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REVIEW ARTICLE

GEOTECHNICAL EVALUATION OF POWER TUNNEL FOR VISHNUGAD-PIPALKOTI HYDEL SCHEME, GARHWAL HIMALAYA, INDIA

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ABSTRACT

The Vishnugad - Pipalkoti hydel scheme, a run-off-the-river scheme envisages the construction of a 65m high concrete gravity dam across the river Alakananda, about 1km downstream of Vishnugad near the Helong village to divert water through a 8m diameter horse shoe shaped Head Race Tunnel over a length of 13.4 km to an underground power house for generating 444MW of power near the Hat village. The project is located in Lesser Himalayan terrain, which is characterized by complicated geological setting. The rocks exposed in the project area include dolomites, slates, quartzites and chlorite schists of Pipalkoti Formation of Garhwal Group. The rocks are traversed by many types of structural discontinuities. The bedding traces seen at places in quartzites are nearly parallel to foliations. In addition, two prominent sets of joint are observed in the rocks. The intersection of these discontinuities may result in a number of stable or unstable rock wedges depending upon their orientation with respect to tunnel alignment. The terrain shows evidences of large scale thermal activities in the area as evidenced by the presence of a few thermal springs. Some of the important problems likely to be faced during the construction of the Power Tunnel are discussed. The rock mass properties were derived using RMR and Q System in order to predict rock load and support requirements.

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INTRODUCTION

The underground space technology has gained greater importance in the recent times to overcome the problems of space and to accommodate strategically important projects. The design and construction of large underground openings such as powerhouse cavities are always difficult in the seismically active Himalayan terrain having high in-situ stresses. The overall stability of underground openings is dependent on a number of factors like condition of rock mass, in-situ stresses, support stiffness, size and shape of cavity, method of construction and sequence of construction among other factors. Rock mass condition and its possible behavior during excavation help in calculating the stability of the cavity and the rock load. The in-situ stresses also play an important role in the stability of an underground opening. The main factor in the design of underground openings is to help the rock mass to support itself.

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The Vishnugad-Pipalkoti hydroelectric project is under construction in the lower reaches of Alakananda River which is a major tributary of Ganga in Garhwal Himalaya (Fig. 1). It envisages the construction of a 65 m high concrete gravity dam about 1 km downstream of Vishnugad near the Helong village to divert water through an 8m diameter horse shoe shaped Head Race Tunnel over a length of 13.4 km to an underground power house for generating 444 MW of power near the Hat village

Geological Setting

The rocks exposed at the dam site and underground power tunnel are quartzites with interbedded thin bands of chlorite schist, slates and dolomites respectively. Meta-dolomite showing long traces of bedding and widely spaced joints also shows effects of weathering. Dip and strike of the bedding or foliations and the joint planes have been taken and compiled in 1:15,000 scale. The tunnel runs in roughly NE-SE direction with four kinks. The maximum depth of rock cover above the crown of the tunnel is more than 800m near Pokhani village.

The minimum depth of cover is of the order of 30m at Maina river (nadi) crossing. The tunnel alignment crosses many perennial streams such as Tappon, Dwing, Tiroshi, Maina and Ghanpani.

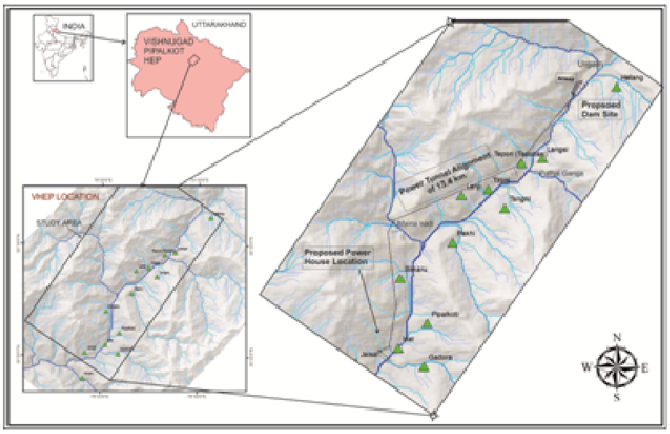


Fig.1. Location Map of the study area

The dam and the initial reach of the tunnel is located in quartzite, which is hard, grey coloured, foliated and medium to fine grained.

