during the unconfined compressive strength tests were both axial splitting and fracturing along the steeply inclined bedding planes or calcite/quartz veins.

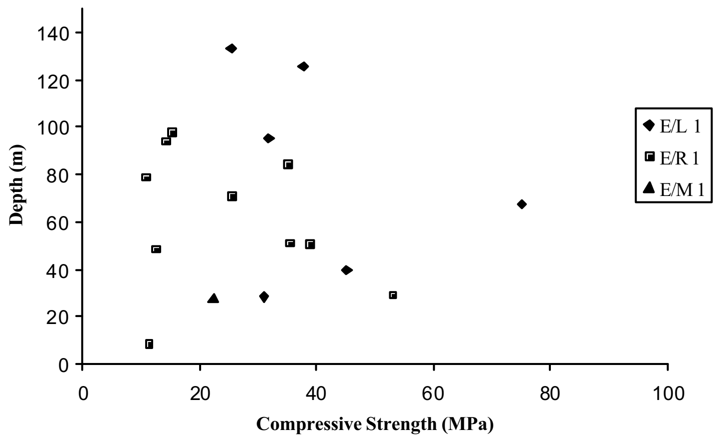


Figure 4. Uniaxial compressive strength of intact rocks in the scheme area (see Table 1 for sample descriptions)

Triaxial Tests

Triaxial compression tests were undertaken on core samples of dry sandstone and sandstone interlaminated with mudstone. Twelve sets of tests were carried out at confining pressures ranging from 1 MPa to approximately 50% of the uniaxial compressive strength. The results of the tests, which are summarized in Figure 5 show the triaxial strength envelopes of the sandstones to be non-linear. Mohr circle plots of the test results indicate an angle of internal friction for sandstone of 50^0 and 40^0 for the sandstone interlaminated with mudstone. The cohesion was measured as 2 MPa and 9.5 MPa for the sandstone and the sandstone interlaminated with mudstone respectively.

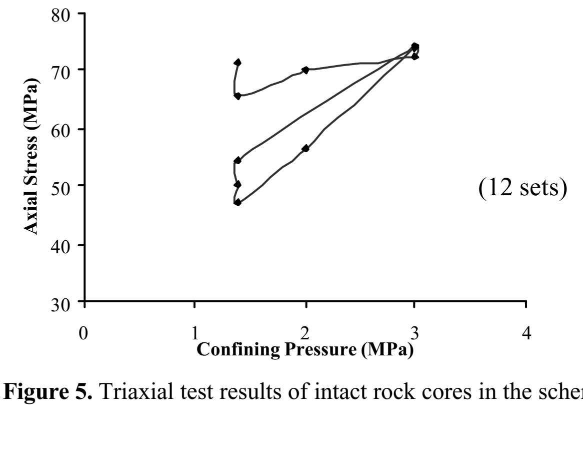


Figure 5. Triaxial test results of intact rock cores in the scheme area

Shear Tests

Both the uniaxial compression and triaxial tests as discussed above were undertaken on core samples of dry sandstone devoid of separating planes ie. discontinuities, bedding planes etc. Therefore, values of geotechnical parameters obtained from these tests are likely to be relatively high and not necessarily a true representation of the properties of the in situ rock mass which has been shown to be both wet and moderately to widely jointed. The geotechnical parameters of jointed and wet rock masses would normally be expected to be lower than those of dry rock, particularly the shear strength which is significantly influenced by the presence of discontinuities and bedding.

Therefore, uniaxial shear tests were undertaken on samples of rock believed to be representative of the rock mass as a whole. The results of the tests, which are plotted in Figure 6, indicate that the mean values for the cohesion and angle of internal friction of the sandstone are 0.5 MPa and 25° respectively and for the sandstone interlaminated with mudstone are 0.74 MPa and 23° respectively. As these values of cohesion and angle of internal friction are considered to be most representative of the in situ properties of the rock mass within the scheme area they have been used in slope stability calculations.

