

Criticalities encountered in Construction of (HRT) Head Race Tunnel of Kameng Hydro Electric Project (600MW), In Arunachal Pradesh, India.

Md.Hadi Alam, Mr. M.K.Sharma

Abstract: This paper presents some of the criticalities encountered in construction of (14.45 Km) head race Tunnel of (600MW) Kameng hydroelectric project in Arunachal Pradesh, India located in North East Himalayan Region. As the region has complex geological formations and is considered to be the best field laboratory to learn Rock Mechanics and Tunneling Technology for weak rockmass. The ongoing excavation of HRT in Kameng H.E Project encountered various criticalities and geological problems thereby hindering the overall progress. The rock class encountered during the excavation of HRT varied from good to fair quality (class II to VIA & VIB as per Q system of rock classification). The HRT support system was designed for different classes of rock mass and the same was implemented with some modification as per the site conditions and some additional strengthening work also taken up in the crown portion of erected steel rib (ISMB200X100) in the squeezing zones. The Rock mass encountered along the excavated tunnel profile, mainly comprised of faults, folds, fracture and shear zones of poor geology. Thus the experiences of tunneling in this tectonically disturbed, young and fragile Himalaya are precious for the tunnel engineers all over the world. The experience of tunneling in 600MW Kameng H.E project for last Ten years is a boon for all of us. It was a challenging job to construct a long Tunnel with high overburden and in this young and tectonically disturbed Himalayan terrain. Geological surprises (faults/shear zones) causing cavity, loosefall, blowout, high ingress of water and squeezing ground conditions were common phenomenon that encountered along tunnel profile. As such deep and long tunnel should be carefully planned to avoid too high overburden causing squeezing ground condition or rock bursts; it was experienced and realized that under different overburden conditions of tunnel excavation the same rock strata which is safe at lower overburden may pose severe tunneling problem at a very high overburden due to increase of in-situ stresses. In this paper some of the criticalities encountered during the excavation of HRT and the cause of the criticalities as well as rehabilitation measures adopted are analysed and discussed.

Index Terms— (HRT) Head Race Tunnel construction of power project, Construction Criticalities, Geological problems, Rock load, Overburden, Insitu-stress, Cavity formation, Seepage in HRT, tunnel Squeezing problems and Blowout of tunnel excavation.

1. INTRODUCTION

The Project is situated in Arunachal Pradesh, India and about 350 km away from Guwahati, the gateway of North East India. The water conductor system from Bichom and Tenga River (both tributaries of river Kameng) over a gross head of 536m to Kimi Power House is comprise of 14.45 km long tunnel, 6.7m diameter of Modified Horse Shoe shaped concrete lined Head Race Tunnel. The project will generate 161 MW of firm power with an annual energy generation benefit of 3592 MU in a 90% dependable year.

good rock class (Class- I, II & III) and 11% in poor rock class (Class- IVA & IVB) having shear zones and comprising of crushed and fractured rock material along with clay gauge with low compressive strength.

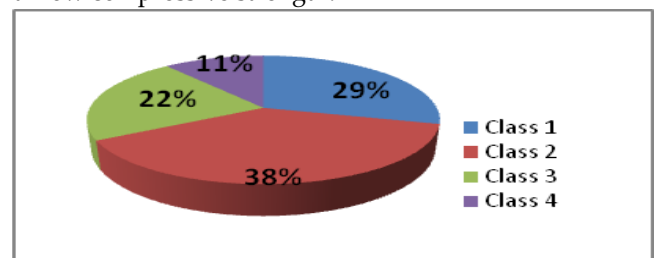


Fig. 1: Original % of Rock Classification of HRT Face-III

2. GEOLOGY ALONG THE TUNNELPROFILE.

The Project is located in a geologically complex and tectonically disturbed area, dissected by a number of major thrusts, faults, shears zones. As per initial investigations carried out the HRT was expected to encounter on upper Bichom side Precambrian rockmass, mainly of schist chlorite, sericite Phyllite and blastic Granite Gneiss. In the middle and lower reach Gondwana rocks expected of quartzite, slate and dolomite, sandstone and carbonaceous phyllites and slates. In HRT Face-III about 89% rock mass was expected to fall in the

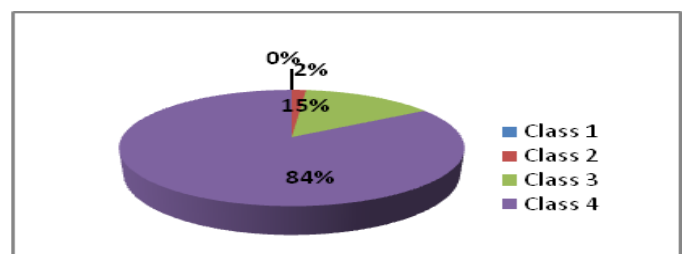


Fig. 2: Actual % of Rock Classification of HRT Face-III

The ongoing construction of 4.80Km HRT Face-III of Kameng H.E project it was found so far that 84% rockmass encountered come under poor rock category and supported by Class-IV support system. The HRT passes through Gondwana formation comprising weak sedimentary rocks mainly consist of thinly laminated carbonaceous shale/siltstone with graphitic coaly bands, Quartzitic sandstone and Precambrian metasedimentary rocks such as Quartzite, phyllite, Quartzitic phyllite, Gneiss, Carbonaceous schist/slate. These metamorphic rocks are highly folded, sheared, crushed, pulverized, jointed, foliated, banded and fractured in nature. In some stretches clay gouge has also been observed along with shear zones. Which have resulted in slow progress and difficult tunneling media, the rockmass such as phyllites and quartzites are found to be closely jointed, folded and fractured having several shear zones, rendering the rock mass into poor category. In some critical stretches where the carbonaceous rockmass encountered, major cavities have been occurred along with water ingress.

The major challenges of HRT have been successfully handled by various methods such as roof Forepoling, chemical grouting, cement grouting, umbrella/ channel forepoling and multiple drifting, the conventional tunneling method is preferred due to lesser construction risk, higher flexibility in dealing with such ground problems. With limited geological and geo-technical data for extremely poor and unforeseen ground condition available for this deep seated tunnel, flexibility in changing the support system and different excavation methodology is practiced, mainly full face, heading and benching, multiple drifting method based on rock conditions.

The HRT Face- VI, between Tenga dam and Surge Shaft and Power House are located in rocks of Gondwana Super-group has undergone intense tectonic activity in the form of folding, faulting and thrusting, represented by alternate bands of medium to coarse grained grey sand stone, carbonaceous shale/ carbonaceous siltstone with occasional presence of minor coal partings and quartz vein. Because of closeness to the Main Boundary Fault, the area has suffered poly phase deformation and due to which the rocks are highly fragmented. The highly folded rock strata encourage water entrapment and subsequent release on boring. Total length of HRT face-VI is 1684.00m, where heavy seepage of underground water was encountered along with very poor geology, the rate of seepage recorded in this stretches of tunnel varied from 6500 to 9000 liters/min.

The rock formations along the tunnel alignment have been classified based on the rock mass behavior and the rock strata encountered in the HRT have been divided into five classes i.e. class-I, class-II, class-III, class-IVA and class-IVB. The Rock Mass Rating (RMR) system proposed by Bieniawski (1976) and Q system proposed by Barton et al. (1974) and updated in Barton (1996) have been used to define generic clas-

ses of rockmass. The RMR value is assessed by six parameters of the face rock conditions and is given below:

- Uniaxial compressive strength of rock.
- RQD.
- Spacing of discontinuities.
- Condition of discontinuities.
- Groundwater condition.
- Orientation of discontinuities.

