Tunneling through phyllitic rocks - A case study of 12 km long water conducting tunnels of Dewas –II project, Udaipur, Rajasthan, India

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Abstract

The Dewas-II project, the second part of a four stage inter basin water diversion scheme envisages construction of two dams and two tunnels of 11.26 km cumulative length. Five shafts of 9 m diameter have been sunk to facilitate the tunneling operation.

The tunnels pass predominantly through monotonous flysh metasediments comprising predominantly phyllites and schists with bands of quartzite and minor calcareous rocks along with metavolcanics represented by talc-chlorite/chlorite schist and minor basic/ultra basic intrusives of Aravali Super group. The rocks are disposed as moderately plunging steeply inclined tight to isoclinally folded sequence. Tunnelling through anisotropic rocks like phyllite and schist with intrusive bodies is a challenging task with many geological surprises viz. chimney formation, heavy water discharge from fractured/shear zones etc. Over breaking is also a common feature in phyllitic rocks. The weathering/disintegration of schist and phyllite bring in development of minerals like chlorite, sericite, etc. in contact zones, which reduces the shearing strength of the rockmass to a great extent. The factors affecting the strength of the phyllitic rocks due to disintegration by weathering/alteration is discussed in detail in this paper. Evaluation of RQD of anisotropic rocks has been a topic of discussion in the recent past. A new approach utilizing grades of weathering is proposed to arrive at representative RQD of anisotropic rocks. The basic/ultra basic intrusives are quite prone to alteration, and when intersected in the tunnel need to be handled carefully. One very big chimney of about 15 m height has been formed in such a zone, and to deal with this had been a tough task for the project engineers.

The applicability of Terzaghi's rock load theory (1946), Rock Mass Rating (RMR) of Bieniawski (1984), Rockmass Quality Index (Q) of Barton (1974) and later modifications by different workers for rockmass characterization and design of support requirement for anisotropic rocks using conventional drilling and blasting techniques is also discussed in this paper. Photogeological studies of the project area and detailed surface mapping are very useful tools in recognizing weak features, which can be projected at tunnel level and helps in prognosticating the sensitive zones to be intersected. Adequate precautions thus could be planned prior to/during excavation well in advance.

1. Introduction:

To meet the ever increasing demand of water and feeding the incapacitated lakes of the of Udaipur city, a four stage inter basin water diversion scheme from Sabarmati basin to Berach basin (Figure 1) under gravity flow has been contemplated. The Dewas-II project, the second stage nearing completion envisages construction of two dams and two tunnels comprising about 1.21 km long Madri link tunnel and 11.05 km long Akodara tunnel having 12.26 km cumulative length. Five shafts of 9 m diameter are sunk to facilitate the tunneling operation; one for Madri link tunnel and four for Akodara tunnel. The size of 'D' shaped tunnel of the Madri link tunnel is 3.00 m x 3.50 m with excavated height of 4.14 m and that of main Akodara tunnel is 5.75 m x 5.75 m with excavated span of 6.10 m.

The area under construction of the project is characterized by mostly hilly terrain of medium to high hills with a maximum cover of about 416 m above tunnel invert level. The highest ridge in the middle reach forms part of water sheds for both the Sabarmati and Berach basin. The water of Sabarmati basin with scanty population, which is otherwise debouching in the Arabian Sea without much domestic use, is being transferred to Berach basin feeding many lakes of highly populated areas of Udaipur City of Rajasthan.

2. Geological set up of the area:

The area constitutes one of the most stable landmass of Proterozoic age falling in the **Zone–II** of seismic map of India. The tunnel passes through monotonous flysh metasediments comprising mainly phyllite and schist with bands of quartzites, minor calcareous rocks along with completely altered metavolcanics represented by talc-chlorite/chlorite schist and basic/ultra basic intrusives belonging to Aravali Super group. About 80% of the tunnel passes through phyllitic rocks. The rocks trend NNE-SSW with steep dip (70^o-80^o) towards WNW. The most pervasive foliation (S2) developed during most intense second phase of deformation has almost same strike with slightly shallower dip (50^o-65^o) in the same direction. The rocks have under gone polyphase deformation and exhibit progressive increase in the grade of metamorphism from low green schist facies in the east to low grade amphibolite facies in the west forming moderately plunging, tight to isoclinally folded sequence (Gupta et al, 1997)(6).