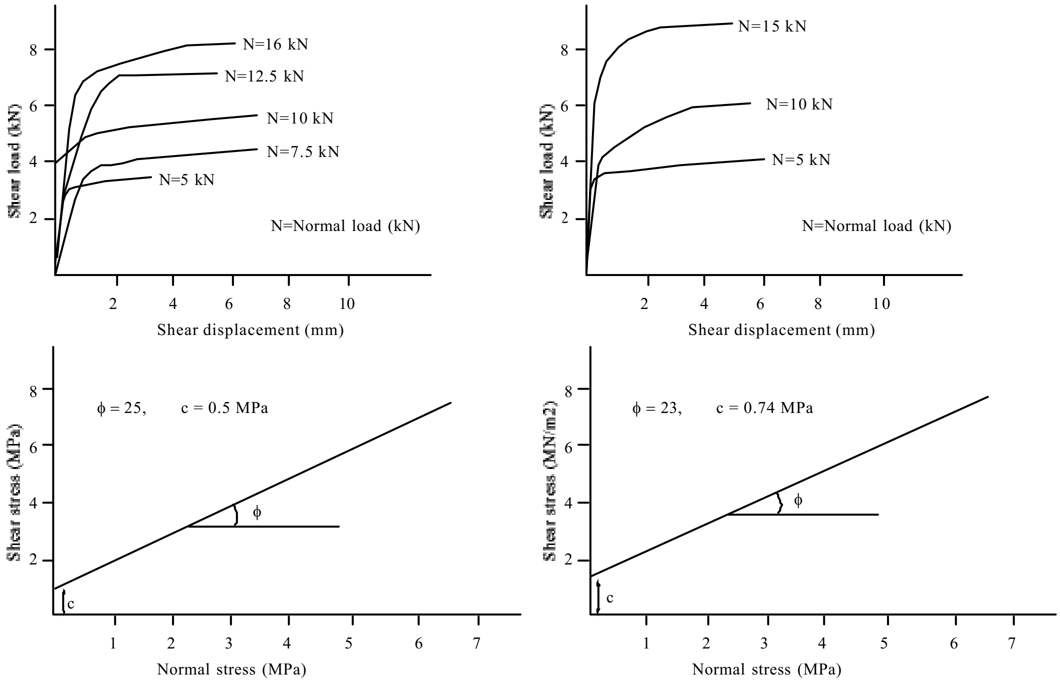


Figure 6. Shear test results of intact rock cores in the scheme area

Deformation Tests

To assess the deformation characteristics of the rock within the scheme area borehole dilatometer tests were conducted. A total of two boreholes were sunk for the tests, namely borehole E/T 1 which was drilled to 15.50m at an angle of 60° and borehole E/T 2 which was drilled at an angle of 70° and to a depth of 25m. Four dilatometer tests were conducted in each borehole.

The results of the tests have been used to calculate both the deformation and elasticity moduli of the in situ rock. The Ev_1 moduli were calculated by considering the total deformation measured for each applied pressure. The Ev_2 moduli were calculated from the deformation measurements for the second-load. The elasticity moduli were determined by measuring the resulting deformation caused by the release of stress after each applied pressure. As can be seen from the graphs in Figure 7 the deformation behaviour of the rock can be judged directly from the data plots. The average moduli of deformation determined by the dilatometer tests for sandstone were 330, 450 and 580 MPa; and for sandstone interlaminated with mudstone were 290, 470 and 780.MPa. As all the tests in the boreholes were carried out below the water table it must be borne in mind that the deformation and elasticity moduli are likely to be higher in the dry rock.

