Rock Supports and Geological Appraisal for Head Race Tunnel of Tala Hydroelectric Project, Bhutan

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ABSTRACT

This paper presents geological appraisal and the rock support systems adopted during excavation of the head race tunnel of Tala hydroelectric project in Bhutan. Rock supports in terms of steel fibre reinforced shotcrete (SFRS), grouted rock bolts, self-drilling anchors and steel ribs were provided for supporting the excavation. Finally, the tunnel was supported with concrete lining. Apart from this, contact and consolidation grouting was also done for improving the properties of rock mass. The rock mass class encountered during the excavation varied from good quality to squeezing condition. The support system was designed for different classes of rock mass and the same was implemented with some modifications as per the site conditions. DRESS technique was used. The extensive quality control and instrumentation were helpful to take the corrective measures, wherever required.

Keywords: Rock support; Rock mass; SFRS; Rock bolt; Steel ribs; Geology

1. INTRODUCTION

The rock mass has undergone a complicated history of loading over many millions of years. Even the excavations in sound rocks may also not be stable because of high prevailing in-situ stresses. Seepage water may also pose enormous problems. In spite of extensive research performed in the field of rock engineering, still rock support design rely on empirical methods and observations during excavation. All the available rock mass classification systems have certain limitations and need to be applied cautiously. Monitoring of rock deformations, pore pressure and load characteristics during the excavation are extremely important to assess the behaviour of rock mass and ground support interaction. It is interesting to note that rock bolts and shotcrete represented the last major innovations in underground excavation support technologies. While rock bolts were enormously successful since their introduction in 1940's, even today, the rock bolt parameters and layout are specified primarily on the basis of empirical procedures and practical experience.

Tala hydroelectric project is a run of the river scheme in South West Bhutan in Eastern Himalayas, located on river Wangchu. The project includes a 92 m high concrete gravity dam; intake structure with three intake tunnels; three desilting chambers each of 250 m x 13.90 m x 18.5 m size for removal of suspended sediments of 0.2 mm and above size; a modified horse shoe shape head race tunnel (HRT) of 6.8 m diameter and 23 km in length to carry the water to underground powerhouse (206 m x 20 m x 44.5 m) for utilising a gross fall of 861.5 m to generate 1020 MW of hydro power (6 x 170 MW) and an underground transformer hall cavern. A simple horse shoe type tail race tunnel of 3.1 km length and 7.75 m diameter discharges the water back into the river Wangchu. Project was executed by the Tala Hydroelectric Project Authority (THPA), a joint venture of the Govt. of India and the Royal Govt. of Bhutan. The layout plan of the project along with HRT is shown in Fig. 1.

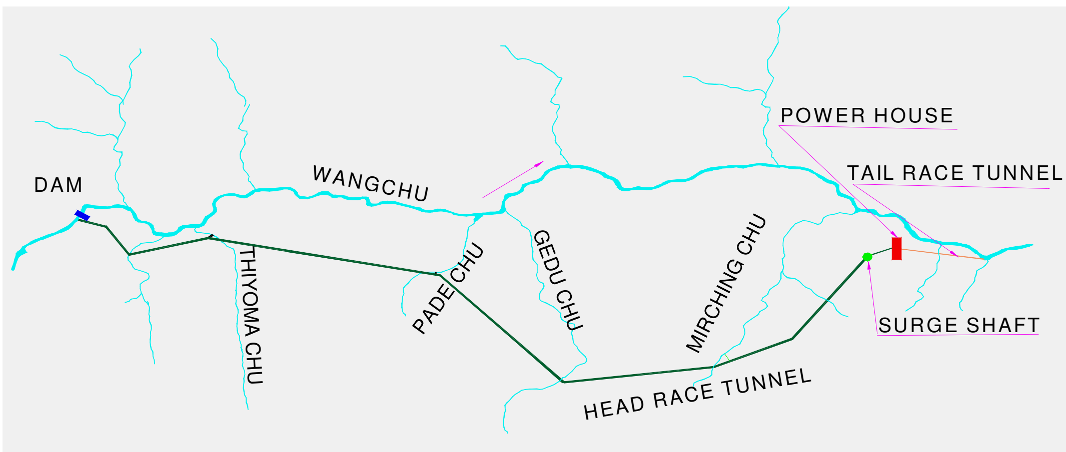

Fig. 1: Layout plan of Tala hydroelectric project, Bhutan

2. HEAD RACE TUNNEL (HRT)

A 23 km long and 6.8 m diameter HRT has been driven through 11 excavation faces in various rock mass classes classified using Q system of Barton et al. (1974). The total length of HRT was divided into four construction packages. Package C-1 comprises of 6.4 km length of HRT starting from downstream desilting chambers end and was excavated through three faces. Package C-2 consist of 5.0 km tunnel length excavated from two faces. Package C-3, comprising of 4.4 km length of HRT, was excavated through two faces whereas package C-4, excavated from four faces with a length of 7.2 km was the most critical package with regard to the difficult tunnelling conditions.

3. GEOLOGICAL APPRAISAL

The entire HRT was excavated through medium to high grade metamorphic rocks of Central Crystalline Group (designated as Thimphu Formation) of Pre-Cambrian age in Eastern Himalaya. Geologically, distinct zones of different rock mass characteristics influencing the stability in various portion of HRT were identified. A number of cross and foliation parallel shears were intercepted. Water seepage of the order of 30 to 500 litres per minute was associated with major shears.

