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Engineering geology of Kankai Hydroelectric tunnel alignment in east Nepal

***Sunil Kumar Dwivedi[1] and Prakash Chandra Adhikary[2]**
[1]*Department of Physics and Earth Sciences, University of the Ryukyus, Okinawa, 901-0213, Japan*
[2]*Central Department of Geology, Tribhuvan University, Kathmandu, Nepal*
*(*Email: k048313@eve.u-ryukyu.ac.jp)*

ABSTRACT

This paper describes the engineering geological characteristics of rock mass in the headrace tunnel, powerhouse, and intake portal of the Kankai Hydroelectric Project. The project area lies in the Lower Siwaliks of east Nepal and consists of alternating sandstone and mudstone beds with frequent siltstone intercalations. The rock mass of the project area was classified according to rock mass rating (RMR) and rock mass quality index (Q) systems. It is of very poor, poor, to fair quality (categories V, IV, and III) in the headrace tunnel; of very poor quality (category V) in the powerhouse; and of fair quality (category III) in the intake portal. The stability analysis of irregularly jointed and fractured rocks of the area was carried out using SWEDGE and UNWEDGE. The analysis gave the safety factor of 0.45, 0.64, and 0.45, respectively for the powerhouse, intake portal, and headrace tunnel. The final safety factors obtained after the installation of support for powerhouse, intake portal, and headrace tunnel were 1.14, 3.33, and 4.53, respectively.

INTRODUCTION

Every hydroelectric project is unique in terms of its engineering problems and the Kankai Hydroelectric Project is no exception. The engineering geological studies of roads, irrigation canals, and bridges are common in Nepal (Dhital et al. 1991; Deoja 2000). However, there are a limited number of studies on hydroelectric projects (Kaphle 1996; Paudel et al. 1998; Dwivedi 2003), and the study of soft rock tunnelling is rather rare. In these circumstances, the study of engineering geological and tunnelling problems in soft rock can contribute to the understanding of hydropower development in the Himalayn conditions.

In the proposed Kankai Hydroelectric Project (KHP), the rock mass was classified according to the rock mass rating (RMR) (Bieniawski 1989) and NGI (Norwegian Technical Institute) tunnelling quality index (Q) systems (Barton et al. 1974). The support systems were further selected according to each of these classification systems. In order to come up with a suitable geometry and to determine the stability of slopes, cut slope and tunnel sections of the project were analysed using SWEDGE and UNWEDGE, and required support systems were worked out.

PROJECT AREA

The proposed KHP is situated on the left bank of the Kankai River, on the Siwalik foothills of east Nepal (Fig. 1). It is bounded by latitude 26°41′00″N and 26°42′00″N, and longitude 87°52'30″E and 87°53′00″E. The poject area exhibits very steep, rugged, and dissected topography, characterised by a typical hogback structure with a southward escarpment. Interbedded sandstone and mudstone beds have given rise to alternating ridges and furrows in the area. The Kankai River, which is a rain-fed river, originates from the Mahabharat range and exhibits a dendritic drainage pattern. Strong denudation and linear erosion below the forest cover are frequent in the area. The proposed KHP is a 60 MW storage-type multipurpose scheme for hydropower generation and irrigation. The project includes a 70 m high dam, a 320 m long headrace tunnel, a toe-powerhouse, a 900 m long diversion tunnel, a spillway, and a reservoir (NEA 2002).

GEOLOGICAL SETTING

The Siwalik Group in the study area is bordered in the north by the Main Boundary Thrust (MBT) and in the south by the Himalayan Frontal Thrust (HFT). The Group is divided into the Lower, Middle, and Upper Siwaliks (Schelling and Arita 1991; Upreti 1999) and is represented by a thick pile of mudstone, sandstone, and conglomerate (Fig. 1). The major engineering structures of KHP (viz. intake portal, powerhouse, headrace tunnel, and dam), lie in the Lower Siwaliks represented by alternating beds of sandstone, mudstone, and siltstone. The sandstone is grey in colour, fine- to medium-grained, soft, argillaceous, poorly indurated, and highly jointed. The mudstone is bioturbated, variegated, and poorly indurated. In this area, laminated mudstone and calcareous siltstone beds are also frequent. Fining-upward cycles and ripple marks are observed in the sandstone beds.