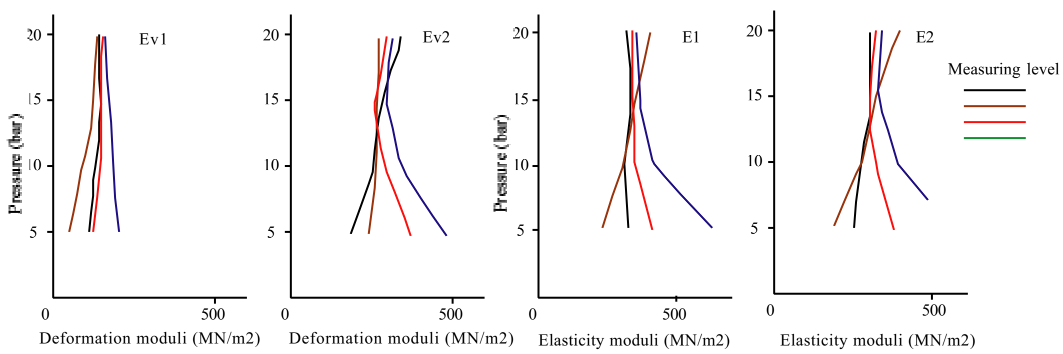


Figure 7. Modulus of deformation and modulus of elasticity in the scheme area

The modulus/strength ratios, E/sigma c of the two rock types as discussed above are 248 and 281, respectively, which both fall in the typical range for homogeneous rock of 200 to 500.

The Poisson's ratio of the rocks ranged from 0.55 to 0.93.

The P-wave velocity of intact rock specimens from ultrasonic tests was in the order of 1627 to 4761 m/sec.

SLOPE STABILITY

As detailed earlier the scheme area is covered with a dense sub tropical vegetation which when considered in the context of slope stability is assumed to have a stabilizing influence on the slopes surrounding the reservoir area. The impounding of the reservoir will result in the removal of much of this vegetation and thus, to avoid both, slope failure by sliding on joints/bedding planes and progressive erosion of the sandstone/mudstone layers by the impounded water, protective measures will be necessary. In order to assess the stability of the slopes in the area of the scheme, slope stability analyses of typical slopes sections were undertaken. As far as possible, specific structural and rock mechanical data collated during site mapping, investigations and testing have been considered, but locally differing orientation of joint sets, joint spacing, degree of separation and habit of joints have been neglected. As detailed earlier in the section on discontinuities, analyses of rock discontinuity data has revealed that the features that will dominate slope stability at the right abutment of the reservoir/dam will be the bedding (J1) and the NE dipping joints (J2) whereas on the left abutment the bedding (J1) and the SW dipping discontinuities (J2) are the dominating features. Stability analyses identify potential sliding planes, which are ultimately related to the orientation of the joint/discontinuity sets. The factor of safety derived from the analyses is the quotient of the resisting and acting shear stresses along sliding planes. The stability of the slopes around the dam/reservoir site has been assessed using simplified rock mechanical assumptions. For those parts, which will be flooded by the reservoir, porewater pressures were introduced into the calculations. The factor of safety obtained for the sections L-L', M-M', N-N', and O-O' (see Figure 2) as set out in Figure 8 varies between 4.2 and 14.1, indicating that the slopes will stable after construction of the reservoir and impoundment of the water.

