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Stability Challenges and Remedial Practices in Himalayan Hydropower Tunnels – A Review

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Abstract

The instability in tunnels is mainly affected by geological anomalies, rock mass quality, complex geological structures, active tectonics, and stress anisotropy. This review article presents challenges associated with stability and applied remedial measures prevailing in hydropower tunnels in the Himalayas. The review covers nine hydropower tunnels located in different parts of the Himalayas. The review found that rock bursting/spalling frequently occurs when the tunnel passes through a high overburden with good rock mass quality. On the other hand, plastic deformation (squeezing) occurs when a tunnel passes through the weak and schistose rock mass. It has been found through the review that the tunnel crew was able to successfully solve instability challenges. Effective planning, design, and selection of appropriate construction techniques help to complete tunneling projects in the Himalayas.

Keywords: Geological Anomalies; Himalayas; Stability; Stress Anisotropy; Tunnelling

1. Introduction

Tunnels are fundamental infrastructure for the development of a mountainous nation. In the recent decade, tunnel construction activities have been advancing with new and appropriate construction techniques. However, complex geological conditions and their effects are still challenging for rock engineers to ensure the stability of tunnels [1-2]. In the Himalayan region, accumulated stresses are released due to active tectonic movement. This process causes rock mass to shear leading formation of faults, folds, and weakness zones [3]. Moreover, this activity leads to an increase in the anisotropic condition of rock mass where the geo-mechanical parameters are significantly changed within the short range. Thus, tunneling in the Himalayan region is challenging and often encounters tunnel instabilities [3-4].

The natural condition of the ground is highly influenced by the excavation of underground structures. After excavation, the periphery of the tunnel contour is influenced by redistributed in-situ stresses and generated tangential stress. These stresses are governed by the shape and size of the underground structures. If the magnitude of tangential stress (induced stress) is more than the rock mass strength, it leads the instability in the periphery of the tunnel contour. Thus, the extent of deformation depends on the rock mass strength, mechanical properties of rock mass, and in-situ stress [3-5].

Stress-induced instability is fundamentally classified into rock spalling/rock burst and large plastic deformation or tunnel squeezing [5]. According to Cai and Kaiser [6], rock spalling/ rock burst happens if the deep tunnel passes through the unjointed hard rock mass. On the other hand, large plastic deformation (squeezing) mainly occurs if the tunnel passes through weak, soft, sheared, thinly foliated/bedded, and highly schistose rock mass [3].

This manuscript reviews nine Hydropower projects located in different parts of the Himalayas in Nepal, India, and Pakistan. These projects were excavated using both drill and blast (DB) method and tunnel boring machine (TBM) methods. The aim is to identify the major tunnel stability challenges and mitigation measures applied to control the instability.

2. Selection of Tunnel Excavation Method

The underground structures are constructed by different construction methods. In the Himalayan region, mostly Road Header (RD), Drill and Blast (DB), and Tunnel boring machine (TBM) methods have been used. The selection of suitable and appropriate construction methods is mainly governed by geological conditions, in-situ conditions, ground overburden conditions, rock mechanical properties,

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project design considerations, advance rate, and flexibility. In addition, ground conditions suitability, cost of the project, and risk associated with the health and safety issues may also influence the suitability of the tunnel excavation method. Many researchers agree that the DB method has higher flexibility, great adaptability, lower initial investment, and quick start-up time. Also, TBM has a high advance rate, less amount of overbreak, high capacity to reduce the requirement of support, and is significantly applicable for longer tunnels [7].

The topography of the Himalayan region is variable and steep. Suppose it is difficult to find the appropriate intermediate access point or Adit for underground excavation work. In that case, the use of TBM is more appropriate than the DB method considering that the rock mass quality is favorable. The underground excavation by mechanized equipment reduces the fracturing and disturbances in the hard rock mass and increases the strained energy storage capacity, which may lead to rock bursting. Therefore, the TBM excavated tunnel has a higher possibility of ruck burst condition than the DB method [8].