The trend of rocks is almost parallel to the tunnel alignment in Madri link tunnel (unfavorable disposition) and across the alignment making an angle of 70° in case of Akodara tunnel (favorable disposition) with steep dips in WNW to WSW direction. The phyllite displays variation in composition with intercalatory quartzite bands, which are thicker in the western and central part of the tunnel (Chandra Madhav, 2006). The calcareous/dolomitic bands are confined to the eastern distal part of the tunnel. As the composition of phyllite vary considerably and so are the strength and other geotechnical parameters. Besides the most pervasive S_1 foliation, the schistose rocks are beset with three sets of joints; sub vertical cross joint set and two oblique sets of joints. The concentration and attitude of these joints vary from place to place due to later phases of deformation. A number of shear zones, mostly thin with clay fillings, running parallel to the foliation in schistose rocks as well as cutting across it at different angles are the most important structural features requiring attention. A few thick shear zones measuring up to 75 cm have also been intersected in the tunnel characterized by silt/clay mixed fractured rock.

3. Factors affecting evaluation of strength of phyllitic rocks:

The phyllite, an anisotropic rock, consisting predominantly of quartz, fine mica (muscovite/sericite) and chlorite forms very good tunneling media when fresh and favorably disposed with respect to tunnel alignment but when weathered, becomes poor to very poor rockmass. The understanding of prevailing weathering process is of vital importance, as this will throw some light over intensity and depth of weathering of the intact rock as well as along the discontinuities, which has direct implications on the geotechnical attributes of the rockmass. In the present study area, the fresh rock is available at very shallow depth on the slopes of the hills, whereas in the valley portions the depth of weathering is considerably more than expected at many places. Moderately to highly weathered phyllite has been noticed up to 50 m depth in the vicinity of a highly altered intrusive body in a portion of the tunnel section passing through valley portion, which resulted into formation of 15 m high chimney at one place. Besides, joints and foliation with varying degree of weathering, there are a number of shears within phyllites with formation of gouge material traversing the rock at many places.

In the present area of semi arid climate, water flowing in ephemeral streams and high diurnal variation are the predominant weathering agents, which results into opening of joints/foliation and formation of micaceous clayey/silty soil. The weathering action loosens up the phyllite along foliation, thereby reducing the RQD (Rock quality designation) of the rockmass to a great extent. A moderately weathered phyllite will have almost nil RQD in drilled cores. The foliation can not be counted as discontinuity, as there is no distinct break across the foliation; rather it is just parallelism of platy/flaky minerals. Minor alteration of the flaky minerals forms innumerable number of planes along which the rock splits easily, which is otherwise not the case in fresh rock. Thus, the RQD of the phyllite should be assessed very judiciously while calculating the 'Q' and RMR values. In the anisotropic rocks like phyllites and schists the joints/discontinuities counted on different planes has drastically different RQD e.g. moderately weathered phyllite on a plane perpendicular to the foliation will have nil RQD, whereas on the plane parallel to the foliation gives very good RQD. The RQD measured by core drilling may give deceptive picture of the strength of the anisotropic rocks. In this context, Palmström (2006) has rightly said "*RQD can often result in a sampling bias due to a preferential orientation distribution of discontinuities and that the RQD values are a function of the total frequency, which is highly sensitive to sampling line orientation".* This statement applies pretty well for the anisotropic rocks. The weakness of the anisotropic rock is due to the low shearing strength along the preferred orientation of platy/flaky minerals, which is quite sensitive to the degree of weathering; otherwise the fresh anisotropic rocks are amply competent for any type of civil structures.

