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Wedge stability analysis and rock squeezing prediction of headrace tunnel, Lower Balephi Hydroelectric Project, Sindhupalchock District, central Nepal

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ABSTRACT

The wedge stability and stress analyses are important in tunnel stability assessment. The identification of the wedge stability and stress condition for the headrace tunnel suggests the required tunnel support in Lower Balephi Hydroelectric Project in Sindhupalchock District, central Nepal. The planned tunnel of the project is 4.5 m in diameter and 4.2 km in length. The main lithologies of the area along the tunnel axis are phyllite and phyllitic quartzite of the Kunchha Formation, Nawakot Complex. Wedge stability analysis in the headrace tunnel showed that the structural wedge would form due to excavation and can be stabilized with the help of rock bolting and shotcreting. Rock squeezing is predicted to occur in high tunnel depth in phyllite and it may be stabilized with the installation of rock support consisting steel rib.

Keywords: Stability, stress, tunnel, rock supports

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INTRODUCTION

Before designing rock supports, it is necessary to understand the failure mechanisms (Sunuwar 2006). Generally, there are two possible failure mechanisms in tunnels and caverns - structurally controlled and stress induced ones. Both are dependent mainly on the rock mass quality and in - situ stress condition. In tunnels, when excavation in jointed rock masses at relatively shallow depth is carried out, the most common types of failure are those involving wedge falling from the roof or sliding out of the side walls of the opening. These wedges are formed by intersecting structural features, such as bedding planes and joints, which separate the rock mass into discrete but interlocked pieces. Unless steps are taken to support these loose wedges, the stability of the roof and side walls of the opening may deteriorate rapidly (Hoek and Bray 1981). The stress level acting around the underground openings is another factor that may cause tunnel stability problems. It is evident that a tunnel fails when the stress exceeds the strength of rock mass around the opening (Shrestha and Broch 2006).

Thus determination of underground wedge stability and stress analysis is useful for the assessment of the tunnel stability. The study aims to identify the potential wedges in underground excavation, calculation of stress analysis to find out the rock squeezing problem in headrace tunnel at greater depth and to suggest required tunnel support. Rocscience softwares UNWEDGE and RockLab (Rocscience Inc. 2010) were used for the analysis.

The Lower Balephi Hydroelectric Project (LBHEP) is located about 80 km northeast of Kathmandu City in Sindhupalchock District, central Nepal (Fig. 1). It is a run-of-the- river scheme across the Balephi River. The Balephi River, which is a snow fed perennial river, originates from the Jugal Himal of the great Himalayan range of Nepal. It is a major tributary of Sunkoshi River in the Saptakoshi Basin. The project has installed a capacity of 18.5 MW with a design discharge of 34 m^3/s. A diversion weir will be constructed near Jalbire Bazaar. The main hydraulic structures of the project are 6 m high and 45 m long diversion weir, 90 m long surface settling basins, 32 m long headrace canal, 4200 m long inverted D type headrace tunnel of 4.5 m diameter, 45 m high surge tank, 150 m long penstock pipe and a surface powerhouse.

GEOLOGICAL SETTING

The Lesser Himalayan Zone in the study area is bordered in the north by the Main Central Thrust (MCT) and in the south by the Main Boundary Thrust (MBT). The rocks of this zone are sub-divided into allochthonous and autochthonous units. The allochthonous unit consists of rocks of the Kathmandu Complex and the autochthonous unit consists of rocks of the Nawakot Complex.