Moreover, if the tunnel passes through the squeezing ground and the TBM method of excavation is used, it may be difficult to achieve the designed tunnel geometry. Jamming of the TBM, sticking of the cutter head, and overloading in the segment lining may increase the instability. Therefore, the selection of an excavation method might play an important role to minimizing the instability induced by the squeezing ground conditions [9].

3. Rock bursting/spalling challenges and remedial measures

Rock bursting is the dynamically occurring violent failure and ejection of the rock mass due to the sudden release of accumulated elastic potential energy under excavation or disturbances [10-11]. In the hard brittle rock mass in a high-stress environment, flaky or plate-shaped fragments break off near the tunnel boundary due to the action of excavation or other dynamic disturbances, which is called spalling. The spalling shows the tensile fractures, which are parallel to the tangential stress around the excavation surface [11-12].

This section demonstrates rock bursting/spalling challenges in different hydropower projects in the Himalayan region. In addition, the applied remedial mitigation measures are explained.

3.1 Parbati II Hydroelectric Power Project

3.1.1 Project Background

Parbati II Hydroelectric Power Project (hereafter referred to as PHPP – II) is a run-of-river plant with a total head of 862 m and an installed capacity of 800 MW. The power plant is located in the Kullu district in the Himanchal province, India. The project area is geologically situated in a lesser Himalayan rock formation, which is also called as ‘Kulu Window’ [5]. The project area is bounded by the Main Central Thrust (MCT). The 31 km long headrace tunnel was excavated from five different Audits using both the Drill and Blast (DB) method with an inverted D-shaped 7.6m width and the TBM method with a circular diameter of 6.8m. The rock sequences in the HRT alignment are biotite schist intercalated with a small band of quartzite schist, carbonaceous phyllite, Manikaran quartzite (relatively fresh, brittle, and massive), and schistose granite gneiss as shown in Fig. 1. The rock mass is suffered from severe deformation and significantly influenced by folded, foliated, faulted, jointed, crushed, and sheared zones. Moreover, the HRT crosses the different minor and major faults and weakness zones, which are situated between different rock formations [5, 13].

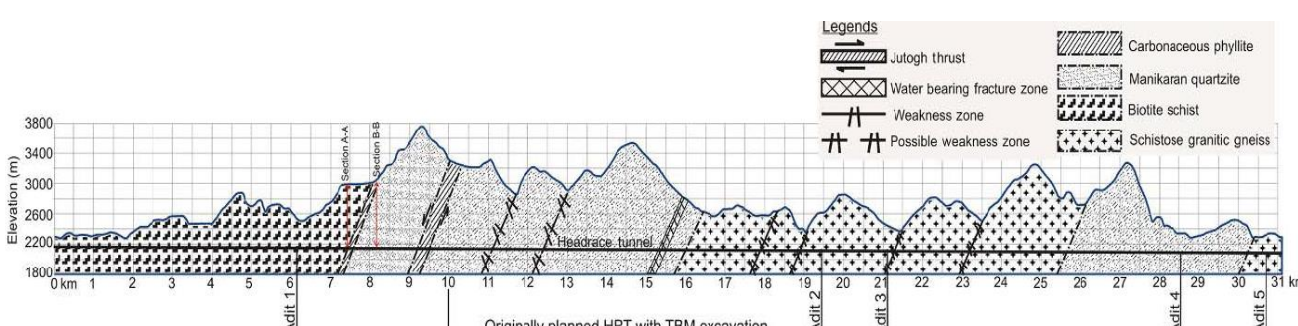

Figure 1: Longitudinal geological profile of headrace tunnel [5]