4. A new approach for RQD evaluation:

To resolve the problem of evaluating RQD for anisotropic foliated rocks and rocks with preferred orientation of closely spaced incipient joints/fractures, it is proposed that the RQD should always be deduced from joint count *(RQD = 115 - 3.3 Jv)* on rock exposures (Palmström, 1974) and should be given a correction for degree of weathering of intact rock (W_0, W_1, W_2, W_3 etc.) discarding the closely spaced foliation plane/joint as discontinuity. RQD_{ca} (corrected RQD for anisotropic rocks) can be obtained by dividing rating for W_0, W_1, W_2 and W_3 stages of weathering, which the author tentatively proposes as 1, 2 and 4 and 7 respectively, which may be modified based on experimental scrutiny. The description of the weathered classes also should be redefined with respect to anisotropic rocks like the ease with which the rock splits along preferred orientation of minerals/planes. The correction for W_4 and W_5 stages need not be given as the RQD of such rock masses will always be zero and lowest rating should be taken for evaluating Q and RMR values. The rating for weathered/altered intact rock has not been considered anywhere in evaluating 'Q' from various parameters. The RQD_{ca}, would also cover the compressive strength of the intact rock (σ_{ci}) to some extent, the factor, which Barton (2002)(2) has later considered to arrive at modified/corrected Q value, 'Q_c' (Q_c=Q x σ_{ci}/100). The validity of incorporating the factor (σ_{ci}/100) by Barton has been discussed by Ramamurthy, (2008) and found it unacceptable. Moreover, it is very difficult to prepare a sample of weathered anisotropic rock for UCS determination. In that case the proposed RQD_{ca} for anisotropic rocks may be quite useful factor for determining 'Q" value instantaneously in the field itself. This approach should be restricted to anisotropic rocks like phyllite, schist and rocks with closely spaced pervasive incipient (inherent) joints only.

5. Methodology of rock mass characterization:

The dominant rock types exposed along the tunnel alignment are phyllite (PH), chlorite schist (CS), thickly bedded/massive quartzite (TMQ), schistose quartzitic phyllite (SQP) and subordinate siliceous dolomite (SD). These rocks have been classified into five categories based on the geotechnical parameters observed on the surface as well as 3D logging of about 2/3 part of the tunnel. The categories are i) weathered rock with low inter block coherency ii) fresh rock with one plus one random sets of joint iii) fresh rock with two plus one random sets of joints iv) fresh rock with shears having <10cm thickness and v) fresh rock with shears having >10cm thickness (Sharma, V.P. 2008). The first category of rockmass is confined to the portal areas (inlet, outlet and portals of cut and cover portions) and the tunnel sections crossing local streams.

According to the Terzaghi's (1946) rock load classification the strata can be classified as hard and stratified/schistose rock to moderately blocky and seamy in major part of the tunnel. The thickness of the overburden over tunnel (Figure 1) varies from 15 m to 415 m. About 50 % of the tunnel section falls in VII category (squeezing ground-moderate depth) of rock classes.

The most widely accepted Rock Tunneling Quality Index ('Q') of Barton et al (1974), Rock Mass Rating (RMR) of Bieniawski (1989) for all five categories have been calculated. The ranges of 'Q' and RMR, modulus of deformation and support pressure (*pv*) for 2 + 1 random sets of joints, the most common rockmass in each category are given in table 1. These values will be drastically reduced for thick shear zone and blocky incoherent rock mass up to 0.55 and 0.06 respectively for water charged chlorite schist, the weakest rockmass. Site specific variations are of course observed during construction stage investigation. The ground water occurs in water table condition (5 m to 30 m depth) in joints, fissures and solution channels. As the maximum thickness of the overburden is about 400 m the height of water column tends to increase as the relief increases affecting the joint water reduction factor (J_w). Heavy discharge of water has been noticed from the quartzite bands with high overburden in the middle portion of the tunnel from open joints/fractures, which require continuous pumping.