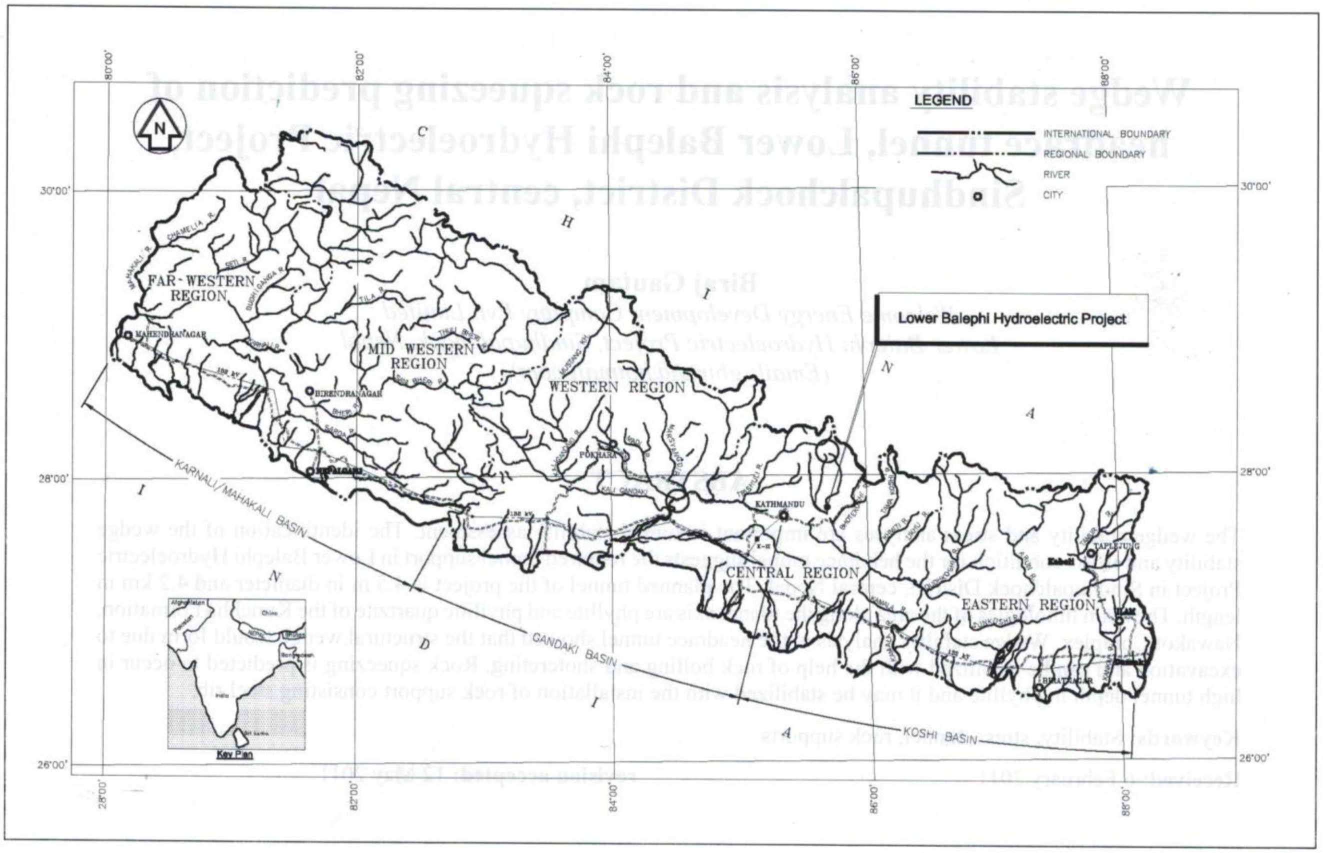

Fig. 1: Location map of the study area (Survey Department, Government of Nepal)

The Kathmandu Complex represents crystalline rocks overlain by argillaceous, araneous and fossiliferous carbonate rocks. The Kathmandu Complex forms the core of the Mahabharat Synclinorium (Stöcklin and Bhattarai 1977). The Nawakot Complex essentially consists of Precambrian sedimentary to metasedimentary rocks.

The rocks of the study area belong to the Kunchha Formation of Lower Nawakot Group of the Nawakot Complex. The Kunchha Formation consists of a monotonous sequence of flysch-like alteration of phyllite, phyllitic quartzite, and phyllite gritstones (Stöcklin and Bhattarai 1977). The study area consists of interbedding sequence of foliated phyllite and phyllitic quartzite of the Kunchha Formation, Nawakot Complex (Stöcklin and Bhattarai 1977; Stöcklin 1980) In most of the areas, the rocks consist of fine grained, thin to medium bedded, wavy, continuous, heavily jointed and slight to moderately weathered phyllite and phyllitic quartzite. Quartz veins are present in most of the exposure which are randomly folded and discontinuous. The proportion of phyllite increases northwards from Naubise Khola. The general strike of the foliation is NW-SE and its dip ranges from 25° to 75° due NE.

ENGINEERING GEOLOGY

The engineering geological investigation in this study was carried out by the study of rock mass, mapping of discontinuities, and collection of rock samples for laboratory test. Rock quality designation (RQD) data were estimated from volumetric analysis of the joints in the exposure. The orientation of discontinuity sets were processed utilizing a computer – based program DIPS. 450 discontinuities were measured in the headrace tunnel alignment

Dam site

The dam site is located at about 200 m south from the Jalbire Bazaar, at the suspension bridge to the Katarbesi Village (Fig. 2). Thick alluvial deposits comprising rounded and sub-rounded boulders and gravels of quartzite, gneiss and schist can be seen along the river banks. River bed along the dam axis is covered with thick river borne materials. Rock outcrops on both banks extend for more than 50 m upstream and downstream from the dam axis. The rock outcrop consists of bedded, planner continuous, medium to coarse grained, light grey phyllitic quartzite. Thin layers of gritty phyllite are also present, which consist of opal like milky-grey or bluish quartzite. The strikes of main foliation vary from N25°W-S25°E to N50°W-S50°E and dip 26°-32° towards NE. The rocks are moderately jointed. The joints are long