ENGINEERING GEOLOGY

The engineering geological investigation included the study of rock mass, discontinuity survey, examination of rock cores, and collection of rock samples for the laboratory study. The uniaxial compressive strength (UCS) of intact rock samples was estimated by point load test (Table 1); rock quality designation (RQD) data were estimated from the borehole as well as volumetric analysis of joints; and the orientation of discontinuity sets were processed utilising a computer-based program DIPS (Diederichs and Hoek 1989). Out of a total of 509 discontinuities measured in the headrace tunnel alignment, four sets were dominant. The analysis showed a possibility of wedge failure on the north cut slope section of intake portal; plane and wedge failures on the south cut slope section of powerhouse; wedge failure on the left flank of dam; and wedge and plane failures on the right flank of dam (Fig. 2, Table 2). Except for some beautiful rock outcrops exposed along the Kankai River section, most of the project area is covered by thick unconsolidated colluvial and alluvial soils. As the tunnel alignment is covered by a thick pile of colluvial soil, the data collected along the river section were projected to the tunnel horizon.

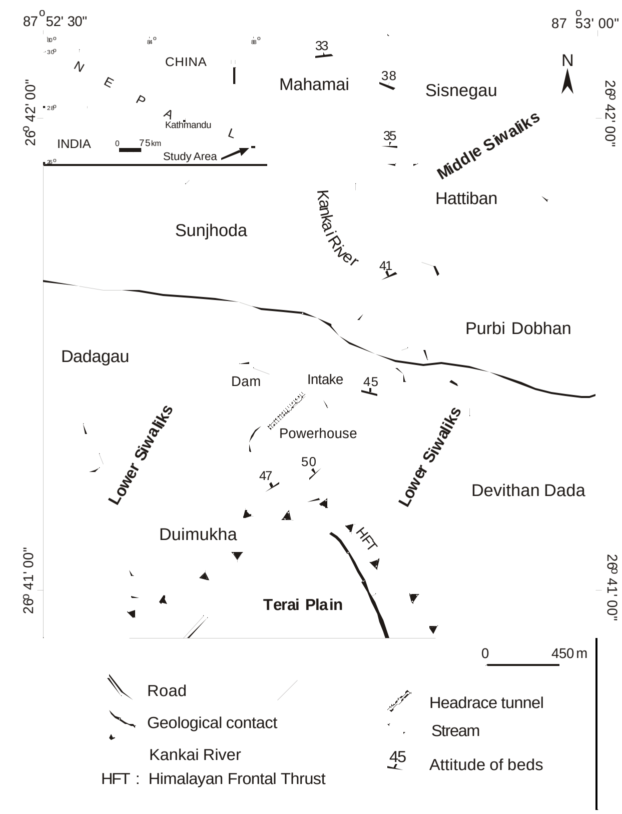

Fig. 1: Location map showing geology of the study area (after Dwivedi 2003)

Powerhouse

The proposed semi-underground powerhouse site is represented by alternating sandstone (minor) and mudstone (dominant) beds. The sandstone beds are thick to very thick and the mudstone beds are thin to massive (Table 3). Frequently the beds are calcareous. The borehole DPH-1 showed the bedrock at a depth of 2.80 m from the riverbed (Table 4). Beds are dipping due NW and their dip amount ranges from 35 to 55°. The natural slope ranges from 40° to 75°. The kinematic analysis of joints shows four major sets at the powerhouse site. The joints are mostly tight, rarely up to 1–3 mm wide, moderately open to open, smooth, slightly rough to rough, planar, and close to moderately close with moderate persistence (Table 5). The UCS of sandstone varies from 14 to 16.75 Mpa and RQD from 28 to 32 (Table 1). The borehole was dry to slightly damp.

Headrace tunnel

The proposed horse shoe-shaped headrace tunnel passes through the rock mass with a mean inclination of 6°. It is 320 m long and 8.5 m in diameter. The rock cover above the tunnel route reaches a maximum of 90 m. Engineering geological mapping along the tunnel section revealed the presence of heterogeneous rock masses consisting of alternating sandstone and mudstone beds with minor intercalations of siltstone, shale, and clay. On the basis of engineering properties, the rock masses were classified into the lower and upper portions (Table 3). In the lower portion, mudstone is dominant over sandstone (mst > sst), whereas in the upper portion it is reversed (sst > mst). The lower portion, where the mudstone beds are thicker than the sandstone beds, has a total thickness of 205 m. Generally,