Table-1

Rock type	'Q' value	RMR (Observed)	Modulus of deformation (E_d) * GPa (Average)	Support pressure (MPa)			
				$p_{v\ (el)}$** Non squeezing	$p_{v(sq)}$** Squeezing		
				H-150 m	H-150 m	H-250 m	H-400 m
CS	1.54-4.67	41 - 60	2.49	0.07	0.25	0.81	1.99
PH	1.76-5.33	50 - 65	2.68	0.07	0.27	0.91	2.35
TMQ	5.28-16.00	65 - 84	5.25	0.03	0.56	3.25	12.67
SQ/PQ	4.95-15.00	50 - 70	4.26	0.03	0.53	3.25	12.67
SD	5.28-16.00	55 - 70	4.59	0.03	0.56	3.25	12.67

*Computed from correlation equations (Barton, 2002)

** Computed from correlation equations (Goel et al.,1995a; Singh et al., 1997)

All five shafts were sunk through phyllite. The rock mass quality of walls (Q_w) computed for shafts for different categories of rock mass vary from 0.22 to 26.67 for dry conditions and 2.3 to 17.6 for water charged rockmass. The average Q_w value may be considered as quite safe for designing the support requirement. The self supporting diameter of the shaft in this rock mass computed from ESR (Excavation Support Ratio) and Q_w is about 13 m (Barton et al, 1974). The long term support pressure calculated from the equation (Singh et al, 1992) ranges between 0.30 MPa for rock with thick shear zones to 0.07 MPa for fresh rock with few joints.

6. Features requiring special attention:

The 3D logging of about 7.83 km tunnel section of Akodara tunnel and about 0.3 km section of Madri tunnel revealed some crucial zones requiring special attention during tunneling viz. weak rockmass below local ephemeral streams, clay coated cross joints parallel to the tunnel alignment, shallow dipping joints in the crown portion, cavities in siliceous dolomite, thick shear zones, blocky incoherent rockmass and unfavorable disposition of the Madri link tunnel i.e. parallel to the trend of the foliation of phyllite.

The water flowing in the local streams percolates down through the joints and foliation of the phyllitic rocks and deteriorates the quality of rockmass to a great extent. All along the alignment such type of deteriorated rockmass has been noticed in many parts of the valley portions. Such sections were treated with adequate temporary as well as additional permanent support. Grouting of the rockmass prior to tunneling in the distressed zone above crown in the tunnel sections below stream portion was suggested to improvise the strength of the rockmass.

One such surprise in the form of a big chimney in a section at RD-1940 m in the hand shake zone has become a challenging task to pass through for the engineers. The material was falling intermittently without giving sufficient time to take remedial measures. The falling material consists of soil and relict pieces of phyllite and weathered basic/ultra basic intrusive body. The XRD analysis of the soil of the muck indicated fine quartz with appreciable quantity of clinochlore and nontronite minerals, which reduces the shearing strength of the rockmass. The feldspar content of the ultra basic/basic rock has been completely altered to kaolinite in some of the relict pieces collected from muck. This zone has an overburden thickness of 54 meters. The height of the chimney as determined from the borehole drilled on the surface is >15 m from crown level and about 3000 m^3 of muck falling from the chimney has been removed.

Clay coated sub vertical cross joints sub parallel to the tunnel alignment (Figure 3) were also an important feature requiring proper attention. Incidences of falling overhung chunks along cross joint have been noticed at many places. Proper rock bolting of such overhung rockmass well in time is essential to avoid occurrence of any such incidents. These joints to some extent also affect the excavated profile of the tunnel causing over breaks in the sections having predominance of such joints (Figure 3). Shallow dipping joints in the crown portion (Figure 3) are also a matter of concern, which in combination with other sets of joints forms vulnerable wedges. The orientation of foliation/foliation joint plays important role in tunneling. As the Akodara tunnel is being driven in both directions through the shafts for faster progress, it has been observed that the effect of orientation (Bieniawski, 1984) of foliation/foliation joint is quite evident in the present case. Tunnel sections driven with dip produced less of over breaks than the sections driven against the dip in phyllitic rocks.

Figure 2 Loose muck flowing from the 15 m high chimney at RD-1940 m

Figure 3 Over breaking along shallow dipping joint in the crown portion and Sub-vertical cross joint on the right wall

An attempt has been made to project the weak features like shear zone and master joints likely to be encountered in the tunnel with the help of geocoded aerial photographs of the area by delineating the linear features and transferring them on the tunnel alignment and up to tunnel level. The back analysis after logging about 2/3 portion of the tunnel reveal that the master joints/shear zones intercepted in the tunnel have good match with the projected weak zones (Figure 1). Some of the projected weak features may die out well above the tunnel level. Nevertheless, such exercise will be of immense help for prognosticating the likely positions of distressing zones in the tunnel. Adequate precautions thus can be planned prior to/during excavation well in advance. The design of support requirement in different sections of the tunnel can also be assessed by identifying likely chances of intersecting weak zones for estimation purpose.