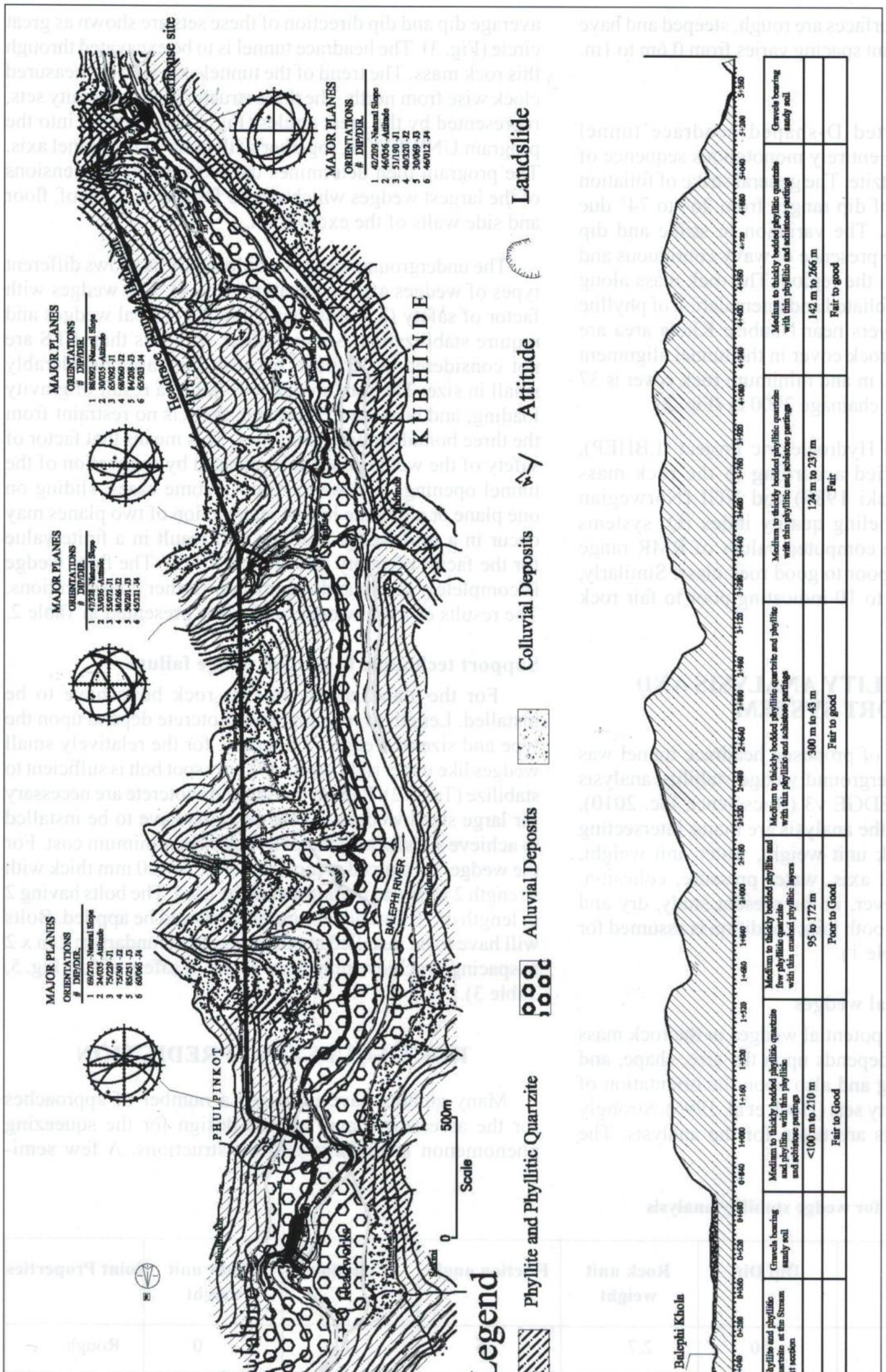

Fig. 2: Engineering geological map and geological profile of the project (WEDCO 2009)

and persistent. The joint surfaces are rough, steeped and have some silty clay fillings. Joint spacing varies from 0.6m to 1m.

Headrace tunnel

The proposed inverted D-shaped headrace tunnel alignment passes through entirely monotonous sequence of phyllite and phyllitic quartzite. The general strike of foliation is NW-SE and its angle of dip ranges from 26° to 74° due NE with local variations. The variation in strike and dip amount may be due to the presence of wavy continuous and wedge shaped foliation in the region. The rock mass along the alignment are mostly foliated and intercalation of phyllite and shared schist like layers near Naubise Khola area are common. The maximum rock cover in the tunnel alignment is 300 m at chainage 2700 m and minimum rock cover is 37 m at the Naubise Khola at chainage 2900 m (Fig. 2).

In the Lower Balephi Hydroelectric Project (LBHEP), the rock mass is classified according to the rock mass rating (RMR) (Bieniawski 1989) and NGI (Norwegian Technical Institute), tunneling quality index (Q) systems (Barton et al. 1974). The computed values of RMR range from 29 to 75 indicating poor to good rock class. Similarly, Q value ranges from 0.5 to 10 indicating poor to fair rock classes.

WEDGE STABILITY ANALYSIS AND SUPPORT SYSTEM

The stability analysis of proposed headrace tunnel was performed using the underground wedge stability analysis software package UNWEDGE v3 (Rocscience Inc. 2010). The input parameters for the analysis are major intersecting discontinuity planes, rock unit weight, water unit weight, tunnel dimension, tunnel axis, water pressure, cohesion, and friction angle. However, in the present study, dry and cohensionless rock for smooth joint condition is assumed for the stability analysis (Table 1).

Identification of potential wedges

The size and shape of potential wedges in the rock mass surrounding an opening depends upon the size, shape, and orientation of the opening and also upon the orientation of the significant discontinuity sets (Hoek et al. 1995). Strongly developed three joint sets are taken for the analysis. The average dip and dip direction of these sets are shown as great circle (Fig. 3). The headrace tunnel is to be excavated through this rock mass. The trend of the tunnel axis is 355° measured clock wise from north. The three structural discontinuity sets, represented by the great circles (Fig. 3) are entered into the program UNWEDGE, together with trend of the tunnel axis. The program then determines the location and dimensions of the largest wedges which can be formed in the roof, floor and side walls of the excavation.

The underground wedge stability analysis shows different types of wedges after excavation (Fig. 4). The wedges with factor of safety (FS) less than 1 are the critical wedges and require stabilization. Some wedges with less than 1 FS are not considered as 'critical' because they are considerably small in size. The roof wedge will fall as a result of gravity loading, and because of its shape, there is no restraint from the three bounding discontinuities. This means that factor of safety of the wedge, once it is released by excavation of the tunnel opening, is close to zero. In some cases, sliding on one plane or along the line of intersection of two planes may occur in a roof wedge and this will result in a finite value for the factor of safety (Hoek et al. 1995). The floor wedge is completely stable and requires no further considerations. The results of the unwedge analysis is presented in Table 2.