Table 1: Result of point load test of rock samples

S. N.	Location	Width (mm)	Height (mm)	load reading (kPa)	P (kPa)	De E2	Is (MPa)	De (mm)	F	Is(50) = (F*Is)	UCS (MPa)	UTS (MPa)
1	Powerhouse	71	66	2750	3121.25	5966.40	0.52	77.24	1.22	0.64	14.63	0.73
2	Powerhouse	61	61.5	2650	3007.75	4776.56	0.63	69.11	1.16	0.73	16.75	0.84
3	Dam site	69	61.5	5250	5958.75	5402.99	1.10	73.51	1.19	1.31	30.17	1.51
4	Dam site	65	54	5025	5703.38	4469.07	1.28	66.85	1.14	1.45	33.45	1.67
5	Dam site	65	60	7550	8569.25	4965.63	1.73	70.47	1.17	2.01	46.32	2.32
6	Intake portal	71	55	8050	9136.75	4972.00	1.84	70.51	1.17	2.15	49.34	2.47
7	Intake portal	74	70	10400	11804.00	6595.38	1.79	81.21	1.24	2.23	51.20	2.56
8	Dam site	62	51	7750	8796.25	4025.98	2.18	63.45	1.11	2.43	55.94	2.80
9	Dam site	73	62	11250	12768.75	5762.68	2.22	75.91	1.21	2.67	61.50	3.07
10	Dam site	61	60	17250	19578.75	4660.06	4.20	68.26	1.15	4.83	111.1	5.56

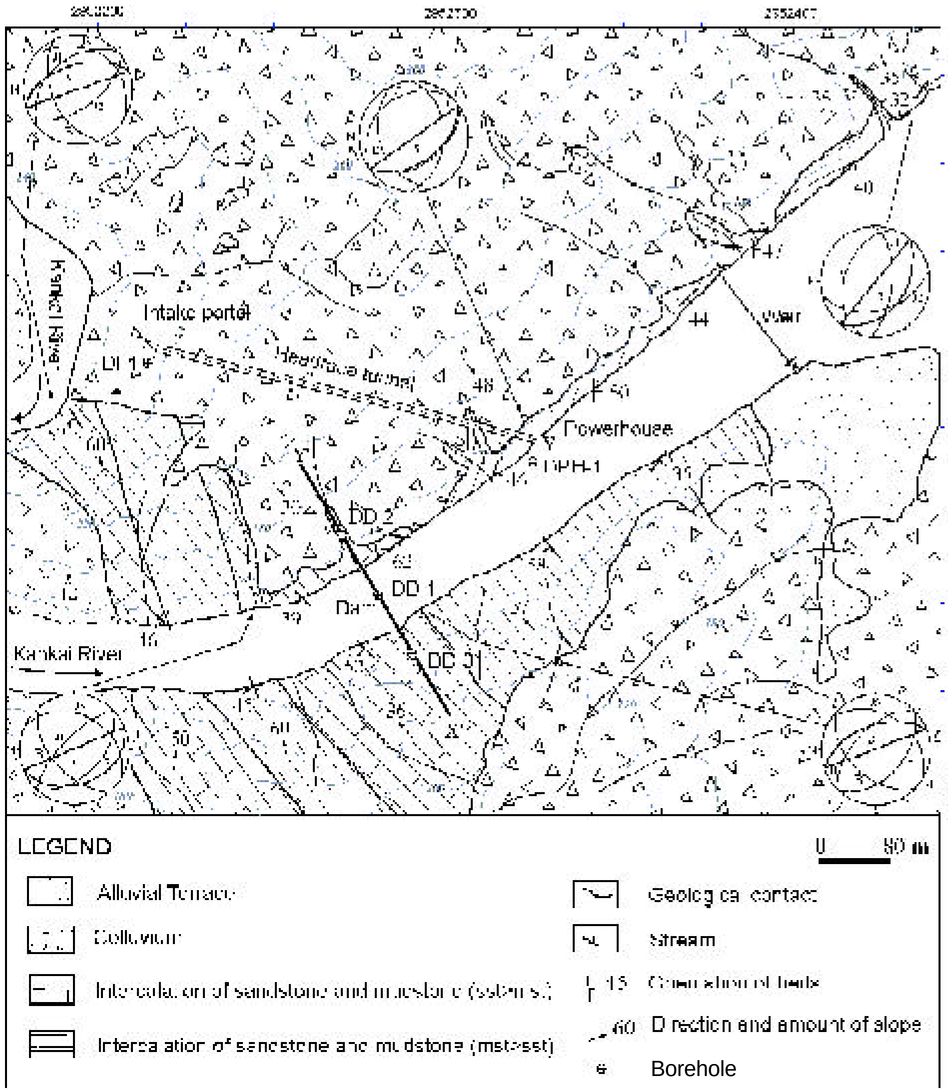

Fig. 2: Engineering geological map of the project area. sst = sandstone, mst = mudstone

the mudstone beds are 1 to 5 m thick and the sandstone beds are 1 to 3 m thick. Similarly, the upper portion, where sandstone beds are thicker than mudstone beds, has a total thickness of 450 m. In the upper portion, sandstone beds are thick to massive, and multi-storeyed beds are common. The thickness of sandstone beds varies between 10 cm and 10 m, whereas that of mudstone beds from 1 to 2 m. In this part fining-upward sequences are common.