Further south, dolomite is exposed along the alignment. Calcareous slate is present in the area around Maina river. Further towards power house, dolomite and slate are exposed alternatively. The power house is located within dolomite. Along the entire stretch of the power tunnel high geothermal gradients can be expected (Fig. 2.). It may be confined to limited stretch. The past manifestations of the hot water springs can be seen in the rocks. The rocks, which have been affected by chemical reactions due to hot springs have lost their inherent strength and have been crumbled to ashes along certain zones. The hot springs had left significant patches of sulfur encrustations, which have been oxidized.

Thus contact of hot water had considerably reduced the strength of calcareous slates and dolomites at many places and due to which the rocks look highly fragile. Hence during excavation of tunnels, these weak rocks may collapse and cause over break conditions. Suitable control measures shall be adopted in these stretches during tunnel excavation. The geothermal gradients may be a major problem during tunnel excavation. Adequate investigations shall be carried out to tackle the geothermal problems. The major structural feature of the area is foliation.

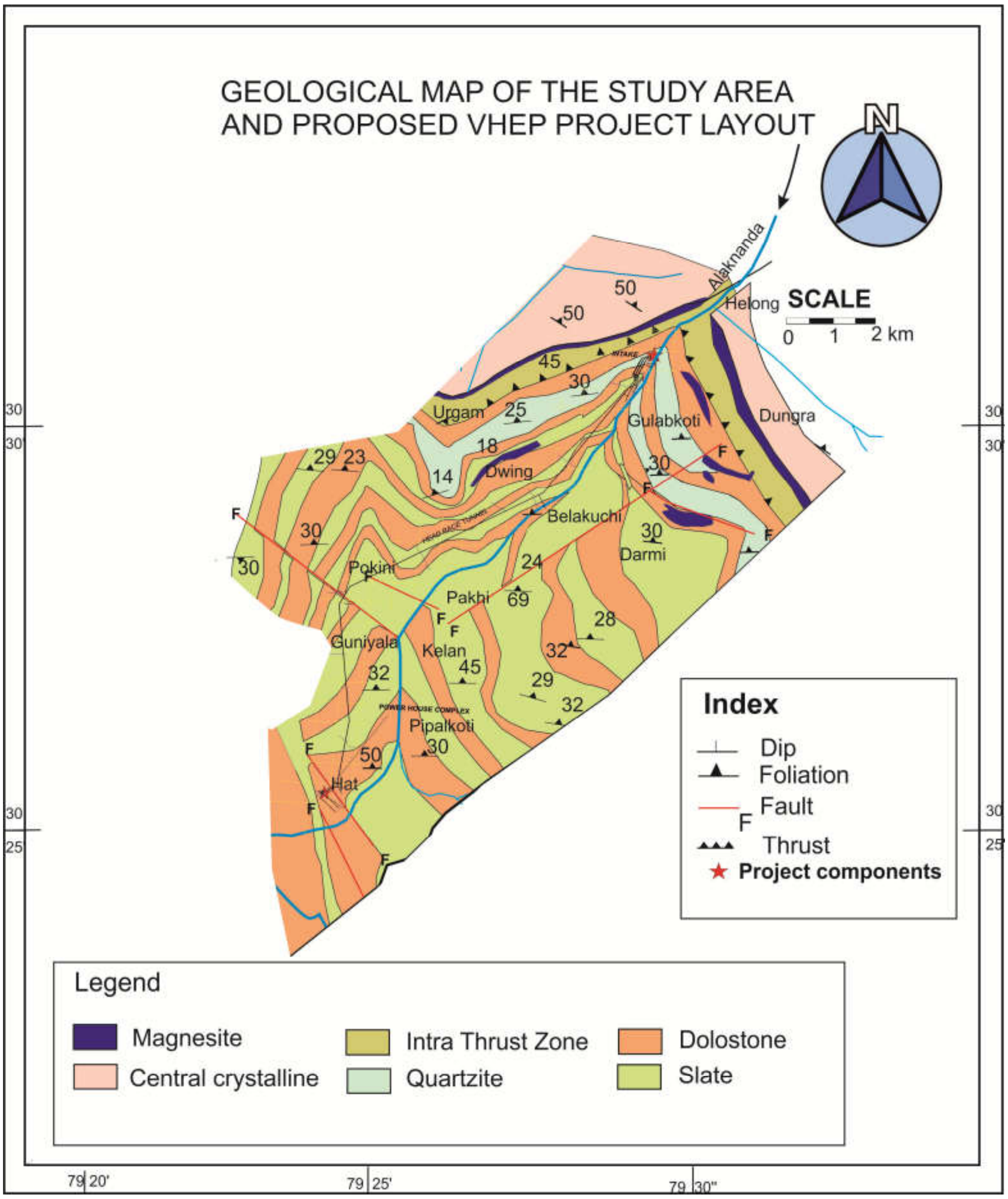


Fig. 2. Geological map of the project site

It is present consistently throughout the tunnel alignment. The rocks are also traversed by three sets of joints. The structural pattern of the area have been studied for different segments of the tunnel (Figs. 4 to 7)

Geological Section along the Head Race Tunnel (HRT) alignment

As the Head Race Tunnel (HRT) has four kinks, a kink section is prepared along the tunnel alignment in 1:15,000 scale (Fig. 3). The orientation of geological discontinuities with respect to the tunnel alignment is a major factor resulting in unstable wedges within the tunnel. The more, the geological discontinuities are parallel to the tunnel alignment, more