In the entire length of tunnel excavation of Kameng H.E Project, above 80% rockmass encountered falls under Class – IV (poor) rock category, some of the most common rock formations contain layers of carbonaceous shale and Phyllite and often compacted carbonaceous clay sediments which have not yet acquired the properties of rock and having very low compressive strength.

The carbonaceous shales encountered in HRT Face-III behave like squeezing or even swelling media. When the rock formation encountered consists of sequence of horizontal layers of immature fractured carbonaceous shale, the excavation of the tunnel is commonly associated with a gradual compression of the rockmass, involving a downward movement of the crown along with loosefall and cavity formation. This is due to relatively low resistance against slippage at the boundaries between the fractured carbonaceous shale, which reduces considerably the strength of rockmass located above the crown of tunnel face. The failure and the extent to the failures propagate depends upon the characteristics of the rock mass, the magnitude and directions of the in-situ stresses, the shape and size of the cavern and the intensity and orientation of the discontinuities.

3. ROCK TYPE AND ITS CHARACTERISTICS IN KAMENG H.E PROJECT.

The rock mass is characterized by its strength, deformability and in-situ stresses, and is considered to be heterogeneous, anisotropic and discontinuous medium. The in-situ deformation modulus of a rock mass is an essential parameter considered for design, analysis and interpretation of monitored data. The Modulus and strength of intact rock reduces on account of presence of discontinuities such as joints, bedding, and foliation planes within the rock mass. In thinly laminated rocks such as schist and phyllite, the modulus of elasticity parallel to the foliation is likely to be much higher than that in the perpendicular direction.

As the excavation is made in intact rockmass there is an adjustment (or redistribution) of stresses and strains around

that excavation profile. The Rock load around the tunnel structures seems to transmit both normal and shear stresses through the discontinuities in rock mass and folding of rock causes shear failure along the bedding planes. The shear strength of discontinuities depends upon the alteration of joints or the discontinuities, the roughness, the thickness of infillings or the gouge material and the moisture content. It was observed that the cavity formations in HRT Face-III were initiated by sliding of rockmass along the joint plane due to its in-situ stresses and rock load. The stresses in the rock are generally low for shallow depth and high for at greater depth.

In HRT Face-III the main rockmass comprise of Sand stone, Quartzite, Phyllite sequence, Carbonaceous shale, Schist and Slate (Table: 1). It was found that the Phyllite, Carbonaceous Shale, Schist and Slate were thinly laminated and fractured in nature which comes under poor rock category, where the RMR value range from 14 to 30. The quartzite rich zones which were mainly subordinate bands in Face-III fall under fair category and the RMR value range from 41 to 60. The rockmass consist of Schists in HRT commonly contains clay minerals such as biotite, mica and chlorite, all of these are platy minerals and are found aligned with the foliation. When present as continuous layers, they form the planes of weakness, in HRT Face-III the thinly laminated Schist and phyllite created most of the stability problems along with carbonaceous rockmass. The rock masses surrounding the tunnel perform much better, except near shear zones, faults, thrusts and intra-thrust zones and the water-charged rock masses.

The following table shows the range of RMR value of different rockmass at different chainages in HRT Face-III.

Table1: Rock type encountered in Face-III of KaHEP with respect to chainage from 0.00m to 4115.00m.

Sl no.	Chainage (m)	RMR value of Face	Class of Rock	Rock Type encountered in HRT Face-III
1	0.00m to 240.00 m	17- 34	Class – IV	Sand stone, Carbonaceous silt stone and shale.
2	241.00m to 334.12m	28-36	Class – IV	Fractured and sheared Phyllite / Quartzites.
3	334.12m to 336.02m	36-44	Class-III	Quartzite.
4	336.02m to 570.89m	21-36	Class – IV	Jointed Quartzite/ Quartz chlorite schist and phyllite.
5	570.89m to 760.37m	32-58	Class-III	Sequence of quartzite/ Phyllite/Chlorite Schist.
6	760.37m to 848.40m	62-74	Class-II	Quartzite, phyllite and Chlorite Schist.
7	848.40m to	44-58	Class-III	Quartzite, phyllite and

	888.20m			Chlorite Schist.
8	888.20m to 1280.00m	22- 36	Class – IV	Heavily fractured Quartz chlorite Talc Schist.
9	1280.00m to 1338.18m	32-44	Class-III	Quartz -chlorite Schist.
10	1338.18m to 1465.70m	18-35	Class – IV	Heavily fractured low grade Quartz chlorite Schist.
11	1465.70m to 1493.80m	35-41	Class-III	Fractured Quartz - chlorite Schist.
12	1493.80m to 1640.60m	27-35	Class – IV	Heavily fractured Quartz chlorite Schist.
13	1640.60m to 1679.60m	32-44	Class-III	Quartzite.
14	1679.60m to 1840.70m	14-32	Class – IV	Heavily fractured low grade Quartz chlorite Schist.
15	1841.70m to 2158.60m	35-58	Class-III	Quartzite, phyllitic quartzite and carbonaceous slate.
16	2158.60m to 2194.66m	25-40	Class – IV	Fractured Quartz - chlorite Schist.
17	2194.66m to 2221.40m	38-40	Class-III	Schist and Quartz - chlorite.
18	2221.40m to 2418.64m	20-38	Class – IV	Sand stone, Slate and Thinly bedded Carbonaceous materials.
19	2418.64m to 2434.50m	32-44	Class-III	Sand stone.
20	2434.50m to 3530.00m	17-33	Class – IV	Weak Carbonaceous shale, Schist, Coaly layers, fractured quartzite, thinly bedded phyllite, Siltstone, graphitic coaly layers and Sand stone.
21	3530.00m to 3700.65m	30- 35	Class – IV	Phyllitic Quartzite, Talc chlorite - Quartzitic phyllite.
22	3700.65m to 3760.00m	29- 35	Class – IV	Quartzitic phyllite.
23	3760.00m to 4115.00m	27-37	Class – IV	Quartzitic phyllite & phyllite.

It was observed that as the tunnel is excavated, stress relief allows elastic rebound of ground previously in compression to relieve stress this also confirms by some booming sound around the rib support in squeezing area of Face-III. This stress relief occurs beyond the working face as well as around the tunnel excavation.

As the depth of a tunnel becomes greater the ground condition becomes less favorable, it was observed that the stress within the surrounding rock mass increases and failure occurred when the stress exceeds the strength of the rock mass. The in-situ stresses develop in rockmass is the result of weight of overlying rockmass, the locked in tectonic stresses and strain energy. Thus the failure mechanism in the tunnel depends on the in-situ stress level and characteristics of the excavated rock mass mainly its compressive strength. As the depth below the ground surface increases, the rock stress increases and may reach a level at which the failure of the rock mass is induced. During course of excavation in rockmass these stresses get redistributed around the openings. The distribution depends largely on rockmass structures such as discontinuities, heterogeneities, folds, faults, dike, fabric etc. It is a fact that the weight of the overlying rock mass is the primary source of stress around the tunnel excavation. It was found that the stretches where cavities occurred in HRT Face-III the in-situ stresses in most of the case higher than the compressive strength of the rockmass. In HRT Face-III the average in-situ stress in distress zones is found to be in the range of **10.31MPa to 20.56MPa (N/mm²)** and when the compressive strength of rockmass falls below in-situ stress limit, various problems in excavation of Tunnel occurred such as loosefall, cavity formation and squeezing ground condition. It was estimated that par 100m rise of overburden increase of in-situ stress is around 2.5 MPa.