About 63% of the tunnel length passes through a sandstone-dominant rock type and 37% passes through a mudstone-dominant sequence (Fig. 3). The sandstone and mudstone are slightly to moderately weathered, slightly to moderately jointed, blocky to seamy, and locally fractured. Weathered and fractured zones were also observed in the drilled cores. Joints are mostly tight, occasionally 1–5 mm wide, moderately open to open, smooth, slightly rough to rough, planar, and close to moderately close with moderate persistence (Table 5). Generally, the dip direction of beds is due N to NW with a dip amount ranging from 40^{o} to 60^{o}. The UCS of sandstone varies from 16.75 to 81.5 Mpa and RQD varies from 32 to 82. The kinematic analysis of joints for overall tunnel section shows four major sets: J1 (54/319; angle of dip/ dip direction), J2 (52/233), J3 (46/184), and B (53/316). Field and borehole data collected for the headrace tunnel in the dam site were used to prepare a section along the dam axis (Fig. 4).

Tunnel stability

Generally, the stability of tunnel is reduced and the possibility of overbreak increases when the angle between the tunnel axis and the predominant joint set 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

Table 2: Summary of kinematic analysis of discontinuities

Location	Attitude of major discontinuities (dip/dip dir.) Bedding plane (B)	Joint set 1 (J1)	Joint set 2 (J2)	Joint set 3 (J3)	Slope face	Type of failure
Dam site (LB)	54/319	72/044	50/225	35/119	33/248	Wedge
Dam site (RB)	52/314	58/045	49/224	31/121	53/071	Plane/ Wedge
Powerhouse	39/337	25/047	52/233	46/184	48/235	plane/ Wedge
Intake	52/335	42/045	-	39/100	70/060	Wedge
Headrace tunnel	53/316	72/046	43/ 238	29/121	-	Wedge

Table 3: Rock type along the tunnel section

Tunnel Section	Rock type		Section length (m)	No. Quantity of layers	Layer thickness (m)	Percentage of single bed thickness (m) Massive >2	Thick 2-0.5	Medium thick 0.5-0.1	Thin <0.1
0–0+30	Lower portion (mst > sst)	mst	30	24	22.87	52.5	26.2	15.3	6.0
0+30–0+60		mst, siltst	30	18	23.85	41.9	29.4	19.0	9.7
0+60–0+90		mst, siltst	30	28	20.1	54.7	24.9	14.9	5.5
0+90–0+120		mst, siltst, sst	30	30	18.65	53.6	21.4	16.6	8.3
0+120–0+150		mst, siltst, sst	30	45	24.8	56.5	20.2	13.1	10.3
0+150–0+180		mst, sst	30	12	18.14	49.6	27.6	17.2	5.6
0+180–0+210		mst, siltst	30	33	28.91	55.3	20.8	14.3	9.7
0+210–0+240		sst, mst	30	30	27.6	52.5	21.7	17.0	8.8
0+240–0+270	Upper portion (sst > mst)	sst, mst	30	12	16.8	41.7	20.8	29.8	7.7
0+270–0+300		sst, mst	30	26	27.05	44.4	20.1	20.7	14.8
0+300–0+330		sst	30	16	21.1	37.9	28.4	27.5	6.2
0+330–0+360		sst	30	22	25.52	49.0	27.4	16.9	6.7
0+360–0+390		sst, mst	30	18	23.87	48.2	29.3	16.8	5.7
0+390–0+420		ssst, siltst, mst	30	28	24.95	52.1	21.2	14.0	12.6
0+420–0+450		sst, mst	30	25	29.59	58.8	22.1	11.8	7.2
0+450–0+480		sst, mst	30	21	24.1	51.7	26.8	19.1	2.5

mst = mudstone, siltst = siltstone, sst = sandstone

Table 4: Distribution of Boreholes in the project area (NEA 2002)

S. N.	Borehole No.	Elevation (m)	Location	Overburden thickness (m)	Total depth (m)
1	DD-1	121.85	Center of dam axis	15.02	60.30
2	DD-2	143.17	Left bank, dam site	2.10	35.15
3	DD-3	139.74	Right bank, dam site	-	35.50
4	DI-1	183.05	Intake portal	11.90	25.15
5	DI-2	183.55	Headrace tunnel	5.00	25.10
6	DPH-1	122.00	Powerhouse site	2.80	43.00

the predominant joint directions. For the KHP tunnel, the rose diagram prepared from total 509 discontinuities shows the orientation of tunnel axis 193^{o} which is favourable for tunnel stability (Fig. 5).