Support technique to control wedge failure

For the stabilization process rock bolts have to be installed. Length of rock bolt and shotcrete depend upon the type and size of wedges. Generally, for the relatively small wedges like upper right wedge (7) only spot bolt is sufficient to stabilize (Table 2). Pattern bolting and shotcrete are necessary for large sized wedges. These supports have to be installed to achieve satisfactory factor of safety in minimum cost. For the wedge 7 produced at roof, shotcrete of 100 mm thick with strength 2 Mpa and bolting should be used. The bolts having 2 m length of 10 tons anchor capacity should be applied. Bolts will have to be installed at normal to the boundary in 2 m x 2 m spacing to achieve minimum factor of safety of 1.6 (Fig. 5, Table 3).

ROCK SQUEEZING PREDICTION

Many authors have proposed a number of approaches for the assessment and support design for the squeezing phenomenon in underground constructions. A few semi-

Table 1: Input parameters for wedge stability analysis

Major Joint Sets	Dip	Dip Dir.	Rock unit weight	Friction angle	Cohesion	Water unit weight	Joint Properties
Joint Set 1	40	50	2.7	35	1	0	Rough
Joint Set 2	65	190	2.7	20	0	0	Smooth
Joint Set 3	73	281	2.7	20	0	0	Smooth

analytical approaches have been proposed for estimation of the deformation caused by squeezing and estimation of support pressure required in the squeezing tunnel. For this approach Hoek and Marinos (2000) showed that a plot of tunnel strain ε against the ratio of uniaxial compressive strength of rock mass to in situ stress could be used effectively to assess tunneling problems under squeezing conditions.

In situ stress

The stresses that exist in the rock mass, are related to the weight of the overburden but also to its geological history. Knowledge of the in situ state of stress in a rock mass is important in civil and mining engineering. The stress magnitudes in the rock mass generally increase with depth. Consequently, stress-related problems such as failure due to high stress magnitude also increase with depth. However, excavations at shallow depth may also be challenging, either because of high horizontal stresses or due to the lack of horizontal stresses (Amadei and Stephansson 1997).

The in situ stress is normally described with vertical stress and horizontal stress that are denoted by σ_v and σ_H. The vertical principal stress is usually a result of the weight of the overburden per unit area above a specific point in the rock mass and is normally assumed to be a function of depth

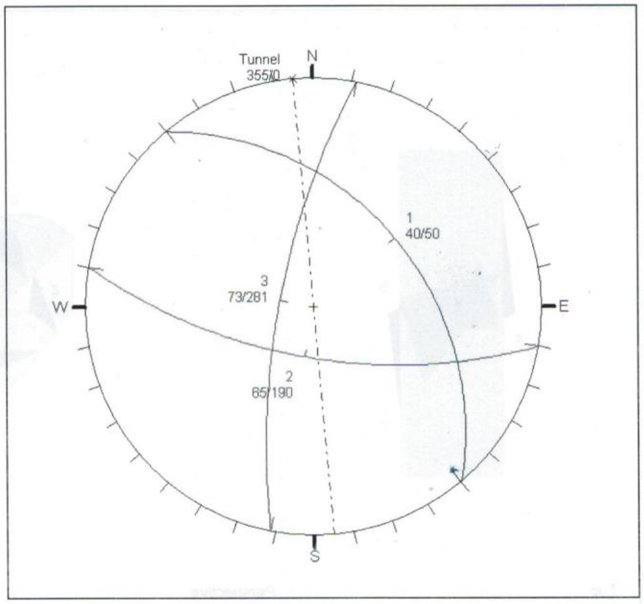

Fig. 3: Stereographic projection of average dip and dip directions of three discontinuities sets in a rock mass and trend of tunnel axis

Table 2: Wedge information calculated after UNWEDGE software

Wedge	Wedge Volume m3	Weight tonnes	Failure mode	Factor of Safety
Floor Wedge (1)	7.019	18.951	Stable	Stable
Lower Left Wedge (4)	0.864	2.333	Wedge sliding on Joint 1	0.434
Lower Right Wedge (5)	0.864	2.333	Wedge sliding along line of intersection of joints 2 and 3	0.651
Upper Right wedge (7)	0.01	0.027	Wedge sliding on joint 3	0.111
Roof wedge (8)	2.148	5.8	Falling wedge	0.00

Table 3: Designed support details to stabilize critical wedges in the headrace tunnel

Wedges	Support detail	FS before support	FS after support
Lower Left wedge (4)	Pattern bolts normal to boundary 2.0m X 2.0m spacing	0.434	3.862
Lower Right Wedge (5)	2.0m length	0.651	3.537
Upper Right wedge (7)	Bolt Type: Mechanically Anchored	0.111	0.111
Roof wedge (8)	Tensile Capacity: 10 tonnes Plate Capacity: 10 tonnes Anchor Capacity: 10 tonnes	0.00	1.636