In order to determine the behaviour of tunnel face, the compressive strength test of rockmass at different chainages of HRT on various rock samples were determined at site laboratory with little modifications and deviations in the testing procedures by using point load test and UCS (Unconfined compressive strength) test of rock (UCS is considered about 23.4 times of point load test) (Singh and Singh, 1993). The range of value of strength of rock samples were found to be in between 1.50MPa to 23.43MPa and RMR value in the range of 11-74.

In order to perform the laboratory test, lump samples with L/W or L/D ratio 2 to 2.5 is maintained. The rock samples were collected randomly after the face blast and extensive compressive strength testing were done in site laboratory to determine its strength. The maximum and minimum rock strengths were observed when the failure is initiated normal to and respectively parallel to the weakness planes of the rock, i.e., bedding, foliation, cleavage, etc. The (UCS) test on lump of blasted rock was preferred due to extensive testing and constrain in rock sample preparation in every alternative face blast. Table: 2 below show some of the findings in HRT Face-III of KaHEP.

TABLE: 2: Laboratory testing of some of Rock samples w.r.t Chainage and some of the criticalities encountered and advantages

of excavations of HRT Face-III:

Sl No.	Critical Chainage (m) in Face-III	Compressive Strength (MPa)	Rock Type /Remarks
1	1390.00m	1.85MPa	Heavy loosefall occurred from crown, MPBX deflection at Ch: 1380.00m is observed in Lower Left.
2	1680.00m to 1776.00m	2.20MPa	Truss and struts are fixed in the crown portion of erected rib to counteract the squeezing effect. MPBX deflection at Ch: 1736.00m, Ch: 1753.00m & Ch: 1780.00m at Lower Right, Lower Left, Upper Left, and Upper Right & Crown is observed. It took average 7 months to stabilize.
3	2435.00m to 2575.00m	1.85MPa	Heavy loosefall and cavity formation have been occurred on several occasions leading to slow progress of tunnel boring. The steel (ISMB200) Truss and struts are fixed in the crown portion.
4	2652.00m	3.00MPa	Deformation of steel ribs (ISMB200) observed in the LHS SPL.
5	2683.00m	1.65MPa	Major cavity formation occurred.
6	2717.00m	2.25MPa	Major cavity formation occurred. The MPBX deflection at Ch: 2726.00m at Lower Right, Lower Left, Upper Left and crown is observed.
7	2770.00m	1.75MPa	Major cavity formation occurred.
8	2870.00m	3.50MPa	Deformation observed in the Steel ribs.
9	2895.10m	1.75MPa	Major cavity formation occurred.
10	2914.10m	1.80MPa	Major cavity formation occurred; the face excavation is done by scooping and chipping, without blasting.
11	2938.60m	1.72MPa	Major cavity formation occurred and tunnel squeezing observed. The MPBX deflection at Ch: 2927.00m Lower Right, Lower Left, Upper Left, Upper Right and Crown are observed.
12	3002.00m	1.75MPa	Deformation observed in the Steel ribs (ISMB 200) & Cavity formation occurred due to loosefall.
13	3250.00m	2.65MPa	Deformation observed in the Steel ribs (ISMB200).
14	3289.40m	2.85MPa	Deformation observed in the Steel ribs (ISMB 200) & Cavity formation occurred due to loosefall.
15	3310.00m	2.00MPa	Major cavity formation occurred.
16	3118.15m	1.75MPa	The tunnel diameter is increased

			from 7.3m to 7.6m to accommodate the squeezing in the tunnel. The MPBX deflection at Ch: 3121.50m at Lower Right, Lower Left, Upper Left, Upper Right and Crown portion is observed.
17	3197.90m	9.50MPa	Full face method of tunneling is adopted instead of heading and benching due to improved rock conditions.
18	3218.00m	1.87MPa	Squeezing ground condition was observed in the tunnel.
19	3245.15m	1.80MPa	Heavy loosefall and cavity formation have been encountered and squeezing ground condition was observed in the tunnel.
20	3270.00m	1.50MPa	Heavy deformation of steel ribs (ISMB200) in the crown portion and the cavity formation occurred from Ch: 3262.00m to Ch: 3290.00m; canopy fixing is done at crown portion.
21	3311.00m	3.00MPa	Squeezing in the Crown portion, twisting of crown ribs (ISMB200) and canopy fixing is done.
22	3335.00m	2.75MPa	Deformation at crown portion is observed. The MPBX deflection at Ch: 3320.00m at Lower Right, Lower Left, Upper Left, Upper Right and crown is observed.
23	3341.65m	1.86MPa	Cavity formation occurred.
24	3345.00m	1.95MPa	Major cavity formation occurred.
25	3381.00m	2.10MPa	Major cavity formation occurred.
26	3394.00m	3.24MPa	Deformation is observed at crown level.
27	3405.65m	3.10MPa	Deformation is observed at crown level.
28	3418.15m	2.56MPa	Deformation is observed at crown level.
29	3500.00m	2.25MPa	Cavity formation occurred due to highly folded Phyllite.
30	3533.00m	7.35MPa	Pull length increased from 1.5m to 2.5m but strength of rock along the parallel of bedding is 1.80 MPa.
31	3550.00m	4.50MPa	Pull length increased from 1.5m to 2.5m.
32	3585.00m	7.25MPa	Pull length is increased to 2.5m.
33	3615.00m	19.95MPa	Pull length is increased up to 2.75m progress increased from 40m to 60m in a month and no squeezing phenomenon is observed but strength along the parallel to bedding is 2.85 MPa.
34	3620.00m	23.43MPa	Pull length up to 2.75m to 3.00m construction of Niche is done in class-III support system, but strength of rock along the parallel

			to bedding is 3.80MPa.
35	3700.00m	21.25MPa	Pull length increased up to 3.00m, but the rock strength parallel to bedding is 3.75MPa.
36	3760.00m	24.15MPa	Pull length increased up to 3.00m.
37	3888.00m -4115.0m	20.00- 23.43MPa	Pull length taken up to 3.00m.

4. STRESSES AND ROCK LOAD IN HEAD RACE TUNNEL.

As the rock mass has undergone a complicated history of loading due to which, the in-situ stresses in rockmass becomes its integral characteristics. It is found that the rock at depth is subjected to stresses resulting from the weight of the overlying strata, from locked in stresses of tectonic origin and strain energy in the form of in-situ stress. The vertical stress below the surface is given by following equation.

$$\sigma_v = \gamma z$$

$$\sigma_v = 0.027z \quad (\text{After Hoek and Brown 1978})$$

Where σ_v is the vertical stress

γ is the unit weight of the overlying rock and

z is the depth below surface in (m).

The ratio of horizontal to vertical stress for different deformation moduli of rockmass can be calculated base on curves proposed by Brown and Hoek (1978), Herget (1988). The sudden cracks in the rockmass and the rock fall during excavation from the crown portion of HRT causes due to these stress release, but it was observed that if the strength of rock mass is more than the in-situ stresses in tunnel, then tunnel and face remains stable.

In HRT Face-III after testing of numerous rock samples in different chainages and calculating in-situ stresses from different levels of strain/ closure in the SPL of HRT and correlating the values from the curve proposed by (Hoek, 2001), it was estimated the range of in-situ stress is in between **10.31MPa to 20.56MPa** (105.16Kgf/cm² to 209.71Kgf/cm²) and average in-situ stress is found to be **14.64MPa** (149.33Kgf/cm²) in distress zones of HRT (Table: 3), the value of in-situ stress is estimated corresponding to the convergence reading and amount of squeezing in the distress zone location with due consideration of rockmass strength and its (UCS) value as proposed by (Hoek, 2001).