Intake portal

The intake portal is represented by alternating sandstone and mudstone beds where the sandstone dominates over the mudstone. The rock dips due NW with an amount ranging from 40^{o} to 60^{o}. The natural slope varies from 45^{o} to 75^{o}. A kinematic analysis of joints showed a wedge failure on the north cut slope. The observed joints are mostly tight, occasionally 1–3 mm wide, moderately open to open, smooth, slightly rough, planar, and close to moderately close with moderate persistence. The UCS of sandstone ranges from 81.5 to 111.5 MPa and RQD varies from 72 to 82. The intake portal alignment is almost dry.

ROCK MASS CLASSIFICATION AND SUPPORT SYSTEM

The tunnel alignment was divided into 16 sections of 30 m interval (Fig. 3). The engineering geological survey data and borehole log (Table 5) were used for the assessment of

Table 5: Field observations of rock conditions for rock mass classification

Tunnel Section	Lithology	UCS (MPa)	Discontinuities					
			Mean spacing	Estimated RQD	Length (mm)	Width (mm)	Roughness	Groundwater condition
0–0+30	mst	14.63	0.03 ± 0. 01	28	1-3.	< 0.1	slightly rough	dry–slightly damp
0+30–0+60	mst, siltst	16.75	0.04 ± 0.01	32	< 1	< 0.1	slightly rough	dry–slightly damp
0+60–0+90	mst, siltst	30.17	0.05 ± 0.01	34	< 1	< 0.1	Rough	dry
0+90–0+120	mst, siltst, sst	55.94	0.08 ± 0.02	49	1-3.	0.1 - 1.0	Rough	dry
0+120–0+150	mst, siltst, sst	33.45	0.08 ± 0.03	56	1-3.	none	Smooth	dry–slightly damp
0+150–0+180	mst, sst	55.94	0.03 ± 0.01	43	1-3.	none	slightly rough–rough	dry
0+180–0+210	mst, siltst	46.32	0.02 ± 0.01	33	1	0.1 - 1.0	slightly rough–rough	dry
0+210–0+240	sst, mst	46.87	0.10 ± 0.02	54	1	none	Smooth	dry
0+240–0+270	sst, mst	66.09	0.13 ± 0.02	62	< 1	none	Smooth	dry
0+270–0+300	sst, mst	49.34	0.14 ± 0.02	73	< 1	none	Rough	dry–slightly damp
0+300–0+330	sst	51.20	0.09 ± 0.02	82	< 1	< 0.1	slightly rough	dry
0+330–0+360	sst	64.75	0.07 ± 0.02	82	1-3.	0.1 - 1.0	slightly rough	dry
0+360–0+390	sst, mst	61.50	0.14 ± 0.02	76	1-3.	< 0.1	Smooth	dry
0+390–0+420	ssst, siltst, mst	81.50	0.01 ± 0.03	82	1	0.1 - 1.0	Smooth	dry
0+420–0+450	sst, mst	111.10	0.11 ± 0.01	72	1	none	slightly rough	dry
0+450–0+480	sst, mst	88.95	0.10 ± 0.01	75	1	0.1 - 1.0	slightly rough	dry