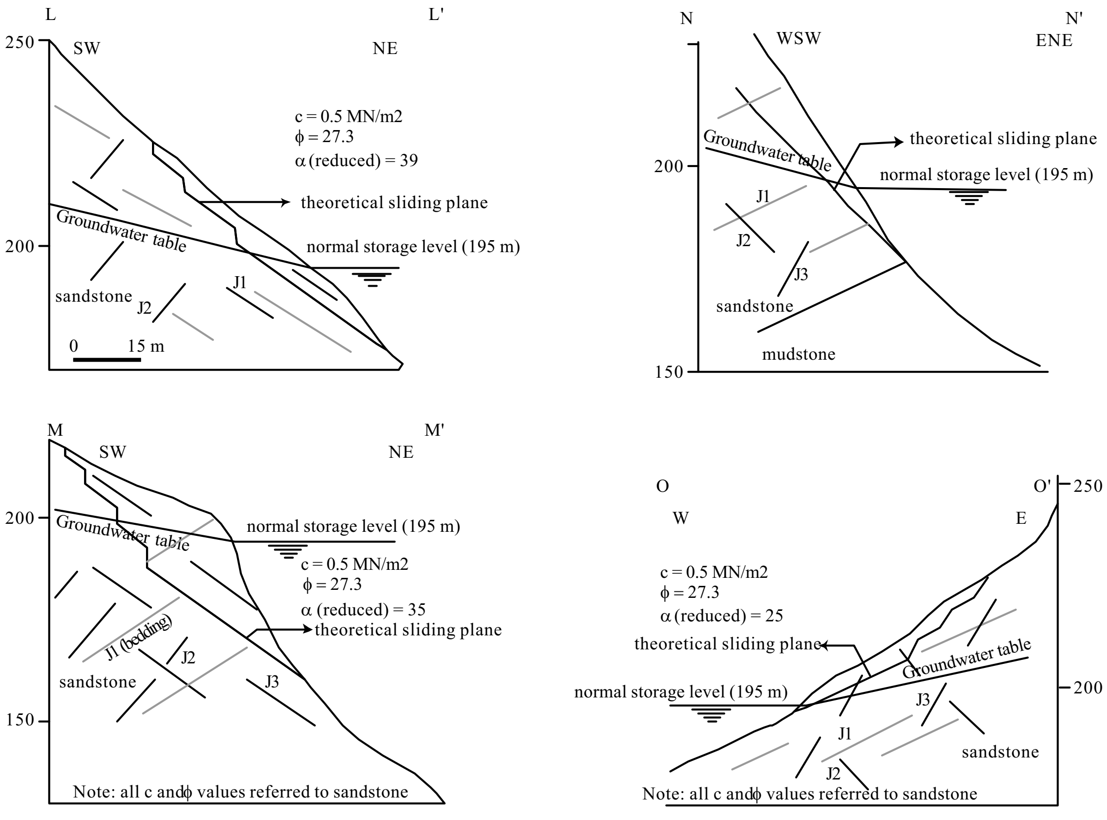


Figure 8. Slope stability analysis at the left and right abutments of the dam/reservoir site

GEOLOGICAL AND GEOTECHNICAL CONDITIONS ALONG THE HEADRACE TUNNEL

The proposed horseshoe-shaped headrace tunnel is approximately 320m long and 8.5m in diameter and passes through the rock mass at a mean inclination of around 6º. The maximum thickness of rock cover above the tunnel route is approximately 90m. To accommodate the intake structure an open cut around 30 m long will be required. The slopes in the area of this cut will be formed in sandstones, which dip approximately parallel to the slope. A semi-underground powerhouse will be located close to the downstream toe of the dam. A deep excavation in river deposits will be required for the foundation of the powerhouse as data obtained from borehole E/M 1 indicates that bedrock can be expected at a depth of approximately 21m. The geology along the length of headrace tunnel, which is detailed in

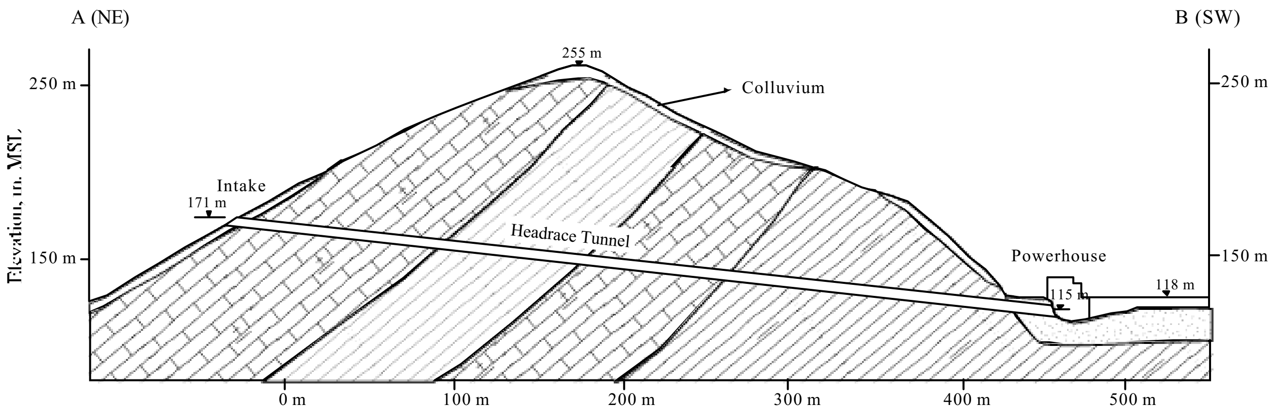
Figure 9, is based primarily on mapping of the outcrops at the surface, particularly along the left bank of the Kankai River. The strike of the rocks is oriented NE–SW along most of the tunnel length with a slight change to E–W in the northern section of the tunnel.