It was found that once the rockmass strength falls below 20% of the in-situ stress level i.e, **2.93MPa** (29.89Kgf/cm²) or below the range **2.06 MPa** (21.01Kgf/cm²) to **4.11MPa (N/mm²)** (41.92Kgf/cm²) the deformations in the rockmass increases subsequently unless these deformations are controlled by forepoling or by pre-grouting, the face collapse,

cavity formation and squeezing phenomenon are experienced in HRT.

The components of in-situ stress act both horizontally and vertically, which may be estimated in HRT by observing the steel rib deformation patterns in different directions in squeezing zones, the horizontal stress is estimated to be more than its vertical stress, Hast (1958) found that horizontal stresses often exceeds by 1.3 to 1.5 times the vertical stresses. Hence, the loads on the tunnel support system are usually erratic and non uniform due to different directions of in-situ stresses. The average direction of maximum horizontal stress in this region of Himalaya is found to be in the North Eastern quadrant. These stresses in the rockmass around periphery of tunnel may not develop immediately after excavation but may take a time period after excavation to develop, due to adjustments and displacements in the rock mass. It is also interesting to find out some of the facts how a rockmass surrounding a tunnel deforms and how the support system acts to control this deformation.

TABLE: 3: Calculation of In-Situ Stress in HRT-Face-III in major distress zone with different levels of strain as per curve proposed by (Hoek and Marinos, 2000):

HRT Face-III, Chainage (m)	Rock Mass Strength (UCS) in MPa (Q _{cmass})	Tunnel Deformation at SPL of HRT Face-III in (m) (U _a)	Radius of Tunnel in (m) (R)	% Strain ε = (U _a /R) x 100	(Value w.r.t % Strain from the curve)	In-Situ stress = Q _{cmass} / (Q _{cmass} /P ₀) in MPa (P ₀)
2472.47	2.15	0.165	3.65	4.52	0.16	13.44
2475.72	2.45	0.150	3.65	4.11	0.17	14.41
2486.22	1.85	0.480	3.65	13.15	0.09	20.56
2490.22	1.85	0.283	3.65	7.75	0.12	15.42
2500.97	1.25	0.450	3.65	12.33	0.08	15.63
2555.47	1.75	0.280	3.65	7.67	0.13	13.46
2560.47	1.95	0.230	3.65	6.30	0.14	13.93
2565.47	1.75	0.250	3.65	6.85	0.12	14.58
2569.72	1.65	0.150	3.65	4.11	0.16	10.31

From the above (Table-3) the maximum in-situ stress in HRT is estimated with different levels of strain as proposed

by (Hoek and Marinos, 2000) (Fig: 3) is found to be **20.56MPa** (209.71Kgf/cm²) (Approx. 210.00 Kilogram per square centimeter) in HRT Face-III with about 800m-1000m overburden.

It is a known fact that the state of rock in tunnel before the excavation remains in equilibrium in a gravity field. The process of tunneling evokes new equilibrium conditions which will change during the various stages of tunneling and construction of supports until a final equilibrium is reached.

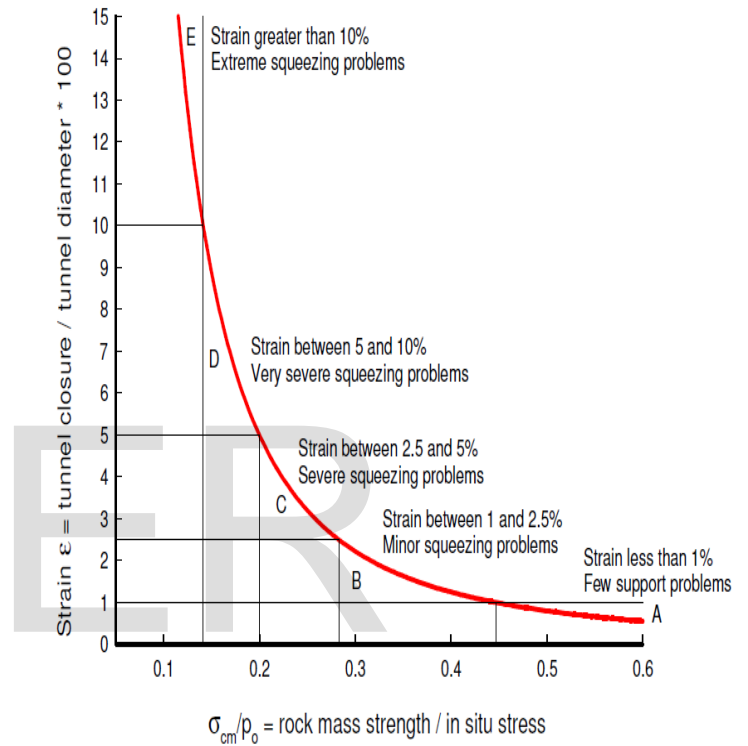


Fig: 3 Relationship between tunnel strain and ratio of rock mass strength to in situ stress (Proposed by Dr. Evert Hoek and Marinos, 2000)

TABLE: 4: Some of the findings of Horizontal and vertical in-situ stress in various projects in Himalaya (India) as reported by Central Soil and Material Research Station (CSMRS), New Delhi are given below:

Sl. No.	Name of Project/ Location	Max. Horizontal stress	Max. Vertical Stress	Remarks
1	Tehri Dam project (Uttaranchal, India)	2.38MPa		Average in-situ stress at Dam site
2	Tala H.E Project (Bhutan)	16.22-10.01 MPa	10.66MPa	Orientation N62°E at an elevation 770m.

3	Nathpa Jhakri H.E project (H.P, India)	13.15MPa	4.36MPa	At power house location N30° E.
4	Pancheswar Multi-Purpose Project (Nepal/India)	2.5-15.80MPa	4.1-5.5MPa	At depth 223m
5	Punatsangchu -II H.E Project II, Bhutan	6.52MPa	4.64MPa	At desilting chambers.
6	Tawang H.E Project Stage-I, Arunachal Pradesh	11.62-7.42MPa	4.92MPa	Orientation N63°E

The 600 MW Tawang H.E project Stage-I on river Tawang Chu having HRT length of 14.00 Km in east Kameng district of Arunachal Pradesh, where (CSMRS) recently conducted number of tests to find out the in-situ stress by hydraulic fracturing tests and the horizontal stress is found to be 7.42MPa to 11.62MPa and Vertical Stress is around 4.92MPa, the direction of horizontal stress is N63°E.

The strain in the rockmass is calculated by MPBX (Multi-Point Borehole Extensometers) and its reading clearly shows the rock movements in HRT Face-III (Fig:4) and same is recorded by 0.5m, 1.50m and 3.00m length anchors in the rockmass. The convergence readings are also taken by plastic survey reflectors of the Geodata Bireflex type. It was found that the average time taken to stabilize the rockmass is around 6 to 7 months after the excavation for HRT Face-III & VI. In rocks where the loads and deformations do not attain stable values, it is recommended that the lining work should not be taken up for safety measures. If the final lining is installed after the tunnel has been stabilized by initial support, the final lining will undergo very little additional loadings.