3.1.2 Challenges encountered and remedial measures

Panthi [14] applied the Norwegian rule of thumb in PHPP-II for the identification of instability situations. The author found that the headrace tunnel has valley side slopes lying between 300 to 500 up to chainage 9 km where rock cover exceeds the rule of thumb limit for almost 40% tunnel length. Up to chainage 7.1 km, the HRT exceeded the rule of thumb, however, no noticeable rock spalling/bursting occurred. Chainage after 7.1 km overburden exceeds 750 m and reaches approximately 1500m at chainage 9.25km. Minor tunnel deformation (2.5 %) was observed when the HRT crossed the band of carbonaceous phyllite at chainage between 7.4 km to 7.5 km. After 7.5 km chainage, HRT excavated by the DB method which crosses Manikaran quartzite. In this section, HRT faced several rock-bursting challenges in every blasting round, and rock-bursting continuously occurred on the valley side of the tunnel roof (Figure 2 left). Due to this rock burst event, tunnel progress was reduced to as low as 10 m per month, and crew members were injured [14]. This rock-bursting event was not an easy condition for the contractor, nevertheless, it was mitigated by the high awareness of the project team with the selection of appropriate rock support methodology. The tunnel progress was effectively enhanced by the installation of steel fiber shotcrete and rock bolts [5]. HRT tunnel was also excavated by the TBM method from a chainage of 19.46 km (from Adit 2). It was observed that tunnel progress was quite good, however, minor squeezing events were observed from chainage 16.02 km up to 15.99 km. After a 15.99 km chainage, the HRT alignment has encountered massive, brittle, and abrasive Manikaran quartzite. In this section, rock splitting was seen along tunnel spring line of the tunnel (Figure 2 right). This event was controlled by the installation of steel ring beams with a spacing of 0.4m, rock bolting, and steel net [5, 13].

Further, HRT encountered a highly fractured rock mass after chainage of 15.56 km where the TBM tunnel face collapsed. Concrete filling was applied to mitigate this collapse, as a result, the TBM progress was continued up to chainage 15.40 km. After this chainage, the HRT encountered high overburden (about 900 m) with the high-water bearing zone with an ingress of water discharge of 120 liter/s containing silt and sand debris of about 40 %. As a result, TBM was buried for more than a week and lost about two and a half years. This situation was controlled by using consolidation grouting with ordinary Portland cement [5, 13].

3.2 Neelum-Jhelum Hydropower Project

3.2.1 Project Background

The Neelum-Jhelum Hydropower Project (hereafter referred to as NJHEP) is a run-of-river plant that generates 969 MW of power. The project is located in the Muzaffarabad district of Azad Jammu and Kashmir Pakistan. The headrace tunnel of 28.5 km was excavated by using both DB and TBM methods. The project is situated in a Sub-Himalayan rock formation and passes through adverse folding and faulting with highly deformed geology conditions (Figure 3, left)). The intake area and tailrace tunnel are situated at the Main Boundary Thrust (MBT) or Murree Fault (MF) area. The excavation of the tunnel crossed the Muzaffarabad fault. The rock sequences in the HRT alignment are interbedded sandstones, siltstones, and mudstones. These were the most challenging geological conditions observed during the excavation of the tunnel. Initially, it was planned to construct a single HRT. However, it was

Figure 2: Rock burst event along headrace tunnel: left) damage in DB method excavated tunnel in valley side roof around chainage 8.6 km and right) damage in the TBM excavated tunnel in the spring line around chainage 15.7 km [5]