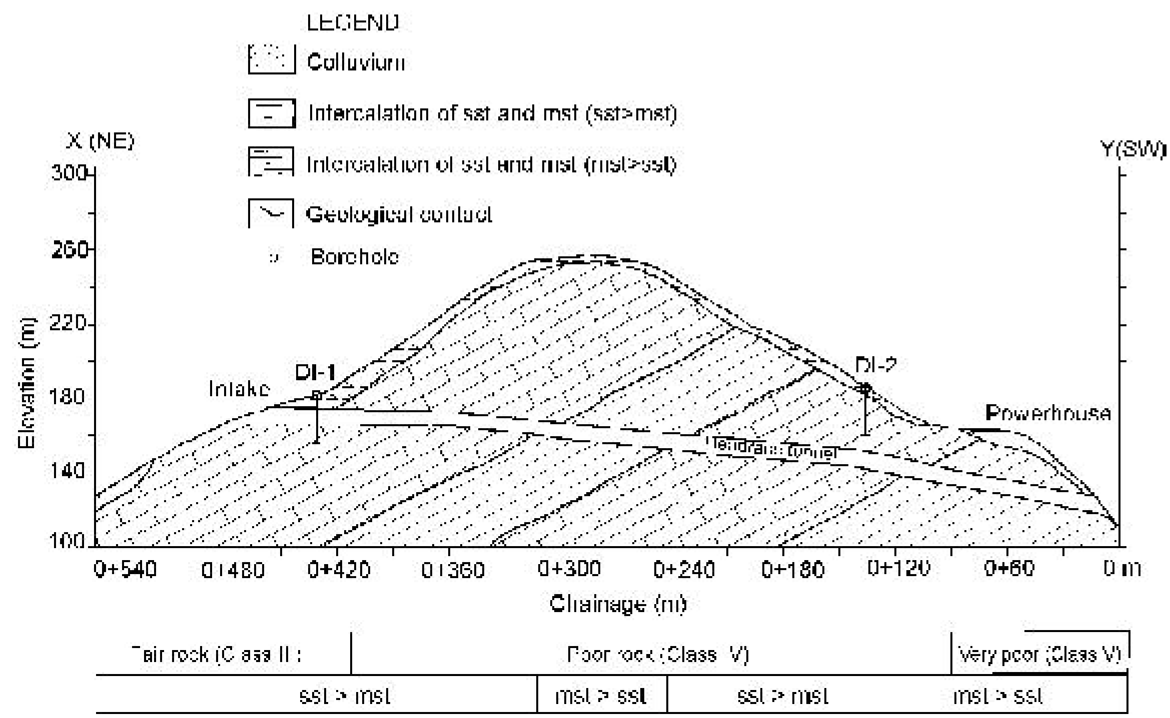

Fig. 3: Geological profile of the headrace tunnel between intake and powerhouse (see Fig. 1 for location)

RMR and Q values (Table 6) in the tunnel alignment. The analysis showed similar RMR and Q values for the entire tunnel alignment. The quality of rock mass in the powerhouse site was very poor (category V). It was of very poor to fair quality (categories V, IV and V) in the headrace tunnel alignment and of fair quality (category III) in the intake portal (Table 6). About 69% of the tunnel length will cross poor rock, 19% will cross very poor rock, and 12% will cross fair rock (Fig. 3).

Since the rock mass in the tunnel alignment was classified using RMR and Q systems, the support systems and excavation methods were also defined in accordance with the recommendations made in both of these systems (Table 7).

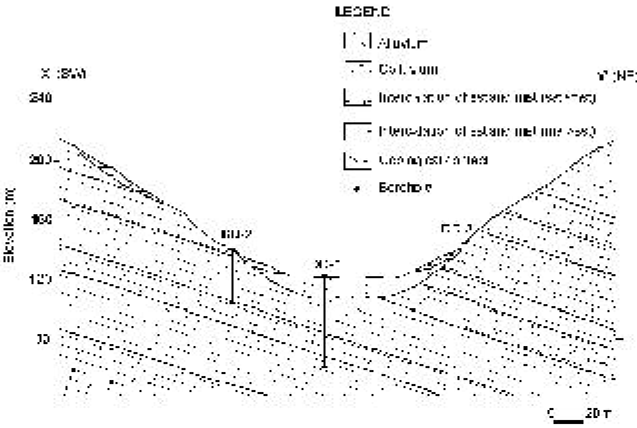

Fig. 4: Geological profile of dam along X' (SW) -Y' (NE)