Thus the element of time plays a very important part in determining the final deformation and the loads coming on to the support systems. The more the time elapses, before the supports are installed, the more the deformation and lesser the stresses in the support system but no direct correlation between the reduction in stresses in the support system and the deformation of the tunnel is worked out. It was understood that the rock mass squeezes in the tunnel and the phenomena is time dependent and depend on the degree of stresses and rock strength.

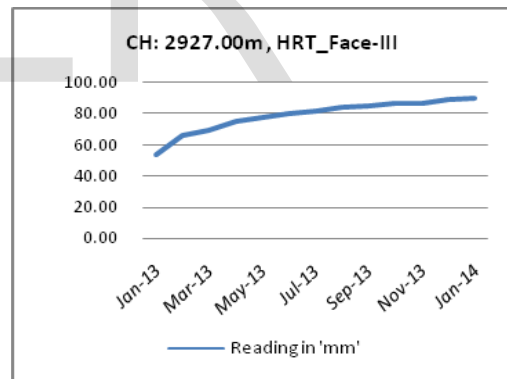
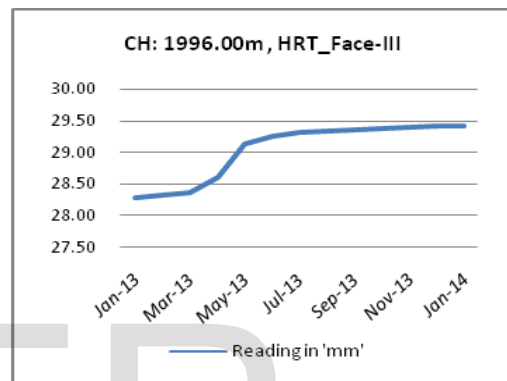
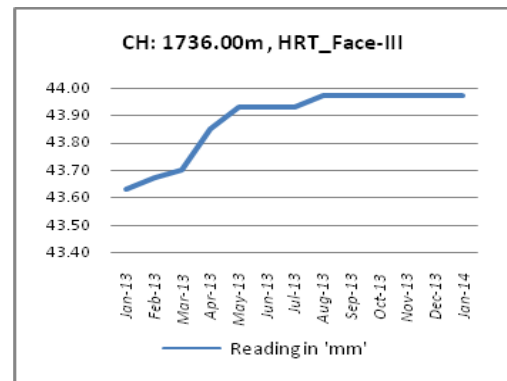


Fig:4: HRT Clouser of Face-III at various changes 1736.00m, 1996.00m and 2927.00mm (MPBX reading)

The photographs (1 & 2) shown below is the face rock condition of HRT face-III where the deformation in the steel ribs occurred after fixing the support system due to very high active stresses in rockmass and the compressive strength of rockmass was found to be below 7.00MPa (71.40Kgf/cm²). The rockmass shows very complicated joint patterns and bedding plane.



Photo: 1 Anticline in HRT ch: 2922.85m Face-III

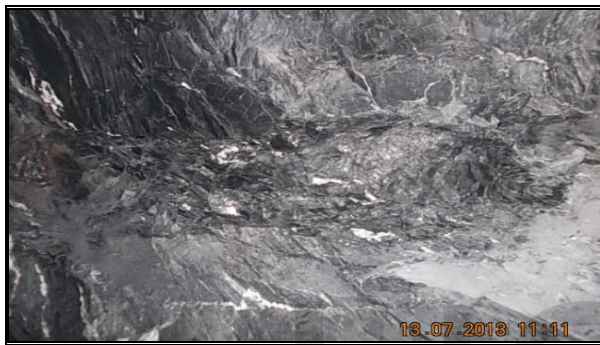


Photo: 2 Shear zone in HRT ch: 3275.15m Face-III

The rock classes generally encountered in HRT Face-III from Ch: 2200.00m to Ch: 3550.00m is of squeezing in nature due to highly presence of carbonaceous rockmass and coaly layers with highly fractured and jointed rockmass. At some places the crown as well as the side walls were converged by mechanisms of stress induced. It was observed in HRT Face-III and VI that the stability of underground excavations, cavity formation and squeezing ground conditions depends on following factors.

- When the in-situ stress in rockmass is more than the compressive strength of the rock. (Table: 2)
- The unfavorable condition of the rock mass of weak and sheared with low uniaxial compressive strength and when foliations were parallel with the tunnel excavation.
- When the carbonaceous rockmass and clay rich formation and rockmass containing clay gouge.
- When the entire rock load fell on crown portion due to its orientation of joint patterns, leading to buckling of steel ribs sometime collapse of erected ribs (Photo: 3).
- Due to high difference in pre and post excavation stresses.
- Size and shape of the openings and the methodology of excavation such as full-face, heading and benching and multiple drifting methods.

- Nature and type of rocks and their engineering properties.
- Groutability of rock.
- Depth of overburden and rock covers over the opening, as it increases approx. @ 2.5MPa, (25.5Kgf/cm²) per 100m rise.
- Extent of discontinuities like faults, number of joint sets, fracture planes etc.
- Frequency of joints, their orientations, joint filling materials, and permeability characteristics.
- Presence of shear zone or over stressed zone.
- Slickensided zone, thick fault gouge and weak clay and shales.
- The permeability of the rockmass around the tunnel opening.

Most failures in tunnel face are initiated at the crown portion, when the highly fractured rockmass that wants to loosen and come out, which do not possess the inherent resistive strength. This can continue until the formation of cavity in tunnel or completely collapses or until the geometry and stress conditions come to equilibrium. It was observed that the tunnel excavation causes a disturbance of the initial state of stress in the ground and creates a three-dimensional stress regime in the form of a bulb around the advancing tunnel face. The failure mechanism depends on the in-situ stress level and characteristics of the given rock mass. Such a stress regime is indicatively displayed in Figure (5).



Photo: 3 Cavity formations in HRT Face-VI

As the (Photo-3) above is of HRT Face-VI, where the rock was not sufficiently strong enough, the strength of rock (UCS) found to be below 7.00MPa, the induced stress due to rock load was more than the rock strength resulting rock failure thereby forming the cavity in the crown portion. It was observed that when tunnels is excavated in jointed rock mass at relatively shallow depth with lower overburden, the most common types of failure are those involving wedges falling from the roof or sliding out of the sidewalls of the openings. These wedges are formed by intersecting structural features, such as bedding planes and joints, which separate the rock

mass into discrete but interlocked pieces. When a free face is created by the excavation of the opening, the restraint from the surrounding rock is removed.

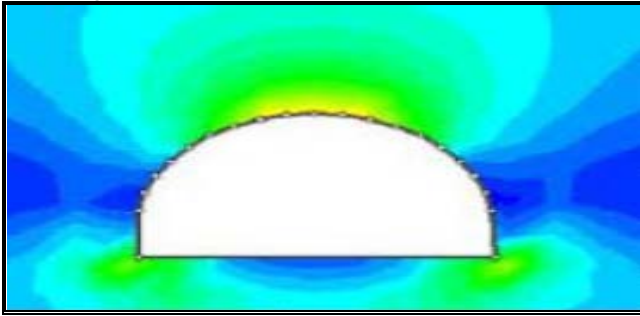


Fig: 5: Predicted bulb of Rock load on the steel support.

Unless steps are taken to support these loose wedges, the stability of the tunnel opening deteriorates rapidly. This failure process will continue until natural arching in the rock mass prevents further unraveling or until the opening is full of fallen material. The cavity formation in some location in HRT was due to encounter of local thick shear zones dipping towards the tunnel face. The soil/gouge materials not having any inherent strength fall down rapidly unless it is supported carefully and immediately.