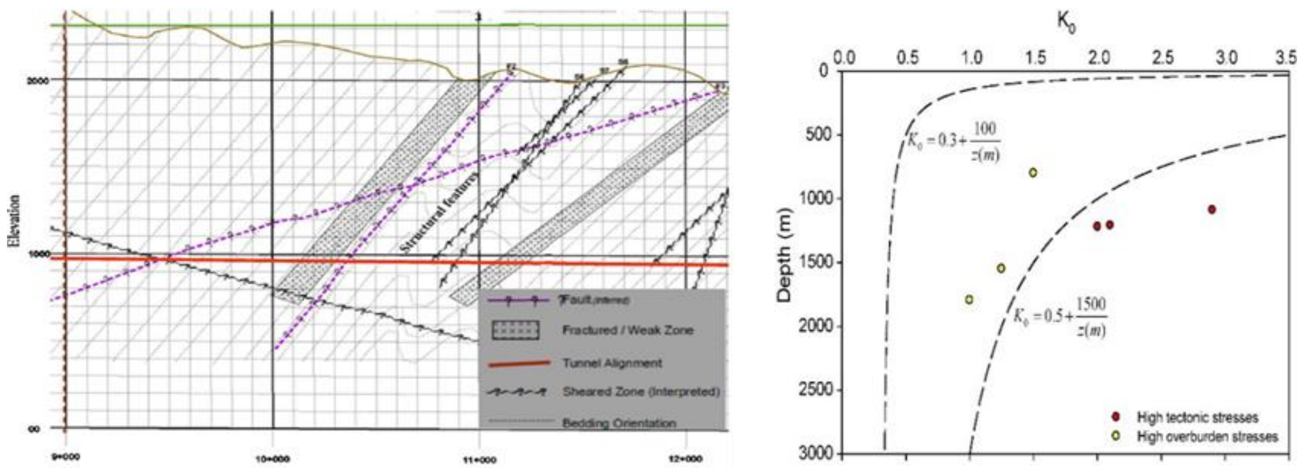

Figure 3: Geological map of NJHEP headrace tunnel (left) and Over-coring measured abnormal stresses(right) [16]

Murree Fault (MF) area. The excavation of the tunnel crossed the Muzaffarabad fault. The rock sequences in the HRT alignment are interbedded sandstones, siltstones, and mudstones. These were the most challenging geological conditions observed during the excavation of the tunnel. Initially, it was planned to construct a single HRT. However, it was finalized with twin tunnels due to high overburden (maximum overburden 1870 m) conditions. The Drill and Blast (DB) method was used to excavate the single tunnel length of 8.94 km with a 104 m^2 cross-sectional area. The remaining 19.6 km was excavated using TBM where twin tunnels with a cross-sectional area of 52 m2 were used [15-16].

3.2.2 Challenges encountered and remedial measures

In the NJHEP project, HRT excavation work has faced severe rock-bursting events. These events occurred due to unusually high horizontal stresses, geological anomalies, and strange geological settings [16-17]. Different in-situ stress measurement results were observed by Hydraulic jacking and Hydrofracturing testing. Over-coring was also conducted at different locations of the twin tunnel to measure the in-situ stresses. This method exposed very high horizontal stress, which occurred when TBM excavated tunnel crossed sedimentary sandstone beds. Also, it was observed that the stress ratio was up to 2.9 (Figure 3, right)). The tunnel encountered highly faulting and folding areas where a fault slip rock burst event was observed [16].
According to Naji et al. [16] for chainage between 9+706 to 9+793 heavy rock burst occurred on 31st May 2015. It was observed that the ring beam and wire mesh were deformed, the large wall and the crown area fell, many workers were injured and three of them lost their lives. Likewise, the TBM machine was damaged, and excavation work was stopped for more than half a year. The major cause of this rock burst was geological anomalies which was observed during face mapping and geological modeling. It was observed that the bedding plane was perpendicular to tunnel direction, which was reoriented into a transverse direction with abnormal stress concentration. Also, a rock burst (fault slip event) event occurred in this section due to high horizontal stress (k value up to 2.9). To release high-stress concentration and mitigate this event, vertical and horizontal relief holes were drilled which were insufficient to mitigate the events. Therefore, a pilot tunnel was excavated by the Drill and Blast method to release high stress, which can be effectively applied to avoid this severe rock-bursting event.

3.3 Tapovan-Vishnugad Hydroelectric Project

3.3.1 Project Background

Tapovan- Vishnugad hydroelectric project (hereafter referred to as TVHP) is a run-of-river scheme that generates 520 MW of power. The project is located in Uttarakhand India. Geologically project area is situated in the tectonically active Higher Himalayan region. In addition, Main Central Thrust (MCT) is located about 2 km south of the powerhouse area. As a result, the rock mass is highly folded, sheared, stressed, and jointed with medium to high grades of metamorphism. The project mostly passes through