3.1 Classification of Rock Mass

In Tala HRT, the rock mass classes were designated using Q system (Barton et al., 1974). The rock mass classes generally encountered varied from stable to flowing conditions. At some places, the crown as well as the side walls were converged by mechanisms of stress induced visco-plastic flow. Excavation face was logged with every pull by the engineering geologists. Immediate support measures were suggested and implemented. Length of various rock mass classes actually encountered in HRT is given in Table 1.

Table 1: Rock mass actually encountered in various contract packages of HRT

Contract package	Length in m as per rock classes in Q system						
	I	II	III	IV	V	VI	Beyond VI
Range of Q Value	100-40	40-10	10-4.0	4.0-1.0	1.0-0.1	0.1-0.01	
C-1	6.50	103.00	2297.97	3460.71	580.00	5.00	-
C-2	-	-	1313.00	2577.90	884.10	222.00	-
C-3	-	6.15	623.25	3592.16	170.27	8.17	-
C-4	-	-	1775.77	3158.61	1538.91	372.41	336.8
Total	6.50	109.15	6009.99	12789.38	3173.28	607.58	336.8
Total length	23032.68 m						

There was a significant variation in the anticipated/projected and actually encountered rock mass during excavation. After Nathpa Jhakri, 23 km long HRT in Tala Project was the second longest tunnel. Due to high length, depth and difficult terrain along the alignment of the tunnel, investigations were very difficult. Prediction of geological mass was based upon the surface mapping of the exposed rock mass and exploratory drilling along the HRT alignment during preparation of Detailed Project Report (DPR). The variation in anticipated and actually encountered rock mass classified as per Q system can be seen in Fig. 2.

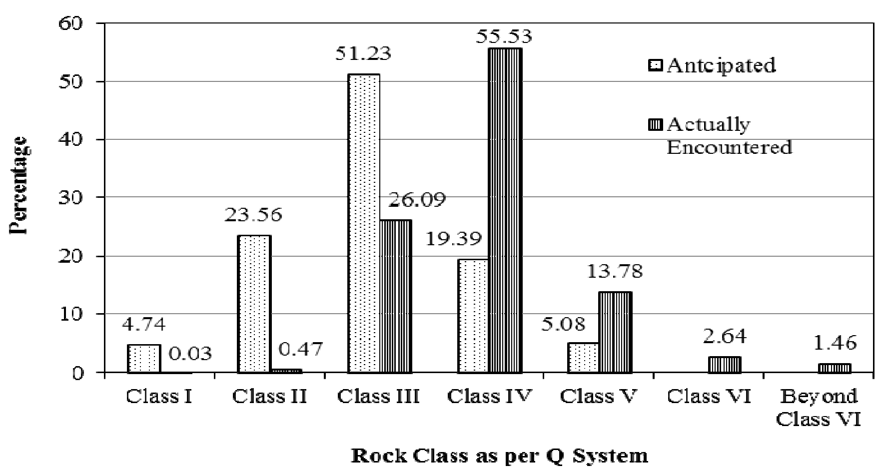

Fig. 2: Variation in anticipated and actually encountered rock mass

3.2 Excavation through Intake and Thyomachu Adits

The stretch of HRT between downstream end of desilting chambers and through Thyomachu adit (Package C1, length 6.4 km) was driven largely through hard and jointed high grade metamorphic rocks such as biotite-gneiss, augen gneiss and quartzo-felspathic gneiss. Because of unweathered compact rock mass without major water seepage, this portion was excavated without much problem and ahead of schedule. Only 26 steel sets (ISMB 250@1000mm c/c) were provided in this initial reach.

3.3 Excavation through Padechu Adit

Excavation of HRT (Package C2, length 5.0 km) was driven through two faces opened from Padechu adit having length of 260 m. The adit excavation faced severe geological problems particularly from 110 m onwards through highly weathered and water charged schists and gneisses. The pace of excavation remained extremely slow for about 580 m length on account of very poor to extremely poor rock mass conditions and water seepage. The rock mass in this reach consists of slightly to moderately weathered, highly sheared, folded, jointed, wet and thinly foliated quartz-biotite-schist with occasional bands of quartzite and thinly to moderately foliated biotite-gneiss. From chainage 580 m onwards, the rock mass registered slight improvement , though shears, water seepage and over break in the crown continued up to chainage 610 m. Improvement in rock strength remained more or less consistent throughout the upstream length except some intermittent patches affected by minor shears and water seepage requiring steel rib support. The excavation in the downstream face mostly encountered thinly foliated quartz-biotite-gneiss and quartz-biotite-schist affected by weathering, intensive shearing, jointing and water seepage in the initial length of 350 m. Rock mass has been classified mostly under IV, V and VI necessitating rib supports. From chainage 350 m, conditions improved and excavation progressed satisfactorily.

3.4 Excavation through Geduchu Adit

Excavation of 4.4 km length of HRT through two faces from Geduchu adit was carried out in mostly in class III and IV rock mass comprising of quartz-biotite-gneiss and quartz-biotite-schist, biotite-schists with few patches of rock mass of class II and V. Folded and warped quartzite-biotite-schist, biotite-schist and frequent interbands of biotite gneiss, quartzitic gneiss and quartzites dissected by shears and joints comprised the rock mass in upstream reach. The downstream reach of tunnel was negotiated through folded, jointed, quartz-biotite-gneiss and schist with frequent interbands of quartzite and quartzitic gneiss. Rock bolts with fibre reinforced shotcrete was provided as rock support. Steel ribs ISMB 250 at 1000 mm c/c with backfill concrete were provided in curved portion under Geduchu nallah.