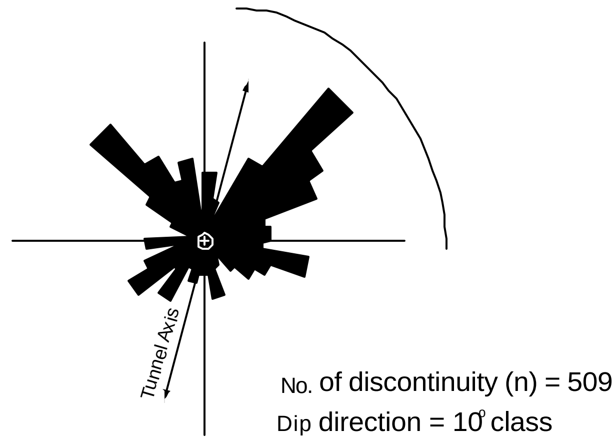

Fig. 5: Rose diagram of discontinuities along headrace tunnel

Table 6: Estimated rock mass classification systems along the headrace tunnel

Tunnel Section	UCS (MPa)	Section length (m)	RMR (range)	RMR (average)	Q	Rock Quality according to rock mass classification systems	
						RMR	Q
0–0+30	14.65	30	30–37	33.5	0.351	(21–40) Poor	(0.1–1.0) Very Poor
0+30–0+60	16.75	30	35–37	36.0	0.456	(21–40) Poor	(0.1–1.0) Very Poor
0+60–0+90	30.17	30	35–39	37.0	0.58	(21–40) Poor	(0.1–1.0) Very Poor
0+90–0+120	55.94	30	39–41	40.0	1.089	(21–40) Poor	(1.0–4.0) Poor
0+120–0+150	33.45	30	40–41	40.5	1.236	(21–40) Poor	(1.0–4.0) Poor
0+150–0+180	55.94	30	39–40	39.5	1.884	(21–40) Poor	(1.0–4.0) Poor
0+180–0+210	46.32	30	39–46	42.5	1.444	(41–60) Fair	(1.0–4.0) Poor
0+210–0+240	46.87	30	46–52	49.0	2.906	(41–60) Fair	(1.0–4.0) Poor
0+240–0+270	66.09	30	52–53	52.5	3.870	(41–60) Fair	(1.0–4.0) Poor
0+270–0+300	49.34	30	53–46	49.5	3.209	(41–60) Fair	(1.0–4.0) Poor
0+300–0+330	51.20	30	48–55	51.5	3.479	(41–60) Fair	(1.0–4.0) Poor
0+330–0+360	64.75	30	53–55	54.0	3.827	(41–60) Fair	(1.0–4.0) Poor
0+360–0+390	61.50	30	53–58	55.5	4.189	(41–60) Fair	(1.0–4.0) Poor
0+390–0+420	81.50	30	55–58	56.5	3.417	(41–60) Fair	(1.0–4.0) Poor
0+420–0+450	111.10	30	55–60	57.5	4.006	(41–60) Fair	(1.0–4.0) Fair
0+450–0+480	88.95	30	55–60	57.5	4.587	(41–60) Fair	(1.0–4.0) Fair

Some precautionary measures should be taken while installing a support system in the weak and highly permeable Siwalik sandstones (NEA 2002).

WEDGE STABILITY ANALYSIS AND SUPPORT SYSTEM

The stability analysis of proposed cut slope sections and headrace tunnel was performed using the surface and underground wedge stability analysis software package SWEDGE and UNWEDGE (Hoek et al. 1995; Rocscience Inc. 2003). SWEDGE and UNWEDGE calculate the factors of safety for potentially unstable wedges and model the influence of support systems on wedge stability. The input parameters for the SWEDGE and UNWEDGE are major intersecting discontinuity planes, face slope, slope height, rock unit weight, water unit weight, tunnel dimension, tunnel axis, water pressure, cohesion, and friction angle. However, in the present

Table 7: RMR and Q support recommendations and excavation guide

Tunnel section	Support recommendations		Guide for excavation (RMR)
	Q	RMR	
0–0+30	Steel reinforced cast concrete arch, 1–3 m thick in crown and walls with shotcrete reinforced with weld mesh, 0.7–2 m thick	Systematic bolt 4–5 m long, spaced 1.0–1.5 m in crown and walls with wire mesh and light ribs steel sets spaced 1.5 m where required. Shotcrete 0.1 - 0.15 m in crown and 0.1 m in sides	1.0–1.5 m advance in top heading, install support concurrently with excavation 10 m from face
0+30–0+60			
0+60–0+90			
0+90–0+120	Tensioned rock bolts on grid spacing 0.5–1 m, Chain link mesh anchored to bolts and intermediate points mean bolt length, 3.90 m in crown and for walls with untensioned grouted dowels on grid spacing 1–1.5 m, shotcrete applied directly to rock, 20–30 mm thick mean bolt length 3.44 m		
0+120–0+150			
0+150–0+180			
0+180–0+210		Systematic bolt 4 m long, spaced 1.5–2.0 m in crown and walls with wire mesh. Shotcrete 0.05 - 0.1 m in crown and 0.03 m in sides	Top heading and bench 1.5–3.0 m advance in top heading, commence support after each blast, complete support 10 m from face
0+210–0+240			
0+240–0+270			
0+270–0+300			
0+300–0+330			
0+330–0+360			
0+360–0+390			
0+390–0+420			
0+420–0+450	Tensioned rock bolts on grid spacing 1–1.5 m, Chain link mesh anchored to bolts and intermediate points mean bolt length, 3.90 m in crown and for walls with spot reinforcement with untensioned grouted dowels, mean bolt length 3.44 m		
0+450–0+480			