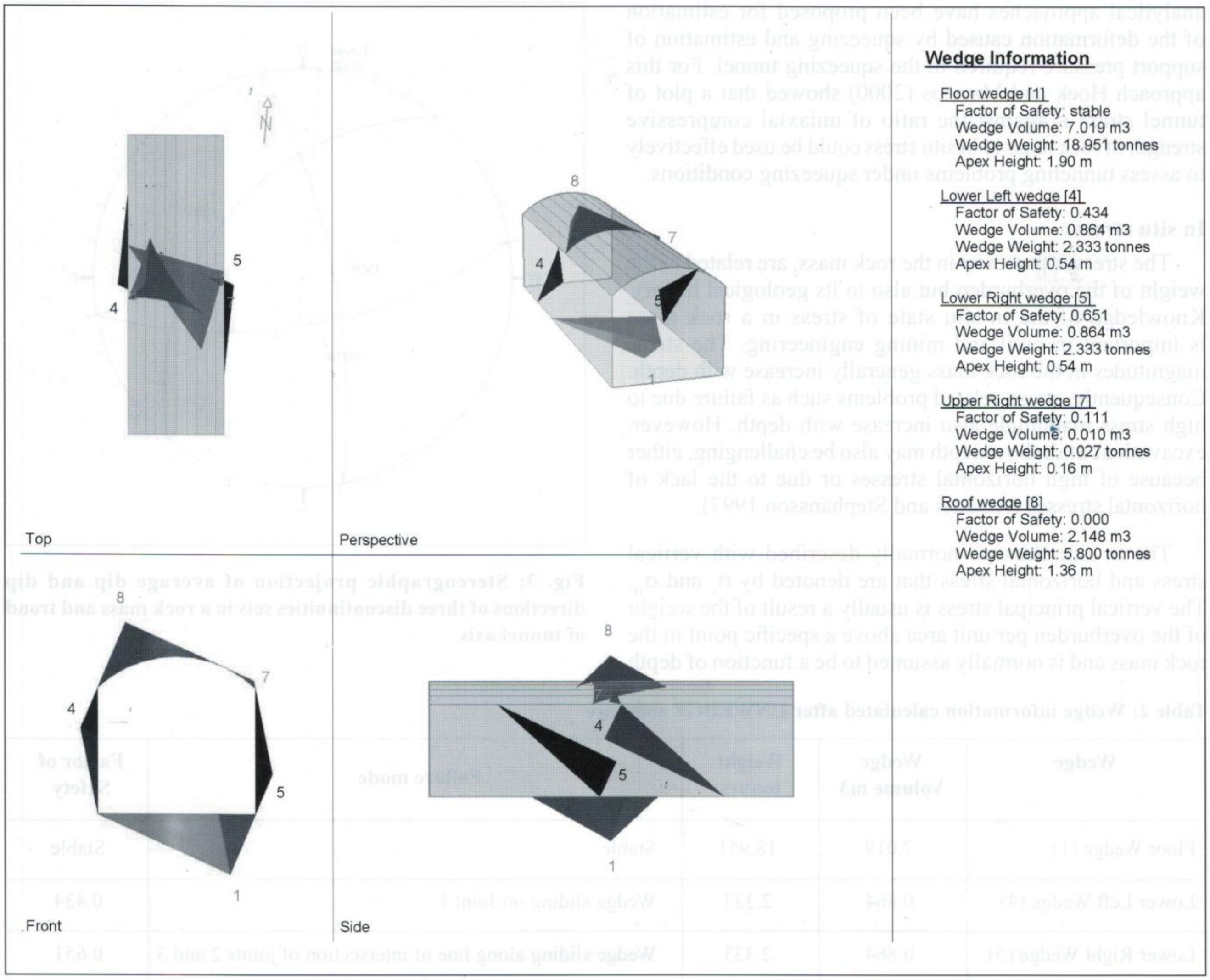

Fig. 4: Wedge formed in the roof, floor, and side walls of proposed headrace tunnel

and is defined as,

$$\sigma_v = \gamma Z \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(1)$$

Where γ = unit weight of the overlying rock (0.027 kg/cm^3), Z= depth below the surface in meter.

The maximum rock cover in the headrace tunnel is 300 m. Using Equation (1) the vertical stress in the study area was 8.1 MPa.

Estimation of rock mass properties

In order to analyze the behavior of tunnel and its face under a variety of conditions, some means of estimating the properties of rock mass is required (Hoek 2000).The strength of jointed rock mass is difficult to assess. Laboratory test on core samples are not representatives of a rock mass of significantly larger volume. On the other hand, in situ strength testing of rock mass is practically or economically feasible. Back-analysis of observed failures can provide representative values of the rock mass strength, but obviously, this is only possible for cases in which rock mass failure has occurred. The more general problem of forward strength prediction for large scale rock masses remains as one of the great challenges in rock mechanics.

The Hoek-Brown failure criterion and use of rock mass classification systems are useful tools to assess the rock mass strength (Hoek et al. 2002).