unfavourable conditions may result during excavation. Similarly, if more than one set of discontinuities are present, the rock wedges formed may be stable or unstable depending upon the plunge of wedge line. The more the plunge direction of wedge line is parallel to the tunnel alignment the wedges may become unfavourable. This logic has been applied for the tunnel section between the intake and surge tank of the power tunnel. Since the power tunnel has many kinks, accordingly the tunnel has been divided into five segments namely A-B, B-C, C-D, D-E and E-F.

N80^{0}E - S80^{0}W with dip 40^0 towards N10^{0}W direction into the hill near village Pokhani, where there is a variation of strike of foliation plane has been observed from top level upto river bed level. Near the river course the beds are more or less horizontal having strike N70^{0}W - S70^{0}E. Above 100m from the river course there occur slate beds having strike N80^{0}W - S80^{0}E with dip 30^0 - 35^0 towards N010^{0}E. In the upstream of the Hyuna bridge at the left bank the foliation planes dip 20^0 towards N130^{0}E. Two sets of joint are present in the slates - one dipping 72^0 towards S040^{0}W and other one dipping 72^0 towards N115^{0}E. The orientation of B-C line is N60^{0}E-S60^{0}W. Here dolomite beds get flattened. Slates are foliated with the apparent dip of 18^0 towards NE direction along the section. Two sets of joint are present with apparent dip 70^0 and 60^0 towards SW and NE direction respectively (Fig. 5).

The maximum depth of overburden above the tunnel is 825m as inferred from the section. Along C-D section line, Maina river crosses, where the rock cover above the tunnel seems to be inadequate. Some debris cover also has been observed here. It may be seen that the overburden cover at the intersection between Maina river and the tunnel alignment is about 20m (Fig. 3). This may also include boulders on the top, highly weathered rock at least upto 10m from the surface.