Generally, the stability of a tunnel is reduced and the overbreak increases gradually when the angle between the tunnel axis and the predominant joint sets becomes smaller, therefore, the length axis of the tunnel and cavern openings at shallow to intermediate depths need to be oriented along the bisection line of the maximum intersection angle between the predominant joint directions. For the headrace tunnel on this scheme, a rose diagram compiled from a total of 509 discontinuity measurements, indicates that the orientation of the tunnel axis ie. 193^0, is favourable for tunnel stability. The angle of intersection between the tunnel axis and the bedding varies from 100–300 at the southern end of the tunnel to 200–300 at the northern end. The dip of the strata varies between 40^0 and 60^0 NW. At either end of the tunnel the rock mass is expected to be slightly to moderately weathered and disintegrated for a distance of 50 to 60 m from the portals. The tunnel will be located below the groundwater table, and therefore minor seepages from the roof and the walls can be expected. Measured rock parameters as detailed in previous sections indicate that for most of the tunnel length the quality of the rock will be poor.

ROCK MASS PROPERTIES

Rock mass classification

The collation of engineering data as discussed in the preceding paragraphs has been used to assess the quality of the rock mass within the scheme area under a rock mass classification system. Two rock mass classification systems have been considered in this assessment, namely, the Geomechanics Classification System (RMR) by Bieniawski (1989), and the NGI (Norwegian Technical Institute) Tunneling Quality Index (Q) System by Barton, 1974. These two systems are well known and widely used in rock mass classification. In applying a rock mass classification the RMR system takes into account the uniaxial compressive strength of the intact rock, the rock quality designation (RQD), the joint spacing, the joint condition, the joint orientation and the ground water conditions. The Q system however, determines rock mass quality by considering the joint sets (Jn), roughness of the discontinuities (Jr), joint alteration (Ja), water pressure (Jw), the stress reduction factor (SRF) and the RQD. Classification of the rock mass along the length of the headrace tunnel in accordance with these two classification systems is shown in Figure 9. Although numerous rock core samples had been obtained from boreholes, the persistence of discontinuities in the rock mass could not be ascertained from these cores and thus the bulk of the data used in the classification was obtained from visual inspections of the geological conditions in the scheme area, which concentrated primarily on discontinuity characteristics and degree of weathering. Classification of the rock mass was undertaken at 10 m intervals along the length of the headrace tunnel taking the intake portal as point zero. Based on the ratings given by the RMR and Q systems, which are, fair rock (RMR = 46–60, Q = 4–4.5), poor rock (RMR = 30–40, Q = 1.4–3.4) and very poor rock (RMR = 18–20, Q = 0.35–0.58), about 12% of the tunnel will cross fair rock, 69% will cross poor rock, and 19% will cross very poor rock. It should be borne in mind however, that the rock mass classification has been based largely on data obtained from surface rock outcrops that have undergone weathering and thus are expected to be of much lower quality than fresh rock at depth. The RMR rating for rock cores obtained at depth within the scheme area was mainly in the good to fair class. Poor to very poor rock may be expected at a few locations only.



Lithology	mainly sandstone, thick bedded to massive	mainly sandstone, sandstone intercalated with mudstone	mainly sandstone intercalated with mudstone, some siltstone
Strike/ dip	strike NE-SW, locally changing to E-W, dip to NW-N		Strike NE-SW, dip to the NW
Rock Class (By RMR & Q system)	class IV to II (poor to fair rock)	class IV to II (poor to fair rock)	class IV to V (poor to very poor rock)

Figure 9. Rock mass classification along the headrace tunnel

Support design

One of the reasons for assessing the rock with respect to a rock mass classification system is to assist in identifying problems with stability such that appropriate support systems can be designed and specified. As the rock mass along the tunnel section has been classified in accordance with the RMR and Q systems, the excavation and support methods required are therefore readily identifiable and are defined by both these systems. A guide to excavation methods/techniques and the appropriate support systems for the headrace tunnel is presented in Table 2.