Table 8: Summary of the stability analysis of wedges by SWEDGE and UNWEDGE

Location	Rock mass type	Rock unit weight (tonnes/m^3)	Friction angle (deg.)	Rock mass class	Excavation cut slope	Sliding along one plane	Sliding along section of two planes	Wedge volume (tonnes)	Safety factor before support	Safety factor after support	Support system
Powerhouse site	mst	2.7	30	V	55/235	52/233	-	4061	0.45	1.14	rock bolts, shotcretes
Intake portal	sst, mst	2.7	30	III	55/235	45/042	24/040	1207	0.64	3.33	rock bolts and shotcretes
Headrace tunnel	sst, mst	2.7	30	III	-	52/233	-	40	0.45	4.53	spot bolts, shotcretes, support pressure

study, a dry and cohesionless rock condition is assumed for the stability analysis.

Surface wedge stability analysis

The surface wedge stability analysis using SWEDGE at the powerhouse site showed the formation of a unstable wedge for a 55^o slope face and 40 m slope height, and the responsible joint sets to form this wedge are 52/233 (J2) and 46/184 (J3) (Table 8). The wedge so formed has a safety factor of 0.45, and a weight of 4061 tonnes slides on the plane 52/233.

Similar analysis at the proposed intake portal showed the formation of a unstable wedge for a 55^o slope face and 30 m slope height, and the responsible joint sets to create this wedge were 52/335 (B) and 42/045 (J1). The wedge so formed has a safety factor of 0.64 and a weight of 1207 tonnes, slides on the plane 45/042 (Table 8).

Underground wedge stability analysis

In contrast to the surface wedges, underground wedges expected inside the tunnel were rather complex depending on their position (such as in the side-wall, crown, or invert of the tunnel). The stability analysis of underground wedges using UNWEDGE showed four wedges resulting from the intersection of the discontinuities 53/316 (B), 52/233 (J2), 46/184 (J3), and the free face created by the excavation of an underground opening along 193^o tunnel axis. Out of them, wedge number 4 had the lowest safety factor of 0.45 and a weight of 40 tonnes was going to move on the joint plane 52/233.

From the stability analysis, the slope faces at the intake portal, powerhouse, and inside the headrace tunnel were found unstable and required various support systems for their stability. After the installation of support recommended by SWEDGE and UNWEDGE, the safety factors for the powerhouse, intake portal, and headrace tunnel increased respectively to 1.14, 3.33, and 4.53 (Table 8). The support systems proposed by SWEDGE were rock bolts and shotcretes, while those of UNWEDGE were spot bolts (end anchored, fully bonded, cables, split sets), shotcrete, and support pressure. In order to prevent or minimise the build up of pore water pressure, which could lead to a slope instability, surface and subsurface drains were required.

CONCLUSIONS AND DISCUSSIONS

The KHP lies in the Lower Siwaliks and is represented by alternating sandstone and mudstone beds. A detailed engineering geological study and rock mass classification showed that the area is dominated by the low-strength soft rocks. Following Kockar and Akgun (2003), Gonzalez de Vallejo (2003), and Sari and Pasamehmetoglu (2004), the RMR and Q systems were applied to classify the rock mass. The rock mass classification showed a very poor and poor to fair rock mass (categories V, IV and III) along the headrace tunnel; a very poor rock mass (category V) in the powerhouse site; and a fair rock mass (category III) in the intake portal area. For the headrace tunnel, about 69% of tunnel length will cross poor rock, 19% will cross very poor rock, and 12% will cross fair rock. The recommended rock supports using RMR and Q systems are similar to those used by Sari and Pasamehmetoglu (2004). According to them, the RMR system is less sensitive to weak rock mass than the Q system. Hence, the Q system should be preferred.

Since the KHP tunnel alignment passes through a shallow level, the discontinuities may create potential wedges influencing its stability. The wedge stability analysis using SWEDGE and UNWEDGE gave initial safety factors of 0.45, 0.64, and 0.45, respectively for the powerhouse, intake portal, and headrace tunnel. After installation of the support recommended by the software, the final safety factors for the powerhouse, intake portal, and headrace tunnel may be increased to 1.14, 3.33, and 4.53, respectively. As mentioned by Chatziangelou et al. (2002), this study also shows that the safety factors obtained from the support measures recommended by SWEDGE and UNWEDGE are much higher than the theoretically required values.

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