3.5 Excavation through Mirchingchu Adit

Mirchingchu upstream was excavated through very thinly to thinly foliated quartz-biotite-schist, biotite schist, occasional quartzite bands and quartz veins (boudinised) parallel to foliation. Late dilation of rock mass in upstream reach between chainage ±1340 and 2214 m was observed. Appearance of cracks in SFRS and bending of face plates of rock bolts was a common phenomenon. Wedge failures also took place at many places. Repeated scaling with fresh application of SFRS and providing additional rock bolts was tried to tackle the problem. Size and thickness of face plates were also increased which helped only in slowing down the problem. Ultimately 7 to 8 steel ribs (ISMB 250 at 1000mm c/c) were provided intermittently to stabilize the rock movements.

The downstream reach through Mirchingchu adit was negotiated through thinly to moderately foliated, moderately to highly weathered quartz-biotite schist, quartz-biotite-gneiss and biotite schist with frequent intercalation of grey quartzite and foliation parallel quartz veins (boudinised). The reach was more structurally disturbed as signatured with smooth, planar, clay filled foliation joints along with three prominent joint sets. Many shears parallel and across the foliation were frequently encountered. Some of these cross shears were 3-4 m thick with affected zones reaching up to 8-12 m. These major shears were associated with heavy water seepage of the order of 250-500 lpm.

Excavation of 328 m length faced immense geological problems through highly sheared and shattered grey quartzite, quartz-biotite-gneiss/schist and amphibolite in class IV, V, VI and beyond class VI rock mass leading to very slow progress (Jeur et al., 2003).

3.6 Excavation through Kalikhola Adit

Excavation from Kalikhola upstream face was progressing through fair tunnelling media up to chainage 618 m (Q = 4.1 to 9.2). Moderately jointed quartz-biotite-gneiss, quartz-biotite-schist with amphibolite and quartzite bands were encountered with more or less dry conditions. After 618 m chainage, rock mass slightly deteriorated (Q = 1.6 to 2.5) because of unfavourably oriented joints in association with minor water seepage causing wedge failures. Highly shattered, moderately to highly weathered, foliated interbands of quartzite, amphibolite and biotite schist with 10 to 50 cm thick foliation shears started appearing from chainage 638 m. The rock mass was highly water charged. Progress became very slow from 660 m chainage on account of interception of these ground water pockets resulting into flowing condition at many locations (Q = 0.05 to 0.14). Between chainage 697 m and 709 m, the rock conditions further worsened due to intersection of these foliation shears with a thick cross shear. The reversal of dip between chainage 703 m and 706 m was attributed to the presence of monoclinal flexure along which faulting has taken place. The downstream portion (1592.5 m) encountered more or less similar lithology with a few stretches dominated by muscovite-sericite schist. However, from chainage 865 m, the schist gradually changed over to drastically different rock type i.e. augen gneisses which continued up to surge shaft junction.

4. EXCAVATION AND SUPPORT MEASURES IN HRT

The design of support systems for underground excavations depends mainly on the purpose, shape and size of the excavation, nature and extent of the discontinuities present in the rock, mechanical properties and water ingress. The tunnelling method also plays an important role in type and magnitude of support systems. Design reviews are necessary during the process of excavation on the basis of encountered rock mass and the instrumented data. The deformations, changes in rock stresses and seepage pressures need to be recorded and monitored without wastage of time after the blast. If this data is not recorded, it is lost forever.

The tunnel was excavated through drill and blast technology except the 337 m length through extremely weak ground conditions. Special measures such as DRESS methodology and pre-grouting techniques were adopted to tackle the highly squeezing rock mass between Mirchingchu downstream and Kalikhola upstream faces.

4.1 Measures Adopted to Tackle Shear Zone in Kalikhola Upstream (Original Alignment)

Following measures were adopted for tunnelling through difficult ground conditions in Kalikhola upstream original alignment:

- Wherever the crown was found stable and rock mass was not flowing, shorter pulls were taken followed by placing of steel ribs one by one.
- In case of flowing conditions, face was plugged and crown was consolidated by cement grout before taking next blast. This also facilitated channelization of seepage water. Due to inherent feature of the rock mass, grout intake was also poor.
- Polyurethane was also injected to isolate heavy water seepage. However, it was impracticable to support the dead load of shear mass with 10-15cm thick polyurethane layer.
- An umbrella with self-drilling hollow core anchors (MAI, Australia) 51 mm diameter and 12 to16 m long spaced at 300 mm c/c was also attempted.
- Long pipes up to 20 m length were also inserted in face to facilitate free drainage and to dissipate the pore water pressure ahead of the face.
- Tunnel face was tried to consolidate with bamboos pilings (Fig. 3)

Fig. 3: Bamboo piling at HRT

- One drift with 2 m diameter was also attempted through the face at its right side for negotiation through smaller diameter tunnel, to release pore pressure and to drain out water.
- Micro-fine cement was also attempted for grouting.
- With Puntel forepoling machine becoming available, umbrella of 6 m to 12 m diameter pipe forepoles were installed by this machine followed by jet grouting from chainage (ch.) 699 m to 709 m.