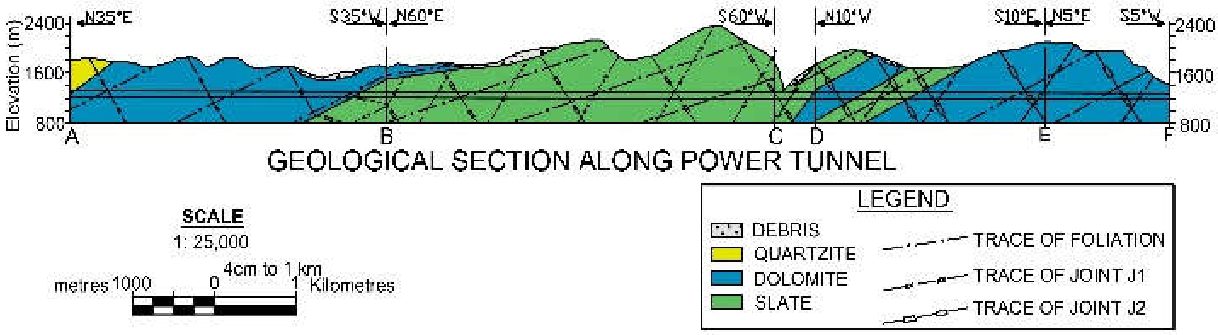


Fig. 3. Geologicalsection along Power Tunnel

The rocks through which tunneling is done, are traversed by two prominent sets of joints in addition to random joints and the consistent bedding and foliation planes. The foliation planes very often tend to merge and hence are not continuous many times. However, the foliation joints show considerable strike continuity, indicating that they may be persistent along dip direction also, over considerable length. So the intersection of these discontinuities may result in a number of rock wedges. These wedges may be stable or unstable depending upon their orientation with respect to tunnel alignment.

The orientation of A-B line is N35^{0}E-S35^{0}W. This segment of HRT encounter dolomite beds having strike N70^{0}W with dip 42^o dip towards N20^{o}W direction. Near Longsi Bridge along the project road grayish black slates are exposed. The foliation planes of slates are dipping 35^0 – 40^0 towards N010^{0}W into the hill. Joints are present in dolomites dipping 60^0 towards S020^{0}W. In slates, joint planes dip 80^0 towards S055^{0}W. Dolomite beds are exposed with apparent dip of 32^0 towards NE. One set of joint is present in the section having apparent dip 60^0 towards SW (Fig. 4). The B-C segment of the HRT passes through slates having strike of the foliation plane is

This may leave fairly fresh to fresh rock cover of about 10m above the tunnel. Presence of a shear zone along the stream course can't be ruled out altogether. As such the excavation of tunnels in this area may pose major excavation problems particularly for the stability of the tunnel. In D-E segment, from Maina river upto Math village thinly foliated grayish black slates are exposed with some minor quartz vein and thick vegetation cover. Some patches of dolomites are also present. The contact between slate and dolomite is not sharp but transitional. The foliation plane in slate strike N 45^{0}E - S45^{0}W with average dip 38^0 towards N315^0. The joints present in slates dip 60^0 towards N040^0 and 66^0 towards N230^0. Slates also show the effect of sulfur encrustation at places, where they are exposed to the atmosphere. The orientation of D-E section line is N10^{0}W - S10^{0}E. Two sets of joint have apparent dips 44^0 and 48^0 towards SE and NW direction (Fig. 5)

The E-F segment of the HRT passes through the dolomite beds with same attitude. These dolomites are brownish in colour showing the effect of weathering. Two sets of joint are present in dolomites with dip 66^0 towards N230^0 and 60^0 towards N040^{0}E and apparent dip 54^0

towards NE direction and 56^0 towards SW direction (Fig. 6). The orientation of the E-F section line is $N5^0E$-$S5^0W$. Dolomite beds are exposed with apparent dip 28^0 towards NE direction. Two sets of joint is present in this section.