Thus a higher cavity/ chimney formation along the thick shear zone are common phenomenon in HRT Face-III, unless the face is protected at crown prior to blasting. It was also observed in HRT Face-VI where the chimney formation occurred in water-charge rockmass, where seepage out of excavated rock susceptible to erosion and dissolution. The rock load acting on the tunnel varies depending upon the type and mechanical characteristics of rock mass pre-existing stresses in the rock mass, the nature and strength of rock. Thus rock at depth is subjected to stresses resulting from the weight of the overlying strata and the excavation in the rockmass causes the stress field to disrupt and a new set of stresses are induced in the rockmass surrounding the opening. Failure occurs if the strength of the rock is less than the induced stress (Photo-4).



Photos: 4: Shear failure of steel rib due to developed stresses in cut and cover portion above the flash portal & cavity formation.

The rock load is also affected by ground water conditions, which lubricate the joints in rock and cause greater load than when the material is dry and causes the cavity formation. During the excavation of tunnel, whenever it was felt that the rock loads at crown of excavated face is likely to exceeds the strength of rockmass, to prevent the further rock fall, immediately the steel canopy in the form of umbrella in the crown portion is provided. In several locations in HRT Face-III the canopy with (ISMB 200X100) is provided as preventive measures.

5. METHODOLOGY ADOPTED FOR EXCAVATION OF HRT.

In KaHEP Package -II, the total length of HRT is 06.90 km, comprise of (Face-III & IV 4.95km and Face-V & VI 1.95Km). The HRT is horseshoe shaped concrete lined has been designed to convey 140 cum design discharge. The length of total HRT from intake to surge shaft is 14.45 km with 4(four) Adits of different lengths have been constructed to facilitate and accelerate the construction activity. The tunnel excavation is carried out by drilling and blasting through full-face and heading and benching method, normally 1.5m to 2.00m meters pull is achieved in each round. The rock mass encountered during HRT face advancement mainly consists of Phyllite, Quartzite and carbonaceous materials and maximum rock is classified and comes under Class IVA /IVB category. The tunnel advancement in heading and benching method is adopted to avoid formation of cavity as well as to tackle the cavity easily if any. The heading is generally carried out for a stretch of 8.00m followed by benching of 8.00m. The heading is kept at least 4.00m in advance than benching. The steel rib of ISMB 200X100 at a spacing of 0.5m, 0.75m and 1.00m c/c is adopted for tunnel support considering the rock condition. The average progress is found to be around 40-45 m in a month. The average requirement of explosive is 0.61 Kg/m³. The construction niche is constructed at a spacing of 300 m along the left side of the tunnel and drainage system is maintained on the right side throughout the length of the tunnel. The ventilation system of 1.90m diameter is provided at SPL on right side.

The face maps are prepared after every blast and RMR value is calculated. Excavation methodology and support system are reviewed progressively during the construction. As the construction method has to deal with varying ground conditions and large deformations in the form of crown settlements, longitudinal as well as horizontal deformations. The New Austrian Tunneling Method (NATM) was also adopted in HRT Face-III about 15% of total length of tunnel was excavated by NATM, when the RMR value range from 36 to 63 with high compressive strength of rock and when the rock encountered mainly comprised of good quality Quartzite,

Schist and Phyllite with lesser joints.

6. CRITICALITIES AND REMEDIAL MEASURES ADOPTED DURING CONSTRUCTION OF HRT:

6.1 Cavity formation in HRT Face-II

Progress of HRT Face-II boring was severely affected due to poor geology, on 12.01.07 a cavity of 4.0m X 5.0m X 08.0m was formed on right hand side of the crown at Ch. 422.60 m of HRT Face-II. Cavity was formed in rockmass with shear zone encountered within banded Quartzite. A small opening was formed at the extreme right side through which mud/slush with high pressure flowed into the tunnel. A mixture of soil and water flows into the tunnel like a viscous fluid. The semi-liquid slush flowed from the cavity was estimated to be 200cum to 400cum each time. Seepage water flow to the tune of 160Litre/Minute to 400Litre/Minute was also observed. The total length of the critical zone consisting of loose muck was assessed to be 7.20m distance from Ch 422.60 m. The height of loose zone was found to be extended up to 24.00m. Basic rock type encountered was mica Schist; talc Schist and Quartzite with abundance of Kaolin of Pre-Cambrian group.

The loose muck of cavity comprised of angular to medium and coarse quartz and kaolin clay of equal proportion. In order to arrest the loosefall face plugging is done by muck and steel canopy was provided at the crown and the face is restored by providing of 83nos. of 100mm dia., forepole tubes, injecting 9.5mt PU (polyurethane) chemicals, erecting box girder, carrying out 9954bags of Micro fine cement grout. Total 7 month was required to tackle the problem of HRT Face-II. On 29.12.07 blowout at Ch. 616.68m occurred with coming down of huge water and rock fragments/debris and filled up to a HRT length of 220m.

Seepage of around 6500Liters/Minute was observed in HRT. The rockmass were comprised of complexly folded in the form of major anticlines and synclines with major fold, vertically faulted and sheared (GSI Report, 2008). Blow out occurred due to heavy accumulation of ground water in the syncline fold closures. The built up pressure and in-situ stress due to gradual rise of tunnel cover triggered the blow out and the debris flow out comprised of Quartzitic materials with infilling clay. The remedial measures encompass use of 80mm to 100mm dia. M.S perforated pipe forepoles grouted with Micro fine cement, Ordinary Portland Cement (OPC) and PU (Polyurethane) grout, the entire restoration work was completed in 9 months. Again on 27-07-2010 another blowout occurred at Ch: 1337.00m in HRT Face-II, the basic rock types encountered were thinly bedded quartzite, Mica Schist, Talk and Graphite Schist belonging to Bomdila Group.

The rock was thinly fractured and folded with variation

of rock within a metre even. The rockmass infillings of talk/sericite/clay within the foliation joint associated with heavy seepage created most of the tunnel problems and the face was restored with similar methodology adopted for tackling the previous problems i.e., 80mm dia. and 12.0m long perforated pipe forepole grouted with OPC/ micro-fine cement and Polyurethane material. The entire restoration work was completed in 5 months.



Photo: 5: Cavity /blowout ch: 422.6m in HRT Face-II



Photo:6: Cavity /blowout ch: 616.68m in HRT Face-II

6.2 Squeezing effect and Cavity formation and in Face-III.

In some stretches the presence of very weak rockmass along with high rock cover (Approx. 800m- 1000m) led to squeezing problems. It has long been understood that the ground, if allows to deform slightly is capable of contributing to its own support but in case of unfavorable condition of the rock masses which were weak , sheared and foliated and the in-situ stress is more than the strength of rock the deformation is much severe. In HRT Face-III, frequent collapses with heavy over-breaks and chimney formations of 3.00m to 9.00m depth were experienced during tunneling due to highly jointed and sheared phyllite and carbonaceous rock conditions. Minor cavities are also frequently encountered in tunneling because of presence clay strata and water along the

shear zone details given in (Table: 2).

It was observed that the ground with low frictional strength the rate of squeeze depends on degree of overstress. At several places in HRT Face-III, the rock shows gradual squeezing tendency or deformations generally over period of about one month after excavation. This is manifested by development of cracks in the lagging and buckling of steel ribs and disjuncting of concrete slabs with steel ribs (Photo-8). The squeezing mainly occurred when the rockmass consists of shiny carbonaceous materials with very low compressive strength of below 2.00Mpa (Photo-7). The rock mass encountered in squeezing area was weak in nature and the uniaxial compressive strength was significantly low and presences of no definite joint patterns were observed. After the excavation immediately the steel support system were fixed and back-filled is done with M20 concrete but after 20 to 30 days the deformation in the rib was observed due to high stresses in the rock mass.