All these measures turned out to be futile and the tunnel finally collapsed from face at ch.709 m on 16th July 2002 and 70 m already excavated HRT was filled with about 3500 m^3 of fallen muck. After collapse, it was, thereupon decided to detour the HRT from RD 607 m at 45^0 from the original alignment and 110 m inside the hill abandoning 102 m of already excavated reach. However, similar extremely poor rock mass conditions designated as Adverse Geological Occurrence (AGO) were encountered in the strike continuity from ch. 1008.25 m in Kalikhola upstream (u/s) and from ch. 2030.55 m in Michingchu downstream (d/s). The diverted alignment of HRT is shown in the Fig. 4.

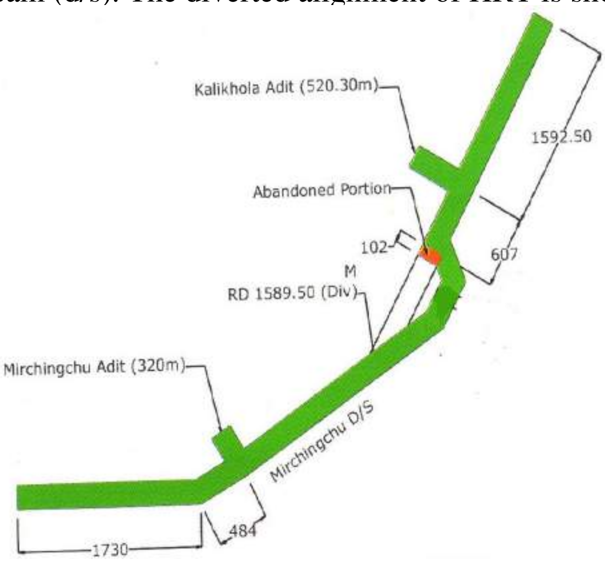

Fig. 4: Detoured alignment of HRT

4.2 Measures Adopted to Tackle Shear Zone in Kalikhola Upstream (Diverted Alignment)

After diversion of HRT, excavation was carried out with caution restricting the pull to 2 m irrespective of the rock mass class. At chainage 1005 m, a foliation parallel shear zone appeared at the crown dipping 40^0 to 45^0 with the drive and caused some over break. Though steel ribs were installed at a spacing of 0.5 m, but buckling and squeezing of the ribs was observed with time. This was due to the dead load of sheared rock mass comprising completely pulverised bands of quartzite, amphibolites and quartz-biotite-schist intermingled with clayey mass. Invert struts were also installed to counter the wall pressure. Buckling of ribs continued and ultimately 5 ribs from the face collapsed. Approximately, 15 m^3 of muck filled the tunnel up to crown (Fig. 5).

Concrete could not be placed in the cavity and cement grouting had to be resorted ultimately. Two stage grouting was adopted. In the first phase, thirteen 8 m long and 51 mm diameter MAI anchors were inserted along the tunnel periphery at an upward angle of 20^0. Grout (cement water mixture) was injected through these anchors. In the second phase, 27 numbers perforated MS pipes (8 m long and 50 mm in diameter) were inserted and grouting was done through them. A total of 146 MT of cement was injected through MS pipes and anchors. This grouting was helpful in filling up of the cavity as well as consolidation of the surrounding rock mass (Fig. 6). The damaged ribs were replaced one by one carefully.

Fig. 5: Collapse in diverted alignment

Fig. 6: Treatment of collapse in diverted alignment

Sample of the excavated material was sent to Central Soil and Materials Research Station (CSMRS), New Delhi for testing and classification. The test results indicated that the particle size larger than 4.75 mm (gravel) is in the range of 23.8 % to 29.7 % only. However, few stray boulders of 300 mm size were also found in the excavated material. The unconfined compressive strength of undisturbed samples varied from 0.141 kg/cm^2 to 0.303 kg/cm^2. As per International Society for Rock Mechanics (ISRM) suggested method (ISRM 1981), the material with unconfined compressive strength below 1.0 MPa i.e. about 10 kg/cm^2 should be considered as soil. The material was classified as SM (Silty Sand) by Babbar (2004) after testing at CSMRS. At site also, the rock mass was designated under AGO which is beyond rock mass class VI as per Barton's 'Q' classification system.

4.3 Excavation by DRESS Technique

After the collapse in the diverted reach, 337 m length of HRT was driven in extremely weak rock mass conditions and 298 m was executed using **Drainage, Reinforcement, Excavation and Support System** (DRESS) technique between Mirchingchu d/s and Kalikhola u/s. The 'DRESS' technique mainly involved the following steps:

Drainage: Drainage holes were drilled ahead of the face to eliminate the detrimental influence of water pressure above the shear zone. Drainage holes varied in length from 4 m to 27 m and diameter from 76 mm to 114 mm. Rock mass ahead was also assessed while drilling drainage holes by the nature of rock cuttings, rate of penetration and cutting sound.

Reinforcement: After installation of drainage system, face stabilization was done by putting wire mesh with shotcrete or SFRS to enable safe and stable working conditions. Perforated steel pipe forepoles (casings) of 114 mm outer diameter and 6 mm thickness in an upward direction of 6^0 to 8^0 were installed with the help of hydraulic drilling rig. The forepoles were spaced at 200 - 300 mm centre to centres. Afterwards, grouting was carried out systematically in a proper sequence starting from the far end of each pipe using a mechanical packer. The packer was retracted after every one meter once a pressure of 35 - 40 kg/cm^2 developed. Pipes were made solid by cement grout after completion of pressure grouting. As forepoles were perforated, grout also filled the annulus space between pipes which were very well observed during excavation stage.