- Rock mass classifications in power tunnel

The slate and dolomite rock mass in power tunnel area of Vishnugad-Pipalkoti Hydel scheme has been classified using

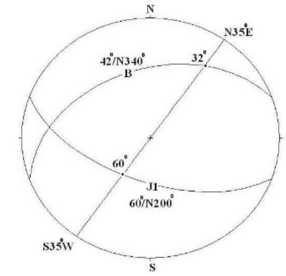


Fig. 4.Stereoplot for A-B Section

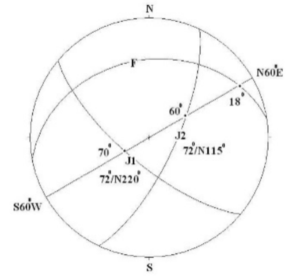


Fig. 5.Stereoplot for B-C Section

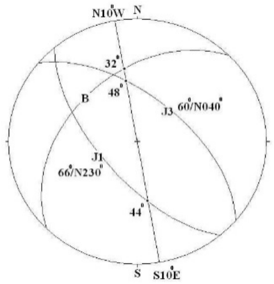


Fig. 6.Stereo-plot for D-E Section

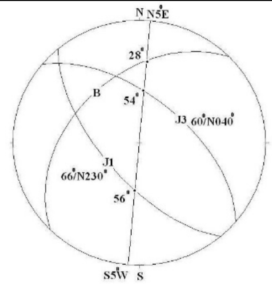
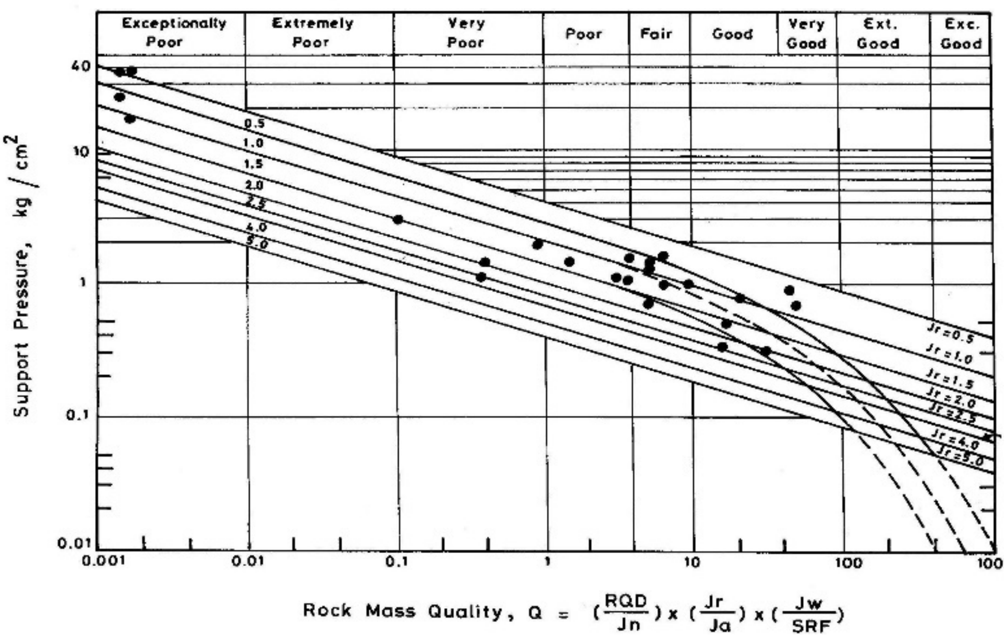


Fig. 7.Stereo-plot for E-F Section

Table 1. Classification of Slate and Dolomite Rock Mass in the Power Tunnel Area Q-system (Barton *et al.*, 1974)

Rock Mass Rating			Rock Quality Index		
Parameters	Slate	Dolomite	Parameters	Slate	Dolomite
Strength of intact rock	12	15	RQD (%)	64	80
RQD	13	17	Joint set No. (J_n)	6	6
Spacing of joints	8	10	Joint roughness No. (J_r)	3	3
Condition of joints	25	30	Joint alteration No.(J_a)	1	1
Groundwater condition	15	15	Joint water reduction factor (J_w)	1	1
Basic RMR value	73	87	Stress reduction factor (SRF)	2.5	2.5
Class	II	I	$Q=\left(\frac{RQD}{J_n}\right)\times\left(\frac{J_r}{J_a}\right)\times\left(\frac{J_w}{SRF}\right)$	12.8	16
Classification	Good	Very Good	Classification	Good	Good