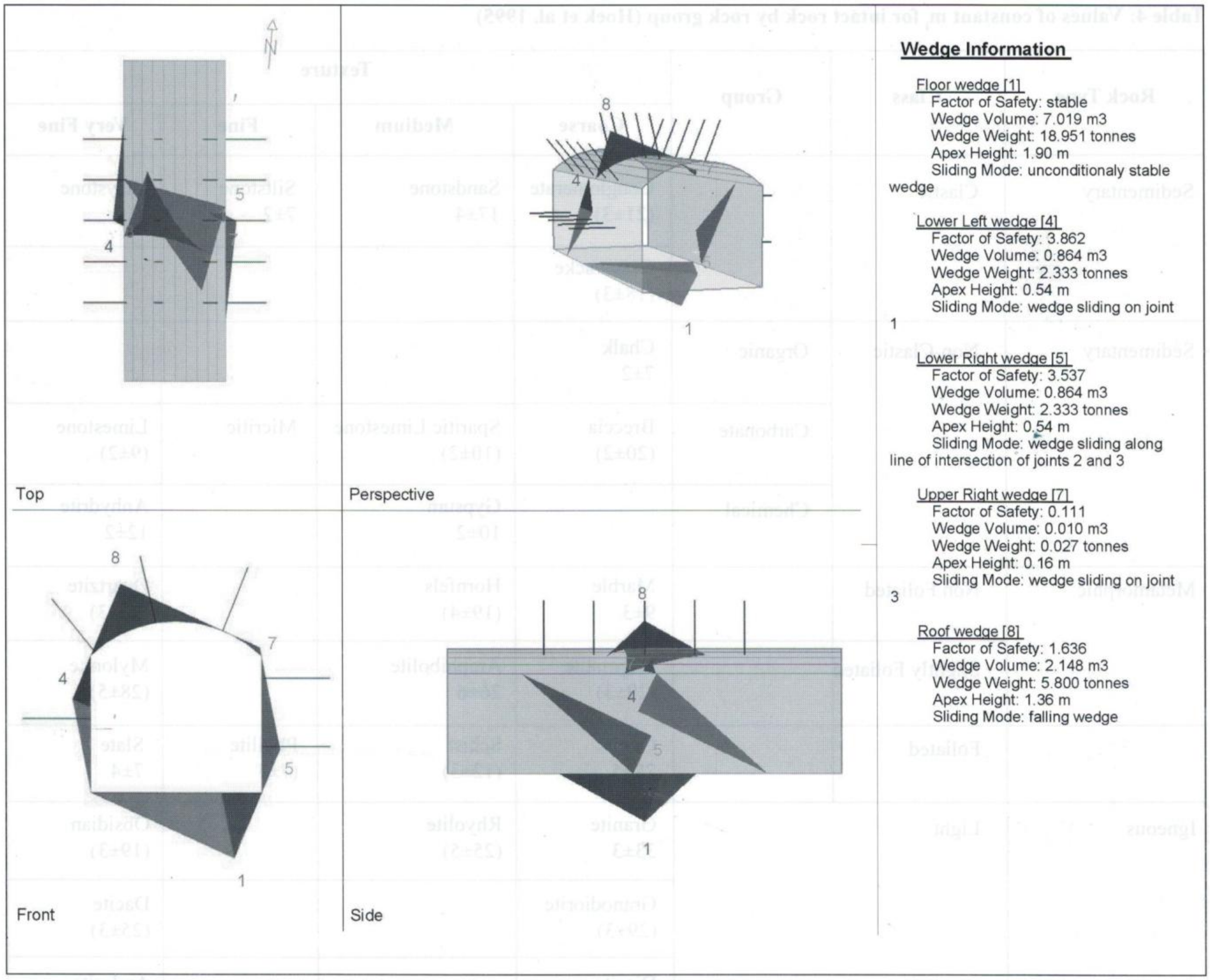

Fig. 5: Rock bolting pattern to stabilize the roof and sidewall wedges in the tunnel

$$m_b = m_i \exp\left[\frac{\text{GSI - 100}}{28-14D}\right] \qquad \text{...(2)}$$

Constant s and a for the rock mass can be calculated by the following relationship

$$s = \exp\left[\frac{\text{GSI - 100}}{9-3D}\right] \qquad \text{...(3)}$$

$$a = \frac{1}{2} + \frac{1}{6}\left[e^{-GSI/15} - e^{-20/3}\right] \qquad \text{...(4)}$$

$$\sigma'_{cm} = \sigma_{ci}.\frac{(m_b + 4s - a(m_b - 8s)(m_b/4+s)^{a-1}}{2(1+a)(2+a)} \qquad \text{...............(5)}$$

Where m_b = reduced value of material constant mi

GSI = Geological Strength Index

D = Disturbance factor

σ'_{cm} = Uniaxial compressive strength of rock mass

σ_{ci} = Uniaxial compressive strength of intact rock material

The constant m_i can be determined from triaxial test on intact rock. In study area the test results are not available therefore, it was determined from the tabulated data provided by Hoek et al. (1995) in Table 4.

Table 4: Values of constant m_i for intact rock by rock group (Hoek et al. 1995)

Rock Type	Class	Group	Texture			
			Coarse	Medium	Fine	Very Fine
Sedimentary	Clastic		Conglomerate (21±3)	Sandstone 17±4	Siltstone 7±2	Claystone 4±2
			Greywacke (18±3)			
Sedimentary	Non-Clastic	Organic	Chalk 7±2			
		Carbonate	Breccia (20±2)	Sparitic Limestone (10±2)	Micritic	Limestone (9±2)
		Chemical		Gypsum 10±2		Anhydrite 12±2
Metamorphic	Non Foliated		Marble 9±3	Hornfels (19±4)		Quartzite (20±3)
	Slightly Foliated		Migmatite (29±3)	Amphibolite 26±6		Mylonite (28±5)
	Foliated		Gneiss 28±5	Schist (12±3)	Phyllite (7±3	Slate 7±4
Igneous	Light		Granite 33±3	Rhyolite (25±5)		Obsidian (19±3)
			Granodiorite (29±3)			Dacite (25±3)
			Diorite (25±5)			Andesite (25±5)
	Dark		Gabbro 27	Dolerite (16±5)		Basalt (25±5)
			Norite 20±5			
	Extrusive Pyroclastic type		Agglomerate (19±3)	Breccia (19±5)		Tuff (13±5)

All of these equations are incorporated in to the windows software RocLab that can be downloaded from the internet site www.rocscience.com.

Calculation of rock mass strength using RocLab software

RocLab is a software program for determining rock mass strength parameters, based on the latest version of generalized Hoek-Brown failure criterion. The program RocLab provides a simple and intuitive implementation of the Hoek-Brown failure criterion, allowing users to easily obtain reliable estimates of rock mass properties, and to visualize the effects of changing rock mass parameters, on the failure envelopes (Rocscience Inc. 2010).

For a given set of input parameters intact uniaxial compressive strength (σci), geological strength index (GSI), rock mass constant (m_i), disturbance factor (D), unit weight of rock mass, depth to tunnel and modulus ratio (Table 5), RocLab calculates the parameters of the generalized Hoek-Brown failure criterion (mb, s and a), Mohr Columb Fit and

Rock Mass parameters as out put parameters (Table 6). Value for σci and unit weight of rock mass have been obtained from the laboratory tests, but the values for GSI, *mi*, MR and D are assumed (Table 5). The rock mass strength has been evaluated for maximum rock cover area of head race tunnel which is 300 m at phyllite rock section (Fig. 6). From the RocLab program, the rock mass strength has been computed as 2.075 Mpa (Table 6).