Excavation: After face stabilisation, drainage holes, shotcreting and grouting, the tunnel advance in one forepoling block was carried out in variable diameter of excavation before the next block of forepoling. An advance of 6.75 m to 10 m was made under one forepole umbrella. Tunnel above spring level was excavated with hydraulic rock breaker in form of half ring/grove leaving central portion to brace the face and walls against bulging and collapse. Immediately after finishing excavation, a protective layer of 50 mm thick SFRS was applied in the excavated crown and face. Then space was made below spring line either by chipping or by taking minor blast depending upon strength of rock mass. Ribs were then placed and space between rock surface and rib intrados was filled with SFRS.

Support Solution: In every stage of forepole umbrella, first rib was installed at minimum excavation line and subsequent ribs were provided in a variable progressive excavation line radius to facilitate forepoling machine working in next stage of forepoling. By providing the incremental rib profile, a step of 45–55 cm in height and 30–50 cm in radius was gained. Three different steel rib sections ISMB 250x125, ISMB 300x140 and ISMB 350x140 were used as rock supports at spacing of 350 mm, 400 mm and 500 mm c/c respectively. Once 4 to 5 ribs were installed, invert struts were also placed making it a closed section to counter movement of side walls and upheaving of invert. View of forepole umbrella arch with drainage holes in the face is shown in Fig. 7. Summary of the forepoles are given in Table 2.

5. SUPPORT SYSTEM ADOPTED IN HRT

The rock support in underground structures includes shotcrete, rock bolting, self-drilling anchors, steel ribs, grouting and final plain concrete/RCC lining. In HRT construction, all the above supports (IS 5878 – Part IV: 1978) were used to stabilise the underground excavation.

5.1 Support System for Different Classes of Rock Mass

Rock supports were designed based on the classification of rock mass as per Q system of Barton. Contact and consolidation grouting was done to minimise the seepage flow and to strengthen the surrounding rock mass around the tunnel. Finally, RCC concrete lining was provided in HRT. Supports for various classes of rock mass as per Q values have been given in Table 3.

Fig. 7: Forepole umbrella with drainage holes in HRT AGO reach

Table 2: Summary of the forepole umbrellas in HRT

No. of umbrella	No. of forepoles	Grouting (MT)	Excavated and supported length (m)	No. of Ribs/ forepole	Time taken (days)
Kalikhola u/s					
21	43-47	11.60-84.40	6.00-10.10	10-21	16-46
Mirchingchu d/s					
16	33-47	9.65-70.05	6.40-9.90	14-23	21-32

5.2 Steel Fibre Reinforced Shotcrete (SFRS)

Steel fibre reinforced shotcrete (SFRS) of M35A10 grade was used for immediate support in HRT. The designed thickness of SFRS as given in Table 3 was ensured. Wet shotcreting method was employed with pre-mixed shotcrete from batching plant. Alkali free accelerating agent was added at the point of application in the tunnel.

Thickness of shotcrete was measured at random points (roughly one measurement every 100 m^2) via coring. SFRS cubes were cast for determining the compressive strength. Cores extracted out of the in-situ shotcrete were tested for evaluation of in-situ strength after cutting/capping. Cores of shotcrete with an age of 28 days should have equivalent cube compressive strength of at least 85% of the specified grade of shotcrete. The equivalent cube compressive strength at 28 days was found to be as per specification requirements (IS 516:1959 – Reaffirmed in 1999). The average compressive strengths of SFRS cores have been shown in Fig. 8.

Table 3: Support system for different classes of rocks

Q value	Rock class	Type of rock support				
		Rock bolts		Steel ribs	Steel fibre reinforced shotcrete (SFRS)	
		Pattern	Spacing mm		Shotcrete thickness mm	Extent of SFRS
100-40	I	Spot bolting		-	50	Spot SFRS
40-10	II	Spot bolting		-	50	Up to haunch
10-4.0	III	Pattern bolting	1750	-	50	Upto spring level (SPL)
4.0-1.0	IV	i) Staggered both ways ii) Pattern bolting	1750 1500	ISMB 250 750 mm c/c -	50 100	Above SPL Above invert level
1.0-0.10	V	Spot bolting	-	ISMB 250 600 mm c/c	75	Above invert level
0.10-0.01	VI	Spot bolting	-	ISMB 250 500 mm c/c	100	Above invert level