rock mass rating (RMR) system (Bieniawski, 1979) and Q-system (Barton *et al.*, 1974). The Significant contribution that Hock and Brown made was to link the equation to geological observation in the form of Bieniawski's Rock Mass Rating. (Hock and Brown, 7982) The RMR system has been mainly used for assessing shear strength parameters, which are useful for computing the stability of wedges in the rock mass. The Geomechanics classification of Bieniawski provides guidelines for selection of rock reinforcement for tunnels. The Q system has been used to assess rock pressures and support requirements. The values of RMR and Q in the power tunnel area are shown in Table 1.

Evaluation of Support Pressure

Using the measured support pressure values from instrumented Indian tunnels, Goel and Jethwa (1991) have proposed the following Equn. (1) for estimating the short-term support pressure for underground openings in both squeezing and non-squeezing ground condition in the case of tunneling by conventional blasting method using steel rib support (but not in rock burst condition).

$$P_v = \frac{\left(7.5B^{0.1}H^{0.5} - RMR\right)}{20\left(RMR\right)} \text{ Equn} \qquad \ldots\ldots\ldots\ldots\ldots(1)$$

The Q system developed by Barton *et al*. (1974) is one of the widely used empirical approaches all over the world for choosing support system for underground excavations. They modified Q system by introducing the term ultimate support pressure and short term support pressure and plotted support capacities of 200 underground openings against the rock mass quality (Q) as shown in (Fig. 8). They found the following empirical correlation for roof and wall pressures.

$$P_v = \left(0.2/J_r\right)Q^{-1/3} \text{ Equn} \qquad \ldots\ldots\ldots\ldots\ldots(2)$$

$$P_h = \left(0.2/J_r\right)Q_w^{-1/3} \text{ Equn} \qquad \ldots\ldots\ldots\ldots\ldots(3)$$

Where,

P_v = Ultimate roof support pressure in MPa,
P_h = Ultimate wall support pressure in MPa,
Q = Rock mass quality (Eq 1), and
Q_w = Wall rock mass quality

It may be noted that dilatant joints or J_r values play a dominant role in the stability of underground openings.

The wall factor (Q_w) is obtained after multiplying Q by a factor which depends on the magnitude of Q as given below:

Range of Q	Wall Factor Q_w
> 10	5.0 Q
0.1 – 1	2.5 Q
< 0.1	1.0 Q

Barton *et al*. (1974) further suggested that if the number of joint sets is less than three, then

$$P_v = \frac{\left[0.2\left(J_n\right)^{1/2} \times Q^{-1/3}\right]}{3\left(J_r\right)} \text{ Equn} \qquad \ldots\ldots\ldots\ldots(4)$$

$$P_h = \frac{\left[0.2\left(J_n\right)^{1/2} \times Q_w^{-1/3}\right]}{3\left(J_r\right)} \text{ Equn} \qquad \ldots\ldots\ldots\ldots(5)$$

They felt that the short- term support pressure can be obtained after substituting 5Q in place of Q in Equn. (2). Thus the ultimate roof support pressure is obtained as 1.7 times the short term support pressure.

Modifications in q-system by Singh *et al.* (1992)

Singh *et al.* (1992) have actually compared the measured support pressure values with the support pressure values estimated by Eq. (2) of Barton *et al*. (1974) in the Himalayan tunnels. They have observed that the support pressure estimated from the Barton's approach is unsafe in case of Himalayan tunnels which have a high overburden pressure. Based on their experiences, in the Himalayan tunnels, they proposed a couple of correction factors in Barton's equation to propose new equations for estimating support pressure:

$$P_v\left(ult\right) = \frac{\left(0.2 \times Q^{-1/3} \times f \times f'\right)}{J_r} \text{ Equn} \qquad \ldots\ldots\ldots\ldots(6)$$