Table 2. Summary of support recommendations for the headrace tunnel section

<table>
<tr><th rowspan="2">Tunnel section (m)</th><th rowspan="2">Lithology</th><th rowspan="2">Rock Category</th><th colspan="2">Support recommendations</th><th rowspan="2">Guide for excavation (RMR)</th></tr>
<tr><th>Q</th><th>RMR</th></tr>
<tr><td>0 – 95</td><td>Sandstone, massive to thickly bedded</td><td>Fair rock
Q 4 – 4.5
RMR 46 – 60</td><td>Steel reinforced cast concrete arch, 1 - 3 m thick in crown and walls with shotcrete reinforced with weld mesh, 0.7 - 2 m thick</td><td rowspan="2">Systematic bolt 4 - 5m long, spaced 1.0 - 1.5m in crown and walls with wire mesh and light ribs steel sets spaced 1.5m where required. Shotcrete 0.1 - 0.15m in crown and 0.1m in sides</td><td rowspan="2">1.5m advance in top heading, install support concurrently with excavation 10m from face</td></tr>
<tr><td>95 – 265</td><td>Sandstone, sandstone intercalated with mudstone</td><td>Poor rock
Q 1.4 – 3.4
RMR 30 – 40</td><td>Tensioned rock bolts on grid spacing 0.5 - 1m. Chain link mesh anchored to bolts and intermediate points mean bolt length, 3.90m in crown and for walls with untensioned grouted dowels on grid spacing 1 - 1.5m, shotcrete applied directly to rock, 20 - 30mm thick, mean bolt length 3.44m</td></tr>
<tr><td>265 – 430</td><td>Sandstone intercalated with mudstone</td><td>Very poor rock
Q 0.35 – 0.58
RMR 18 – 20</td><td>Tensioned rock bolts on grid spacing 1 - 1.5m. Chain link mesh anchored to bolts and intermediate points mean bolt length, 3.90m in crown and for walls with spot reinforcement with untensioned grouted dowels, mean bolt length 3.44m</td><td>Systematic bolt 4m long, spaced 1.5 - 2.0m in crown and walls with wire mesh. Shotcrete 0.05 - 0.1m in crown and 0.03m in sides</td><td>Top heading and bench 1.5 - 3.0m advance in top heading, commence support after each blast, complete support 10m from face</td></tr>
</table>

CONCLUSIONS

The following conclusions can be drawn from the engineering geological and geotechnical investigations undertaken for the Kankai hydroelectric scheme located in eastern Nepal:

- The rock type in the scheme area is predominantly massive to slightly jointed sandstone and sandstone intercalated with mudstone both of which have very low, to low to medium strength. Bedding planes strike mostly perpendicular to the river and have moderate dip. The most predominant joint set in the rock mass is in the direction of the bedding. The rocks generally contain up to 5% of the swelling clay mineral montmorillonite.
- The engineering properties and geotechnical parameters of the rock mass within the scheme area can be considered to be poor in terms of typical parameters measured in sandstones. The unconfined compression strength varies between 6.8 and 76.2MPa, the cohesion between 0.5 and 0.74MPa, and the angle of internal friction between 23^0 and 25^0. The deformation modulus varies from 290 to 780MPa. The results of water pressure tests, laboratory tests and general groundwater measurements and observations indicate that the rock body of the project area is less permeable.
- Rock mass classification was carried out using the RMR and Q systems. The systems classified the rock mass within the scheme area as being very poor, to poor to fairing quality. In terms of the determination of strength and deformability, both systems indicated the rock to be of generally low strength. Construction of the tunnel will require the installation of primary support measures after excavation, such as rock bolting, applying of shotcrete with a reinforced mesh, steel ribs etc.
- Slope stability analyses indicated that during dam construction and impounding of the Kankai river to form the reservoir, the slopes on the abutments of the dam will become unstable, if excavated to angles steeper than 30^0 on the left abutment and 35^0 on the right abutment. To avoid failure of the slopes and progressive erosion

of the rock strata, protective measures. The also applies to other areas where slopes may be formed to facilitate the construction of other major civil engineering structures such as the intake portal, the headrace tunnel and the powerhouse.