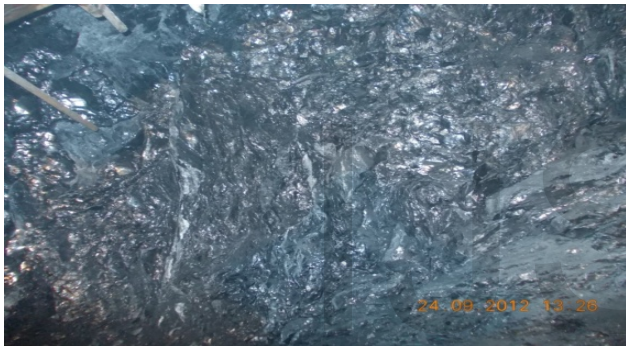


Photo: 7 Squeezing in crown portion in HRT Face-III



Photo: 8 Squeezing Rock strata in HRT Face-III

After excavation the local stress regime changes in ground condition, the rockmass around the opening loses its inherent strength under the influence of the stresses. The stress increases due overburden rock load, in most of the squeezing rock formation the compressive strength of rockmass is found to be below 7.00MPa. It was observed that the load in the rock

mass is transferred by internal shear zones to adjacent ground until an equilibrium condition is reached. If the ground is weak and the overburden load is too great, the tunnel may close by deforming the support system. Thus, if final equilibrium is reached before support system is provided in the tunnel, the support may not receive loads from the medium at all. In HRT Face-III, apart from weak ground conditions number of shear zones have been encountered and negotiated during tunneling, where also the squeezing phenomenon is predominant. Apart from in-situ stresses the squeezing may also due to result of pore pressure and swelling of clay minerals. Presence of some clay minerals such as Kaolin and Montmorillonite, which have the tendency of squeezing as well as swelling, exerts pressure on the support system; the rockmass Schist that encountered in HRT Face-III contains mainly the clay minerals. According to Mohr's theory, the squeezing conditions were also predicted under rock cover more than 300m and rock burst conditions were met where overburden is more than 1000m.

It was experienced that the in-situ stresses are influenced by the depth below the ground surface, the maximum squeezing occurred in HRT Face-III, where the range of overburden is 800m – 1000m. Therefore, it is recognized that the depth of tunnel or the overburden of carbonaceous rocks are important parameter and probable cause of squeezing phenomenon in the HRT. The effects of tunnel depth or the overburden on support pressure and closure in tunnel have been studied for HRT Face-III, which are summarized below.

- The tunnel depth has a significant effect on support pressure and tunnel closure in squeezing ground conditions of carbonaceous rock.
- The depth effect on support pressure increases with deterioration in rock mass quality probably because the confinement decreases and the degree of freedom for the movement of rock blocks increases.
- The fact would be of help to take decisions on realigning a tunnel through a better tunneling media or a lesser depth or both in order to reduce the anticipated support pressure and closure in tunnels.

The thickness of shear zones encountered in HRT Face-III, varied from few centimeters to a meter. The formation of cavity as well as squeezing effect are frequent in these shear zones. Therefore the asking rate of HRT excavation went up due to very poor ground conditions. The rock support system being provided in HRT Face-III has been reviewed due to unsatisfactory rate of progress. To tackle the problem of squeezing in HRT, modifications in the tunnel section have been incorporated by increasing tunnel diameter from 7.3m to 7.6m from Ch: 3118.15m (March'2013).

The following (Photograph-9) show the condition of rockmass before fixing of support system and the squeezing effect on support system were seen after laps of time period. The anticline and syncline in the rockmass indicates the presence of stresses and the significant signs of squeezing in the form of deformation in ribs are observed in the SPL.



Photo: 9: The rock condition of face and the squeezing takes place the rock seems to experience high stress.

The pattern of deformation indicates that the induced stresses around the tunnel acts directly on the tunnel steel support system through the encountered shear zones comprising of thick clay-filled fault gouges, weak clay shales and flowing ground conditions. Where the interlocking of blocks are missing, the joint strength, the rock strength is lost and the rockmass is unable to counteract the in-situ stress due to its orientation of joint patterns.

The encounter of such fractured shear zones during excavation results the cavity formation and subsequently squeezing phenomenon in course of time. Some of the reasons for the cavity formation in the squeezing zones of HRT Face-III are analyzed and are given below:

- The fractured rockmass filled with crushed rock and clay particles with seepage, when excavated the circular and curvilinear fracture plane in the crown portion susceptible to fall and forms cavity.
- Foliation joints dipping at an angle of 10-15 deg towards face of tunnel and forming low dip planes in the crown.
- Very low compressive strength of rockmass and in most of the cases the rock strength is found to be below 2.00 MPa.
- Smooth undulated and clay coated joint surface with low cohesion and foliation planes separating out from each other causing loosening in rock mass.
- Micaceous nature of sheared quartzite having clayey nature which swells with water and also creates pressure on the support system.

- Presence of crushed and sheared materials along the shear zones.

To tackle Cavity problems, following remedial measures were adopted in HRT:

- To allow all loose materials to fall-off completely in case of minor cavity or to plug the face with muck than concrete filling is done.
- Channelizes the excessive seepage water if any by drilling holes and inserting pipes.
- In soft rock strata to prevent loosefall umbrella roofing or canopy is provided by using channel sections or with ½ ISMB 200X100 forepoling in the crown portion.
- Concrete is filled-up in the gaps created by cavities after face plugging.
- Backfill and consolidation cement grouting in the cavity portions and area ahead of cavity to consolidate weak strata.
- Pre-support using various techniques, such as spiles and forepoling with steel bars, soil nails, soil doweling typically are installed through and ahead of the tunnel face.
- Systematic doweling in top heading considering joint spacing.



Photo:10 The face condition where the dry crushed sheared material suddenly comes out exerting high pressure in rib support. Ch: 1385.0m, HRT Face-III.

In an inhomogeneous rockmass, the process of failure is initiated by its weakest link the zone of loose soil and weak rock, crack, bedding plane, soft seam, etc. Further, the failure in an inhomogeneous rockmass is progressive, whereas in homogeneous rockmass failure is abrupt. Hence the advantage of inhomogeneous materials is that it gives an advance warning of the failure process starting slowly from the weakest zone. The thin rock layers may buckle under high in-situ stresses first and then they may rupture progressively by violent brittle failure. Experience from tunneling has

shown that cavity and squeezing ground conditions are generally encountered where the peak angle of internal friction is less than 30. Moreover rocks have pre-existing planes of weakness like joints and bedding planes etc. as such failure occurs mostly along these planes of weaknesses. The fact is clear that in squeezing ground condition the stiff support system will attract high support pressure as it will restrict the tunnel closure but if flexible support system is provided after some delay it will attract much less support pressure thereby more tunnel stability.

6.3 Ingress of water at HRT Face-VI

The rock types encountered during excavation of HRT Face-VI are mainly alternate bend of gray sandstone, laminated carbonaceous shale, carbonaceous siltstone and thin layers of coal seam of 'Gondwana Supergroup. The rocks were found to be highly folded, fractured and sheared in nature, the fractures and closely spaced joint planes are filled with crushed rocks, rock powders, clay particles and secondary carbonaceous infillings.



Photo: 11: The seepage water comes out at high pressure.