It is measured in Mpa

$$f = \frac{1+\left(H-320\right)}{800} \geq 1 \text{ Equn} \qquad \ldots\ldots\ldots\ldots(7)$$

Table 2. Correction factor 'f' for Tunnel Closure

Sl. No	Rock Condition	Support System	Tunnel Closure (u_a/a)%	Correction factor (f')
1	Non-squeezing (H>350Q^{033})	--------	<1	1.1
2	Squeezing (H>350Q^{033})	Very stiff	<2	>1.8
3	Squeezing (H>350Q^{033})	Stiff	2-4	0.85
4	Squeezing (H>350Q^{033})	Flexible	4-6	0.70
5	Squeezing (H>350Q^{033})	Very Flexible	6-8	1.15
6	Squeezing (H>350Q^{033})	Extremely Flexible	>8	1.8

Note: Tunnel closure depends significantly on the method of excavation. In highly squeezing ground conditions, heading and benching method may lead to tunnel closure > 8%.

Tunnel closures more than 4% of tunnel span should not be allowed, otherwise support pressures are likely to build-up rapidly due to failure in the rock arch. In such cases, additional rock anchors should be installed immediately to arrest the tunnel closure within a limiting value.

Steel ribs with struts may not absorb more than 2% tunnel closure. Thus slotted Steel Fibre Reinforced Shotcrete is suggested as an immediate support.

Where,
Q = Rock mass quality
P_v (ult)= Ultimate tunnel support pressure
f = Correction factor
f''=Correction factor for tunnel closure (Table 2)
H = Overburden above crown or tunnel depth below ground level (m)

Singh *et al*. (1992) have also studied the effect of tunnel size (2m - 22m) on support pressures. They inferred no significant effect on observed support pressure.

Horizontal or wall support pressure

For estimating wall support pressure following equation is used

$$P_v\left(ult\right) = \frac{\left(0.2 \times Q_w^{-1/3} \times f \times f^{'}\right)}{J_r} \text{ Equn} \quad ....\ldots\ldots(8)$$

The wall factor (Q_w) is obtained after multiplying Q by a factor which depends on magnitude of Q as given below:

Range of Q	Wall Factor Q_w
>10	5.0Q
0.1-10	2.5Q
<0.1	1.0Q

For using this theory data must be there and for that purpose required data is collected from field study.

Using approach of Singh *et al.* (1992)

The modified equation of Singh is used in estimating support pressure Equn. (5)

$$D_e^{'} = 2\left(Q^{0.4}\right) meters \text{ Equn} \quad \ldots\ldots\ldots\ldots..(9)$$

If H < 350 Q $^{1/3}$ meters

Where

$$D_e^{'} = \frac{Span}{Excavation\ Support\ Ratio\left(ESR\right)} \text{ Equn} \quad \ldots\ldots\ldots\ldots(10)$$

D_e'= Equivalent dimension
Span = Diameter or Height of tunnel in meters
ESR = Excavation support ratio

Table 6.Values of excavation support ratio (Barton *et al*., 1974)

Length of bolts and anchors

Sl. No	Type of excavation	ESR
1	Temporary mine opening, etc	3-5
2	Vertical shafts:	
	circular section	2.5
	rectangular/square section	2.0
3	Permanent mine openings, water tunnels for hydro power (excluding high pressure penstocks), pilot tunnels, drifts and heading for large excavations, etc	1.6
4	Storage rooms, water treatment planets, minor road and railway tunnels, surge chambers, access tunnels, etc.	1.3
5	Oil storage caverns, power stations, major road and railway tunnels, civil defence chambers, portals, intersections, etc.	1.0
6	Underground nuclear power stations, railway station, sports and public facilities, factories, etc.	0.8

Table 4. Correction factor for overburden f and tunnel closure f' by using approach of Singh *et al.* (1992)