- For the final design more detailed engineering geological investigations including drilling of boreholes, seismic survey, rock mechanical in situ tests and laboratory tests are proposed.

Acknowledgements: This study has been supported by Nepal Electricity Authority, the owner of the scheme. The authors are sincerely grateful to the Authority for providing the opportunity to participate in the feasibility study and make use of the investigation data to advance knowledge in rock engineering practice in the region. The assistance given by the engineers and geologists of the Authority to the authors during the course of the study is sincerely appreciated.

Corresponding author: Mr Sunil Kumar Dwivedi, Department of Physics and Earth Sciences, University of the Ryukyus, 1 Senbaru, Nishihara, Okinawa, 903-0213, Japan. Tel: +81 90 6866 1540. Email: k048313@eve.u-ryukyu.ac.jp.

REFERENCES

BARTON, N.R., LIEN, R. & LUNDE, J. 1974. Engineering classification of rock masses for the design of tunnel support. *Rock Mechanics*, **6**(4), 189-239.

BIENIAWSKI, Z.T. 1989. *Engineering Rock Mass Classifications*. Wiley, New York.

DEOJA, B.B. 2000. Mountain roads development in Nepal: engineering geological concern. Journal of Nepal Geological Society (special issue), **21**, 167-178.

DHITAL, M.R., UPRETI, B.N., DANGOL, V., BHANDARI, A.N. & BHATTARAI, T.N. 1991. Engineering geological methods applied in mountain road survey: An example from Baitadi-Darchula road project (Nepal). *Journal of Nepal Geological Society*, **7**, 49-67.

DIEDERICHS, M.S. & HOEK, E. 1989. *DIPS Advanced version 3.* Rock Engineering Group, Department of Civil Engineering, University of Toronto, Canada.

FOOKES, P.G., SWEENEY, M., MANBY, C.N.D. & MARTIN, R.P. 1985. Geological and geotechnical engineering aspects of low-cost roads in mountainous terrain. *Engineering Geology*, **21**, 1-152.

GOEL, R.K., JETHWA & J.L, PAITHANKAR, A.G. 1995. Tunnelling through the young Himalayas – A case history of the Maneri – Utterkashi power tunnel. *Engineering Geology*, **39**, 31-44.

GURUNG, N. & IWAO, Y. 1998. Observations of deformation and engineering geology in the Lam Ta Khong tunnel, Thailand. *Engineering Geology*, **51**, 55-63.

KAFLE, K.N. 1996. Engineering geological study of the Kali Gandaki "A" hydroelectric project area, western Nepal Himalaya. *Journal of Nepal Geological Society*, **13**, 65-71.

PAUDEL, T.R., DANGOL, V. & SHARMA, R.H. 1998. Construction phase engineering geological study in Modi Khola hydroelectric project, Parbat district, western Nepal. *Journal of Nepal Geological Society*, **18**, 343-355.

SCHELLING, D. & ARITA, K., 1991. Thrust tectonics, crustal shortening, and the structure of the far-eastern Nepal Himalaya. *Tectonics*, **10**, 851-862.

THAPA, D.B. 1985. Geology and foundation treatment of Kulekhani dam, Central Nepal. *Journal of Nepal Geological Society*, **3**, 21-32.

UPRETI, B.N. 1999. An Overview of the stratigraphy and tectonics of the Nepal Himalaya. *Journal of Asian Earth Sciences*, **17**, 577-606.

WANG, T.T. & HUANG T.H. 2002. An experience of tunneling in mudstone area in southwestern Taiwan. *Tunnelling and Underground Space Technology*, **17**, 425-436.