Potential squeezing problem in tunnel

The rock mass around the tunnel contour fails due to various reasons such as when the tangential stresses approach/exceed the rock mass strength. Tunnel excavation results in redistribution of stress around tunnel. Since no in situ rock stress measurement has been performed in this project, the major principle stress has been assumed to be due to rock cover above the tunnel roof. Maximum rock cover along the headrace tunnel is about 300 m at chainage 2700 m from intake portal at phyllite rock section. Considering the unit weight 0.027 Kg/cm^3 for phyllite rock, the rock cover produces vertical in-situ stress of 8.1 MPa. The rock mass strength has been computed to be 2.07 MPa, less than in-situ stress.

Hoek (1999) published details of an analysis showing that the ratio of rock mass strength σ_{cm} of the rock mass to the in situ stress ρ_0 can be used as an indicator of potential tunnel squeezing problems. Following the suggestions of Sakurai (1983), an analysis was carried out to determine the relationship between σ_{cm}/ρ_0 and the percentage "strain" of the tunnel. The percentage strain ε is defined as 100 X the ratio of tunnel closure to tunnel diameter (Hoek and Marinos 2000).

The ratio of rock mass strength to in - situ stress is 0.26 and tunnel diameter is 4.5 m. For this, Fig. 7 shows the percent strain (tunnel deformation) will be about 3% giving a tunnel closure of 13.5 ~14 cm. This analysis, although very crude, gives a good first estimate of potential tunneling problems due to squeezing conditions in weak rock at significant depth below surfaces. Table 7 gives the support type for various percentage strains. Because of the very poor quality of the rock mass and the presence of significant amount of clay, the use of rockbolts or cables is not appropriate because of the difficulty of achieving adequate anchorage. Consequently,