Rock Type	Depth of overburden (H) in m	Correction for overburden f=1+(H-320)/800≥1	Rock Condition	Tunnel Closure (ua),%	Correction factor(f')
Slate	495	1.218	Non-Squeezing H<350Q $^{0.33}$	<1	1.1
Slate	600	1.350	Non-Squeezing H<350Q $^{0.33}$	<1	1.1
Slate	825	1.631	Non-Squeezing H<350Q $^{0.33}$	<1	1.1
Dolomite	375	1.068	Non-Squeezing H<350Q 0.33	<1	1.1
Dolomite	495	1.218	Non-Squeezing H<350Q 0.33	<1	1.1
Dolomite	450	1.162	Non-Squeezing H<350Q 0.33	<1	1.1

Table 5. Support pressure using equation of Singh *et al.* (1992)

Rock Type	Depth of overburden (H) in m	Q_{av}	Ultimate Roof Support Pressure By Singh *et al*. (1992) P_v (ult) (Mpa)	Ultimate Wall Support Pressure By Singh *et al*. (1992) P_h (ult)(Mpa)
Slate	495	12.77	0.03877	0.02277
Slate	600	12.77	0.04298	0.02524
Slate	825	12.77	0.05193	0.03053
Dolomite	375	16.07	0.03132	0.01842
Dolomite	495	16.07	0.03572	0.02100
Dolomite	450	16.07	0.03408	0.02003

For present purpose the excavation span = 8 m

Estimation of Support Requirement

Determination of Maximum Unsupported Span

Barton *et al.* (1974) proposed the following equation for estimating equivalent dimension (D_e') of a self supporting or an unsupported tunnel.

- Length of bolts and anchors

Bolt length is determined by the following equation given by Barton *et al*. (1974)

Table 7.Estimation of support by Q-System

Rock Type	Q_{av}	Conditional Factors		Span/ESR (m)	Type of support	Support category
		RQD / J_n	J_r / J_n			
Slate	12.77	10.89	0.5	5	B(utg) 1.5-2 m	13
Dolomite	16.07	13.39	0.5	5	B(utg) 1.5-2 m	13

Note: B = systematic bolting;
(utg) = untensioned grouted

$$L_b = 2 + 0.15 D_e^{'} \text{ Equn} \qquad \text{..............(11)}$$

Where,

L_b = Bolt length (m)

Anchor relation is given by the following relation

$$L_a = \frac{0.4D}{ESR} \text{ Equn} \qquad \text{...............(12)}$$

The spacing between the anchors is taking as half the length of anchor.

In the studied area the span or diameter of tunnel (D) is 8 m and taking ESR = 1.6 (Table 6), we can get the value of D_e' from Equn. (10)

Hence

$$D_e^{'} = \frac{8}{1.6} = 5$$

Putting the value of De' in Equn. (11) we get,

$$L_b = 2 + (0.15 \times 5) = 2.75m$$

Thus the length of the bolt work out is 2.75 m in an opening with 8 m width.

The length of the anchor

$$L_a = 0.4 \times 5 = 2.0m$$

As the anchor spacing is half of the anchor length. Thus the anchor spacing will be 1 m.

Types of support by Q-System

Support system has also been evaluated by Q-System (Table 7)

Conclusion

The Vishnugad-Pipalkoti hydel scheme envisages the construction of 65m high concrete gravity dam to divert water through a 8m diameter horse shoe shaped Head Race Tunnel over a length of 13.4km to an underground power house of 444 MW capacity. The power tunnel may face the problems of over break due to orientation of discontinuity and hot water spring. Thus suitable measures should be adopted in these stretches during tunnel excavation. The rock load and the required support measures were studied using RMR and Q system. The quality of rock mass has been accessed by RMR method of Bieniawski (1979) and Q system by Barton (1974). For support pressure from RMR approach of Goel and Jethwa (1991) and for Q system approaches of Barton *et al*. (1974) and its modified version by Singh *et al*. (1992) have been used. Both the methods indicate that the quality of rock mass is good to very good. The support pressure estimation indicates that the rocks fall in support category 13, where a spacing of 1m for bolts and anchors have been suggested along with untensioned grouting.

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