7. Permanent *vis a vis* temporary support:

The surface observations and synthesis of the data generated during 3D logging of 2/3 portions of the tunnels reveal that the rock mass quality in the tunnel varies from place to place but within certain limits i.e. poor to fair in general for most of the part and very poor in about 20% to 30 % of the tunnel length, where weak zones are encountered. The support categories on the basis of 'Q' values (5Q for temporary support) and equivalent dimension (De) of the tunnel are 1 to 4 (after Grimstad and Barton, 1993)

For major part of the tunnel and accordingly rock bolting and shotcreting with/without wire mesh will be required as temporary support. Beside this, steel rib support at all the portals and in the crucial zones having cavities and major shear zones due to very poor rock mass condition is warranted as additional permanent support. About 20% to 30% of the tunnel section may be left unsupported owing to higher 'Q' values. The entire length of the tunnel is to be lined with 30 mm concrete with nominal reinforcement. For hard and stratified/schist rock in the major part of the tunnel a rock load of 4.1 tones/m^3 and for moderately jointed and seamy rock 7.8 tones/m^3 are taken for design of permanent lining support. The reinforcement has to be increased in the zone of squeezing conditions and other crucial zones with installation of additional permanent support. The support pressure (pv) as calculated above may be taken into consideration for designing additional permanent support. The shafts as such do not have much problem. Only the portions with down dipping joints require systematic rock bolting. In the remaining 3/4th portion, spot bolting of the overhung rock mass along with shotcreting of the shear zones and loose rock mass with/without wire mesh will serve the purpose of safety.

In Madri link tunnel systematic rock bolting in the crown portion throughout the length of the tunnel besides shotcreting of loose fragmentary rockmass is the support requirement in general. Steel rib support with backfilling and lagging is the desired support in crucial zones. All the portals also require steel rib support at least in initial 5-10 m section as the 'Q' values are drastically reduced.

The role of a geologist during construction stage investigation of the tunnel is to suggest rockmass condition, which is now possible with fair degree of accuracy quantitatively for selection of temporary support as well as additional permanent support, whenever required. The 'Q' and RMR values do help in recommending temporary support requirement but one cannot be fully dependent upon these values as a guide to temporary support requirement during tunneling. The temporary support requirement is more of an orientation of joints specific rather than simply 'Q' and RMR values. A poor rockmass as per Q and RMR value devoid of vulnerable joints may not require any temporary support, whereas a good rockmass with vulnerable joints may require extensive temporary support which may otherwise fail due to wedge failure. The RMR classification does take care of orientation of joints and are more helpful in suggesting temporary support.