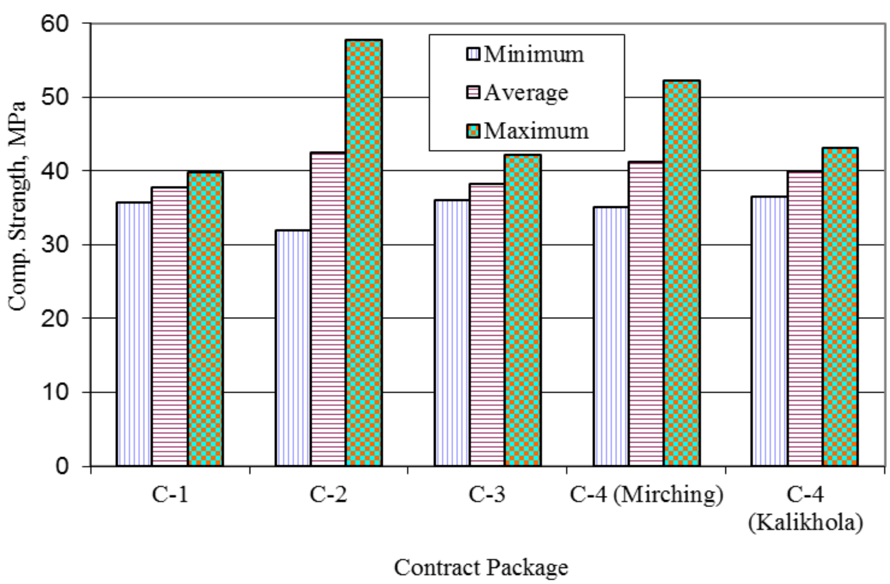

Fig. 8: Compressive strength of SFRS cores

An exercise in an experimental reach of 7.0 m was carried out in HRT to determine the percentages of additional quantity of shotcrete on extra area of over break, in depressions and on account of rebound vis-à-vis actual consumption. This exercise was also used to measure the shotcrete rebound apart from solving the contractual issues. Contract document specified a rebound of not more than 20% and 10% for alkali based accelerator and alkali free accelerators, respectively. Rebound observed in this experimental reach using the alkali free accelerator was found to be 4.75%.

5.3 Rock bolts

In HRT, 3.5/4 m long and 25 mm diameter resin grouted rock bolts were used as rock support. Steel reinforcement bars of Fe-415 grade were used as rock bolts. End anchorage length was determined after carrying out efficacy tests by varying the number of fast set resin capsules and conducting pullout tests. Based upon the results of efficacy tests, end anchorage length of rock bolts with fast set resin capsules was fixed as 1/3rd of the length of rock bolt. The remaining length of the drillhole was grouted with slow set cement capsules. The spacing of the rock bolts varied with the rock mass class as given in Table 1 and it was varied depending upon the specific geological requirements or instrumentation monitoring data.

Specified numbers of rock bolts (minimum 2% of the installed rock bolts) were tested for pullout strength in accordance with suggested methods of International Society for Rock Mechanics (ISRM-1987). The details of rock bolts installed and tested are given in Table 4. Failed rock bolts were replaced and tested again for pullout strength. Rock bolts were selected randomly for pullout test so as to keep the applied load as axial as possible. Variation of 5 to 10 degrees was tolerable.

Table 4: Details of rock bolts installed and pull out tests conducted

Description	C-1 Package	C-2 Package		C-3 Package		C-4 Package	
Total number of rock bolts installed	Total	Total	After Sep 2002	Total	After Jan 2002	Total	After Mar 2002
	33753	14572	3638	23287	3606	28303	5472
Pull out Tests Conducted	507	146	91	285	81	337	148
Rock bolts Failed	21	15	13	3	3	14	6
Rock bolts Tested (%)	1.50	1.00	2.50	1.57	2.25	1.19	2.70

Displacements of the order of 8-10 mm were generally observed after applying 12 tons load for 25 mm diameter and 4 m long rock bolts. A typical load versus displacement curve for pull out strength test on a few rock bolts is shown in Fig. 9.

An increase in percentage of rock bolts tested was observed after deployment of sufficient quality control engineers as given in Table 4. The tensile properties of the rock bolts as per IS 1786 (1985) were also evaluated from time to time.

5.4 Steel Rib Support with Backfill Concrete

Steel ribs were used to support the HRT with M15A20 grade of backfill concrete for rock classes IV to VI wherever found necessary. Invert struts were also provided in extremely poor rock mass conditions so as to form closed section to counter the wall pressures and upheaval. Bending of steel ribs was observed at some critical sections. Because of the considerable wall pressures, upheaval took place resulting into bending and cracking of invert struts in AGO reach. These damaged invert struts were replaced before start of concrete lining in this AGO reach (Fig. 10). Invert struts were welded properly to the ribs as shown in Fig. 11. Backfilling of the ribs and the rock interface was given due importance to prevent concentrated loads on the steel ribs which may otherwise result in distortion of steel ribs. Tensile properties of steel ribs were evaluated in accordance with IS 2062 (1985).