The highly folded rock strata encourage water entrapment and subsequent release on boring. Total length of HRT face-VI is 1684.00m and heavy seepage of underground water was encountered along with very poor geology from Ch. 331m to Ch. 343m, Ch. 532.00m to Ch.595.00 m, and from Ch. 750.75 to 781.15 m. The heavy ingress of water was first encountered on 17th Feb'2007 at Ch: 333.0m. The rate of seepage recorded in this stretches of tunnel varied from 6500 to 9000 liters/min. The progress during the year 2007-'08 was only 360 m due to the criticalities encountered. The tunnel alignment had also passed through three major shear zones, leading to collapse of face, formation of huge cavity and flow of loose muck/sludge accompanied by heavy seepage water from the face of HRT.

The seepage out of excavated rock susceptible to erosion and dissolution and resulted the collapse of the opening. The first major shear zone encountered at Ch 743.75 m in July'2008, second at Ch 784.00 m Aug'08 and the third at Ch

958.00 m in September'2009. Hence total time required to overcome these criticalities was about 13 months.

In order to stabilize the rockmass and to divert the excessive seepage water, extensive cement and admixtures grouting was carried out. The Colloidal silica grouting was also carried out for rock improvement and reducing seepage water. Finally Polyurethane (Sodium silicate or Polyurethane) (PU) grouting about 10mt was carried out and Polyurethane grouting was needed to divert seepage water from the HRT at two locations, one at Ch 784.00 m and another at Ch 958.00 m, for further face advancement of tunnel. Work suffered badly during the period and Tunneling was suspended for many months in these stretches. In order to drain out the seepage water from tunnel dewatering system was strengthened by installing additional pipe lines and pump sets.



Photo: 12: PU (polyurethane) Grout, HRT Face-VI.

The PU Grouting was done under high pressure in HRT Face-VI, by drilling 45 mm dia, 5.00 m long holes all along the HRT periphery and in the face keeping an overlap of 2.00 m. The grout pressure up to (60 bar) was used to inject the PU grout into the fractured rockmass, which reacts chemically with seepage water. The PU grout stabilized, strengthen and seal the rockmass by diverting the huge seepage water from the tunnel face to adjoining area. The face condition was improved by injecting PU grout and further tunneling was possible in HRT Face-VI.

7. CONCLUSIONS

In Kameng H.E Project the HRT is passing through one of the most geologically complicated and challenging terrain, where disposition of geological formations is controlled by several tectonic plans, share zones with in-situ stresses. It was found in the HRT face-II, face-III & VI of KaHEP, that when the rock strength of tunnel face goes below 20% of average in-situ stresses, which is around **2.93MPa**, (2.93N/mm²) or (29.89Kgf/cm²) the chances of loosefall and cavity formation is much more and the occurrence of squeezing phenomenon is

predominant, thereby threatening the stability of tunnel. It was also observed that if the compressive strength of rock is more than **20.56MPa**, (20.56N/mm²) or (209.71Kgf/cm²) the problem of cavity formation and loosefall rarely encountered and tunnel is found to be stable, provided restriction of free movement of rockmass in their join pattern. The pattern of joints, folding and seepage water of rockmass have great impact on stability of rockmass in tunnel face. It was also found during the compressive strength test of HRT rock samples, that when the force is applied to the rockmass parallel to its bedding plane the strength of rock found to be around 20% strength, then when the force is applied perpendicular to its bedding plane. Therefore for the same rock type the tunnel stability depends on the orientation of the bedding plane and its capacity to counteract the in-situ stresses by virtue of its strength. The fact is clear that in squeezing ground condition that the stiff support system will attract high support pressure as it will restrict the tunnel closure but if flexible support system is provided after some delay it will attract much less support pressure thereby more tunnel stability.

In Kameng H.E project the magnitude of HRT mishaps and criticalities could not be predicated during DPR stage due to limited geo-technical investigation data. The experience of Kameng H.E project covering different engineering and geological issues may be taken as an eye opener for hydro structures being planned in geo-technically sensitive lesser Himalayan region. The tough and challenging task of tunneling through poor geology by evolving strategies involving engineering skills and innovative ideas were the key to successful tunneling. The criticalities and remedial measures adopted here shall be of great use during tunneling in similar conditions in other projects in India.

Thus the tunneling through the Himalayan rock formations is a very difficult task but the unforeseen problems are unavoidable in any under-ground excavation. If not properly handled, these problems can lead to serious consequences and failure of the projects. Apart from the safety implications, substantial economic cost is also associated in the form of delay caused in the project. Proper geological investigations with vigilant monitoring, testing and proper engineering supervisions are required during tunnel excavation. Also, as far as possible timely remedial action plan should be made in advance keeping all previous shortcoming and observations in mind. This will be helpful in saving time and cost with more safety during the execution to tunneling project.

• Author name is **Md. Hadi Alam (B.E) (Civil), (M.E) (Geotechnical)** currently currently working as **Dy. Manager (Civil) /Tunnel Engineer (HRT) and Mr. M.K.Sharma E.D (Hydro) in North Eastern electric Power Corporation Ltd.India.**

E-mail: hadi.alam29@gmail.com.

Acknowledgement: Author thanks Mr. Sanjib Dhar (EIC/DGM(C), Mr. B.Deb Sr.M(C), Mr.Jayanta Hazarika Dy.M (C)/Dam, All site Engineers of NEEPCO and Mr.S.Prakash, Geologist M/S PEL /laboratory staff of M/S PEL for their kind co-operation. Authors Special thanks to Mrs.Serene Habiba Robbani, Md.Tanbir Alam, Md. Wazir Alam, Miss Nabeeha Hadi and Zidan Hadi for there help and constant support.

REFERENCES

1. Hasan Abdullah (CSMRS, India) (2013) "Laboratory Assessment of Rock".
2. Rajbal Singh (CSMRS, India) (2013) "Shear Strength and Deformability Characteristic of Rock Mass".
3. Hari Dev (CSMRS, India) (2013) "Importance of In-Situ Stresses for Design of Underground Structures in Rock Mass".
4. V.M Sharma (2013) "Modeling the Element of Time in Rock-Support Interaction Analysis".
5. R. K. Goel (CMRI, India) (2004) "Experiences of Tunneling in fragile Himalayan Rocks".
6. Arid Palmstrom, Rajbal Singh (2001) "The deformation modulus of Rock Masses".
7. E. Broch (1982), Department of Geology, The Norwegian Institute of Technology "Estimation of strength Anisotropy using the Point-Load Test".
8. Bhawani Singh and Rajnish K. Goel " Strength enhancement of Rock mass in tunnel (Page-122), Tunneling in weak Rocks"
9. Bhawani Singh and Rajnish K. Goel " Tunneling hazards" (Page-308), Tunneling in weak Rocks"
10. L.Yang and R.L Sterling "Back Analysis of rock tunnel using boundary element method".
11. Gabriel Fernandez "Behavior of pressure tunnels and guidelines for liner design".
12. T. W. Hulme, A.J. Burchell "Bored tunnel for Singapore Metro".

13. *Edwin T. Brown, John W. Bray, Branko Ladanyi & Evert Hoek "Ground response curves for rock tunnel".*
14. *Akira kitamura & Yuzo Takeuchi "Seikan Tunnel".*
15. *C.Stella, G.Ceppi & E.D Appolonia "Temporary tunnel support using Jet-Grouted cylinders".*
16. *N. Phienwej, P.K. Thakur & E.J Cording "Time- Dependent response of tunnels considering creep effect".*

IJSER