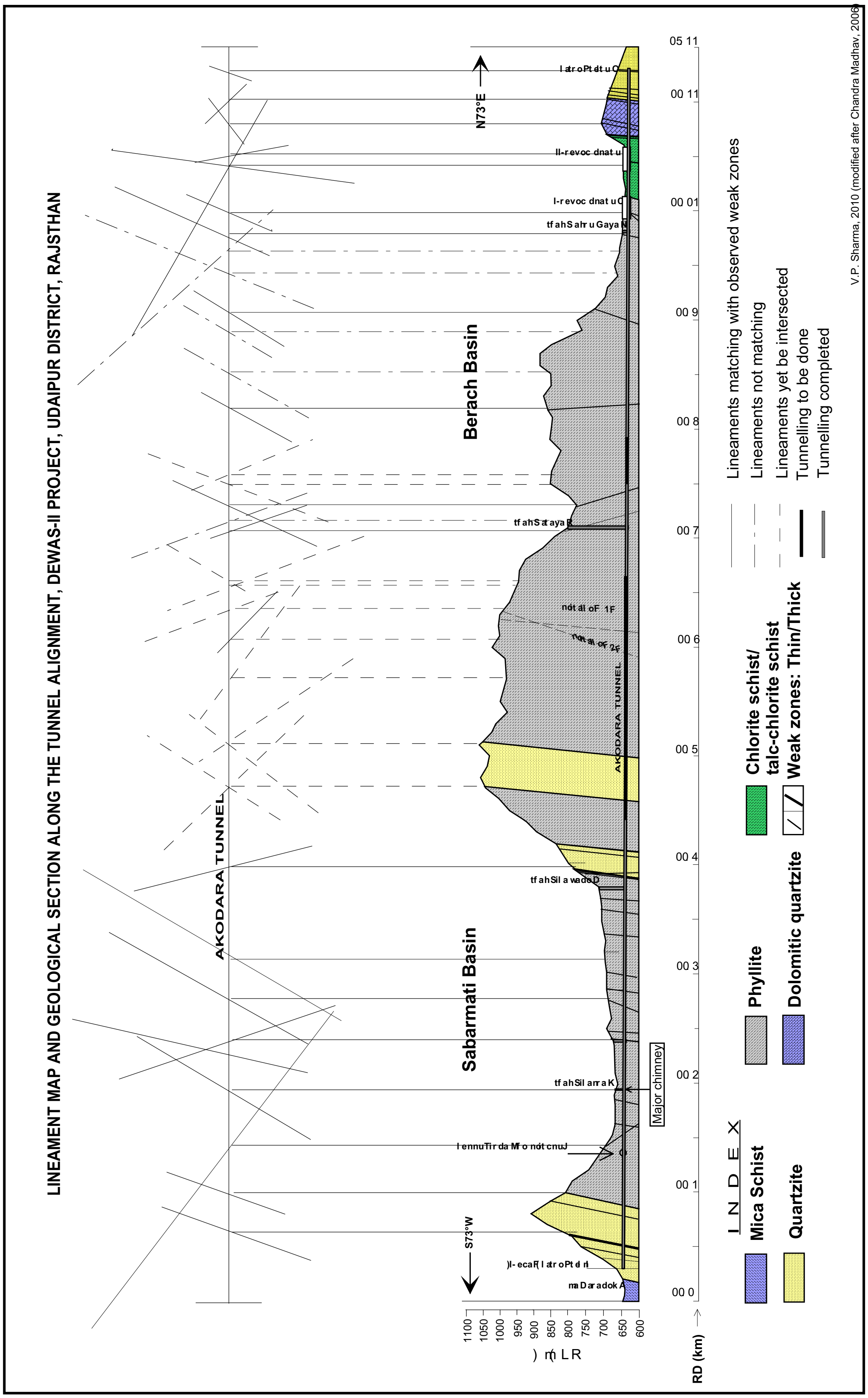

FIGURE-1- MATCHING OF LIEAMNENTS WITH WEAK ZONES OBSERVED DURING 3D LOGGING

8. Conclusions:

The strength of the phyllitic rock is governed by degree of weathering, the rock has under gone, which in turn depends upon the exposure of the rock to prevailing weathering agents. In the valley portions the weathering is more pronounced and hence tunneling in such areas at shallow depth should be done very carefully. Photogeological study is quite useful in identifying weak features, which can be projected at tunnel level just to know the likely position of the sensitive zones. The stages of weathering (W_0, W_1, and W_2) may also be taken into account in evaluating the RQD of the anisotropic rockmass while calculating 'Q' and RMR values by replacing RQD by RQD_{ca}. The RQD measured by drilling logs may be deceptive for anisotropic rocks and should always be measured by joint count method.

The 'Q' and RMR values are very good tools for planning purposes for estimating the general support requirement. The temporary support as well as additional permanent support requirement is site specific rather than simply 'Q' and RMR values. The RMR classification is more helpful in suggesting temporary support as this system takes into account the orientation of the weak features.

Acknowledgment

The author is grateful the Director General, GSI, for giving permission to present the paper in the congress. The author also expresses his gratitude to Mr. Khirod Parida, Director, EG division and colleagues of the division for fruitful discussions on the subject matter of the paper.

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