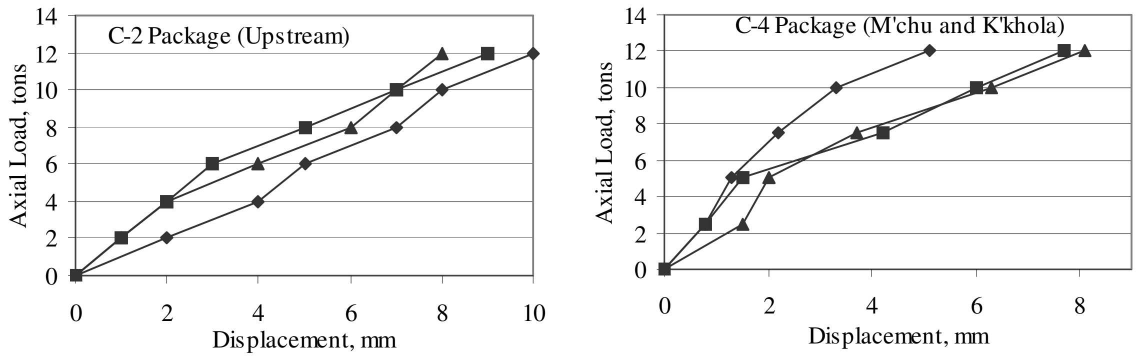

Fig. 9: Load versus displacement plot for pullout test on rock bolts

Fig. 10: Broken/replaced invert struts

Fig. 11: Invert struts welded to the steel ribs

Due to poor geological strata in 610 m length (540 m on the upstream and 70 m on downstream side of junction of HRT with adit) in Package C-2, the ribs got buckled due to excessive rock loads. This was supplemented by the instrumented data of total station targets and load cells. Columns of some of the ribs got distorted into ‘S’ shapes. The lining of HRT with original finished diameter of 6.8 m was very difficult because of the convergence of walls in this particular reach. In view of this, design of HRT lining was reviewed. *It was decided to reduce the tunnel diameter to 6.5 m in this stretch. Also, the damaged ribs were required to be replaced before concreting so as to protect the lining from the excessive rock loads in the squeezing ground condition. It was decided not to destabilise the crown arch section of ribs. The invert was lowered up to 35 cm for providing minimum thickness of concrete lining. The ribs required to be rectified were identified. The crown arch of the ribs was supported with horizontal struts. Then the buckled columns of ribs were replaced by new sections by removing one column at a* *time. A total of 512 ribs (442 in upstream and 70 in downstream) were rectified. Apart from this, 41 additional ribs were also provided at critical locations. The gap between the rock surface and ribs was backfilled with M15A20 grade concrete for proper load distribution on the ribs.*

5.5 MAI Anchors

Hollow core self-drilling anchors of 12-16 m long were also provided in the squeezing rock mass conditions in Kalikhola and Mirchingchu for face stabilisation as well as countering the side wall pressures. These anchors have been used as means of draining the rock mass and also for grouting to improve properties of the rock strata (Bhandari et al., 2004). These self drilling anchors were very effective in channelizing the seepage because of hollow core. For anchoring steel arches, anchors of 8 m length were installed in a systematic pattern. The locations indicating rising trend of rock loads were provided with 12 m long anchors. Photograph of the hollow core self-drilling anchors and the load-displacement curve for pull out test on these anchors are shown in Figs. 12 and 13 respectively.

Fig. 12: Hollow core self-drilling anchor

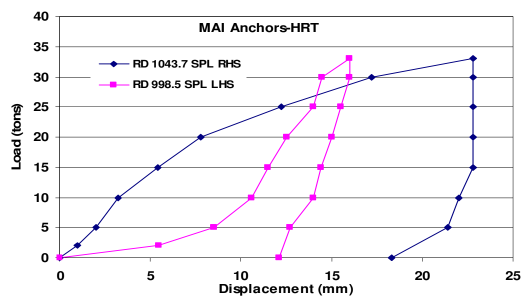

Fig. 13: Load versus displacement plot for pullout test on rock anchors

5.6 Concrete Lining

Concrete lining in tunnels is essential for various reasons viz. to reduce friction losses in the system, to protect the support system viz. steel ribs and reinforcement from deteriorating, to prevent leakage of water, to prevent loose rock particles entering the turbines and to share the internal pressure not taken by the surrounding rock.

The sequence of concrete lining, grouting and other repairs adopted in HRT at Tala Hydroelectric Project is as follows:

(i) invert stage I concreting below minimum excavation line (MEL),
(ii) laying of kerb concrete,
(iii) concreting in overt section with telescopic collapsible shutters,
(iv) invert stage II concreting,
(v) contact grouting,
(vi) consolidation grouting, and
(vii) crack/honeycomb repairing in concrete, if any.

Telescopic collapsible shutters of 12/15 m length for concreting in overt section were used. M20A20 grade of concrete was used for RCC lining of HRT except in 337 m long AGO reach where M30A20 grade concrete with high workability was provided along with two rows of 32 mm diameter main reinforcement bars of Fe 415 grade steel at a spacing of 75/90 mm centre to centre. In rest of the HRT, 16 mm diameter Fe 415 grade steel reinforcement bars were used at a spacing of 200 mm centre to centre.

5.7 Grouting

In underground caverns and tunnels, the grouting is required for various purposes such as:

- filling of voids and cavities between the concrete lining and the rock mass,
- strengthening the rock mass by filling up the open joints and cracks,
- strengthening the shattered/disturbed rock mass around the excavation,
- closing up the water bearing passages to prevent the flow of water into tunnel and or to concentrate area of seepage into channel from where it can be easily drained out.

Contact and consolidation grouting was performed in HRT in accordance with IS 5878 (Part VII):1972 and as suggested by the designers.

Contact grouting involves the filling of voids between concrete lining and excavated surface of the parent rock. Contact grouting is sometimes referred to as backpack grouting also. Design of drill holes for contract grouting in HRT between concrete lining and rock mass are shown in Figs. 14 and 15 for rock classes I to IV and V to VI, respectively.

Fig. 14: Design of drill holes in HRT for contact grouting for rock classes I to IV

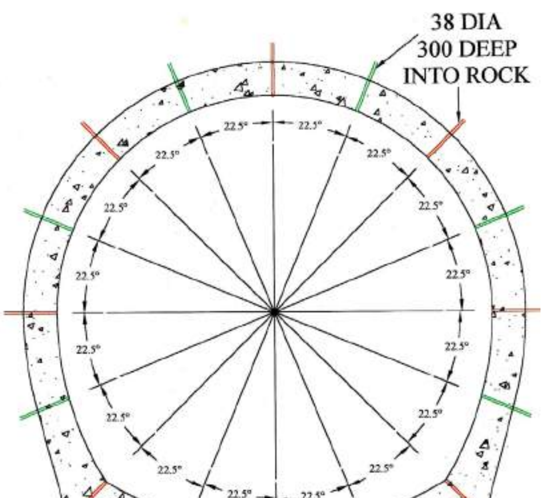

Fig. 15: Design of drill holes in HRT for contact grouting for rock classes V to VI

Consolidation grouting in tunnels is essential to reduce the coefficient of permeability and improving the deformability characteristics of surrounding rock mass around an underground opening. Consolidation grouting in the surrounding rock mass around a tunnel should be carried out up to a minimum depth of one tunnel diameter.