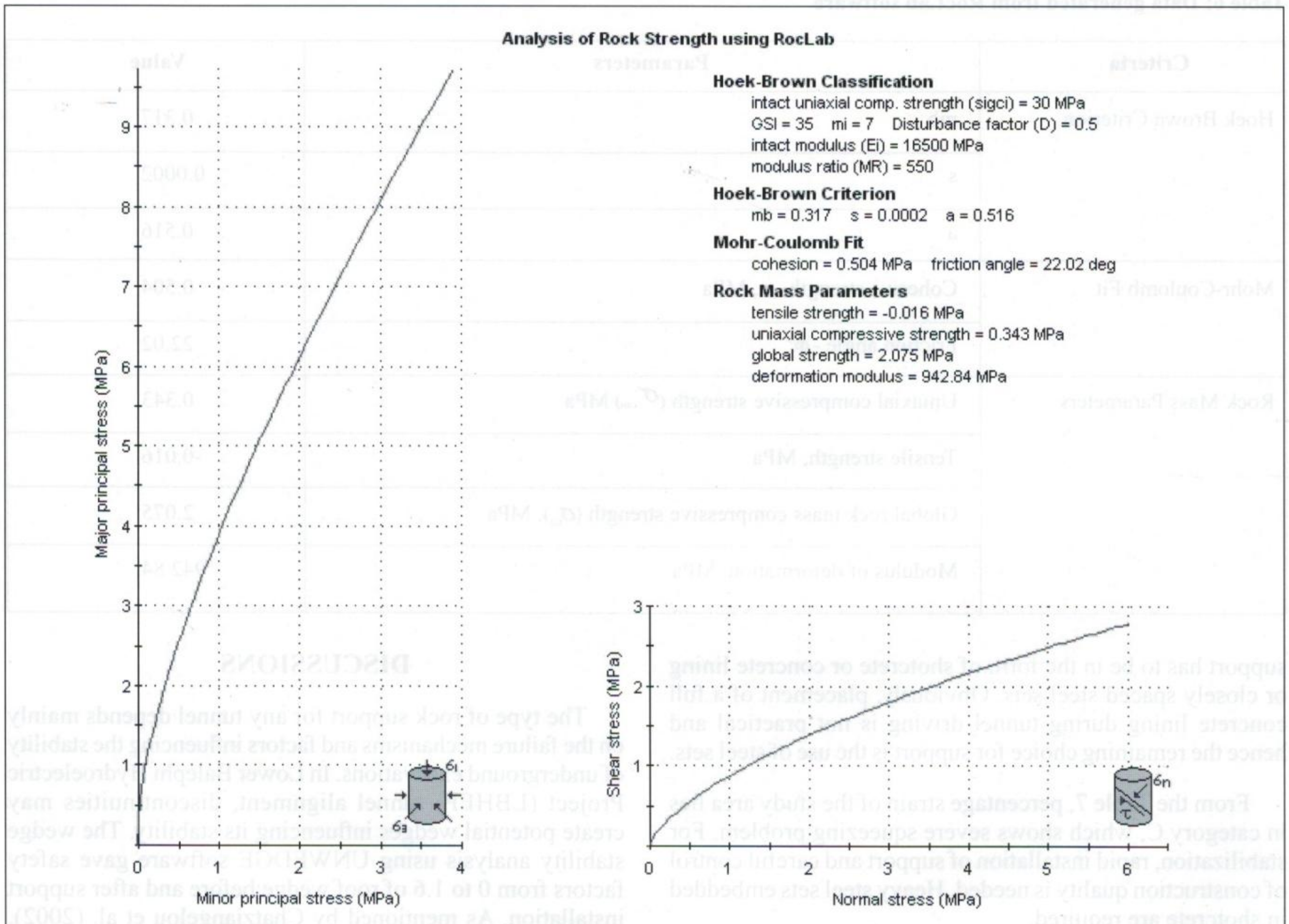

Fig. 6: Analysis of rock strength using RocLab

Table 5: Input parameters/values for calculation of rock mass strength

Input parameters	Input value	Remarks
Intact uniaxial compressive strength (Sig ci), MPa	30	Lab test
Geological strength index	35	From Roclab reference
Rock mass constant, mi	7	From table 4
Disturbance factor (D)	0.5	From Roclab reference
Unit weight of rock mass (kg/cm³)	0.027	Lab test
Application	Tunnel	
Depth to tunnel, m	300	
Intact modulus (Ei)	16,500	Ei = MRX Sig ci
Modulus Ratio, (MR)	550	From Roclab reference

Table 6: Data generated from RocLab software

Criteria	Parameters	Value
Hoek Brown Criterion	mb	0.317
	s	0.0002
	a	0.516
Mohr-Coulomb Fit	Cohesive strength - c, MPa	0.504
	Friction angle - Φ°	22.02
Rock Mass Parameters	Uniaxial compressive strength (σ'_{cm}) MPa	0.343
	Tensile strength, MPa	-0.016
	Global rock mass compressive strength (σ_{ci}), MPa	2.075
	Modulus of deformation, MPa	942.84

support has to be in the form of shotcrete or concrete lining or closely spaced steel sets. Obviously, placement of a full concrete lining during tunnel driving is not practical and hence the remaining choice for support is the use of steel sets.

From the Table 7, percentage strain of the study area lies in category C, which shows severe squeezing problem. For stabilization, rapid installation of support and careful control of construction quality is needed. Heavy steel sets embedded in shotcrete are required.

DISCUSSIONS

The type of rock support for any tunnel depends mainly on the failure mechanisms and factors influencing the stability of underground excavations. In Lower Balephi Hydroelectric Project (LBHEP) tunnel alignment, discontinuities may create potential wedges influencing its stability. The wedge stability analysis using UNWEDGE software gave safety factors from 0 to 1.6 of roof wedge before and after support installation. As mentioned by Chatziangelou et al. (2002), this study shows that the safety factors obtained from the support measures recommended by UNWEDGE are much higher than the theoretically required values.

Table 7: Relationship of percent strains with support type (Hoek and Marinos 2000)

	Strain e %,	Support types
A	Less than 1	Very simple tunnelling conditions, with rockbolts and shotcrete typically used for support.
B	1 to 2.5	Minor squeezing problems which are generally dealt with by rockbolts and shotcrete; sometimes with light steel sets or lattice girders are added for additional security.
C	2.5 to 5	Severe squeezing problems requiring rapid installation of support and careful control of construction quality. Heavy steel sets embedded in shotcrete are generally required.
D	5 to 10	Very severe squeezing and face stability problems. Forepoling and face reinforcement with steel sets embedded in shotcrete are usually necessary.
E	More than 10	Extreme squeezing problems. Forepoling and face reinforcement are usually applied and yielding support may be required in extreme cases.

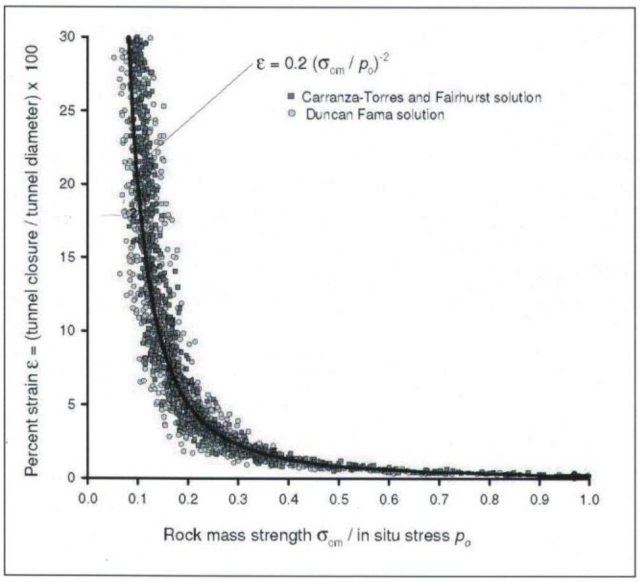

Fig. 7: Plot of percentage strain versus the ratio of rock mass strength to in situ stress (after Hoek and Marinos 2000)

The squeezing of tunnels is a common phenomenon in poor rock masses under high in situ stress conditions. The critical strain parameter is an indicator that allows the degree of squeezing potential to be quantified. It is defined as the strain level on the tunnel periphery beyond which instability and squeezing problems are likely to occur. Among various approaches for the assessment and support design for the squeezing phenomenon in underground constructions, Hoek and Marinos (2000) approach has been adopted for the analysis. The analysis shows that percent strain will be about 3% giving a tunnel closure of 14 cm.

CONCLUSIONS

The LBHEP is located in Sindhupalchock District, central Nepal. Phyllite and phyllitic quartzite are the dominant rock types in the study area and most of the tunnel alignment lies in these rocks. Underground wedge stability analysis showed that several wedges would be formed due to underground excavation and would be stabilized with the help of rock bolting and shortcreting. Maximum overburden in the headrace tunnel alignment is 300 m at chainage 2700 m from inlet portal. Squeezing problem could be occurred in phyllite rock at maximum overburden and can be stabilized by rapid installation of support with heavy steel sets and careful control of construction quality.

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