Consolidation grouting minimises the flow of water outward through the concrete lining into the rock mass after the tunnel has been put into service.

Consolidation grouting in HRT was carried out in 38 mm diameter and 6 m deep drill holes in the rock up to a maximum pressure of 7 kg/cm^2 (0.7MPa). Pattern and depth of drill holes for consolidation grouting as shown in Fig. 16 were governed by the geological and the design requirements.

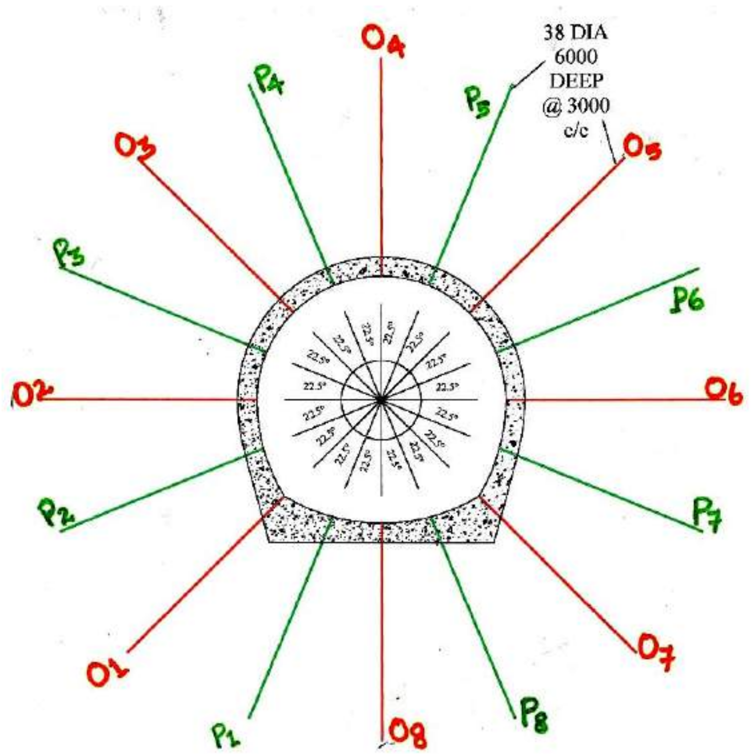

Fig. 16: Design of drill holes in HRT for consolidation grouting (all classes of rock)

The amount of grout intake in HRT depend upon many factors, viz. nature and extent of over breaks, nature and pattern of discontinuities, method of excavation, seepage in the tunnel, grout pressure, nature and consistency of the grout material/admixtures. Amount of grout intake in HRT during contact and consolidation grouting is given in Table 5.

Table 5: Grout intake in HRT

Sl. No.	Contract package	Total grout intake (cement), MT		Length (m)	Total grout intake/m, MT
		Contact grouting	Consolidation grouting		
1.	C-1	15648.53		6453	2.425
2.	C-2	3067.99	8811.25	4997	2.377
3.	C-3	8159.55	3546.9	4375	2.676
4.	C-4	11926.33	12022.6	7172	3.339

Laboratory testing was used to evaluate the properties of grouting materials such as viscosity, sedimentation, shrinkage, density and compressive strength. Field permeability tests were performed for checking the effectiveness of the grouting operation at all stages of execution (Hari Dev and Singh, 2009) before and after consolidation grouting. This helped in deciding the starter mix and special additives that might be necessary. Permeability tests were conducted in accordance with IS 5529 (Part-2):1985.

6. CONCLUSIONS

In view of the large variation in anticipated and actually encountered rock mass, geological explorations must be carried out extensively so that comprehensive and realistic information of geological formation can be obtained.

Empirical methods are only guidelines for suggesting the rock supports and therefore should be used with caution. Rock supports should be based on the design requirements and class of rock mass after detailed geological mapping. Field experience of engineering geologists or engineers at construction site plays a significant role in support selection.

Field tests on rock bolts and SFRS must be conducted for which specialised manpower is required. The frequent field and laboratory tests on the SFRS, rock bolts and constituents/ingredients result in implementation of proper support system. Proper mix design with optimum proportions of ingredients can lead to excellent results in terms of strength and rebound characteristics of SFRS application.

Efficacy tests on resin-grouted-rock bolts conducted for optimum anchorage length can result in economisation in terms of cost and time. It is emphasised to install primary support system before elapse of stand-up time of rock mass so as to avoid excessive deformations. Support systems must be installed properly following installation procedures.

Invert struts must be welded/joined to the main members so as to form a closed section. Backfilling of space between outer flange of ribs and rock surface completely with lagging, concrete blocks, shotcrete or concrete is very important for uniform distribution of rock loads on the steel ribs and to avoid concentrated loads. DRESS technique was used.

Proper grouting of the surrounding rock mass is essential for sealing of cracks/joints and other discontinuities present in the geological strata. This will not only help in minimising the seepage of water from tunnel to the geological mass but will also improve the deformation characteristics of rock mass. Permeability tests conducted before and after consolidation grouting proved to be an important tool for checking the efficacy of grouting.

Specialised equipments, such as forepoling machine, should be available to combat any eventuality in tunnel excavation particularly in Himalayan region where large variations in rock mass is observed. This will help in saving time as well as cost.

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