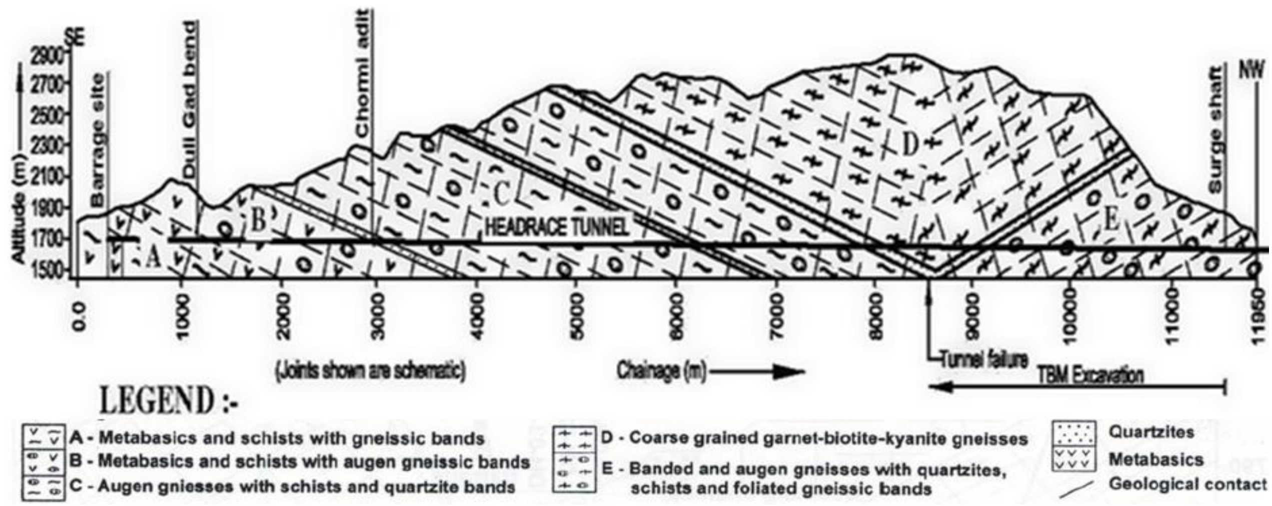

Figure 4: Longitudinal section along the headrace tunnel [13, 19]

three to four joint sets with main rock types like gneiss, quartzite, augen gneiss, and mica-schists (Figure 4). A headrace tunnel of 12.1 km was excavated by using both DB and double shield TBM methods. The DBM was used to excavate 3.6 km length and the remaining 8.6 km length was excavated using TBM with a 5.64m finished internal diameter [13, 18-19].

3.3.2 Challenges encountered and remedial measures

In December 2009 the TBM machine reached to chainage of 3.016 km and met a seriously fractured and faulted zone where the tunnel had an overburden of 900 m. At this chainage, a tunnel collapsed in front of the TBM machine. At the fracture zone, a water ingress with a high-water pressure flow consisting of 700-800 liter/sec caused several damages to the cutter head and the TBM jammed for some days. The major cause of the high ingress of water was due to occurrence of syncline fold in the fault zone. To mitigate this problem, a D-shaped (2m*2m drift) bypass tunnel (BPT) of about 180 m long was excavated to release the TBM cutter head from jamming. The ingress water was drained through BPT and the released TBM was repaired [13, 19].

4. Challenges due to Squeezing and remedial measures

Squeezing is a large plastic deformation that is influenced by the combination of tunnel excavation techniques with rock mechanical properties, induced stress, and fault/weakness zones [4]. The squeezing may occur during excavation (time-independent) or over a long period after the completion of tunnel excavation (time-dependent) [9]. The stability of the tunnel is principally influenced by the tunnel deformation limit and support capacity requirement to resist the deformation [4]. In this section, squeezing challenges in different hydropower tunnels from the Himalayas are reviewed. Also, applied remedial measures during and after the excavation of underground tunnels are summarized.

4.1 Kaligandaki ‘A’, Khimti I, and Middle Marsyangdi Headrace Tunnels

4.1.1 Project Background

Figure 5 presents a longitudinal section of three headrace tunnels of Kaligandaki ‘A’, Khimti I, and Middle Marsyangdi hydropower projects (hereafter referred to as KGA, KH, MM) in the Himalayan regions. These tunnels cross weak and schistose rock masses having stress anisotropy [4]. According to the authors, KGA has a 5.95 km long horseshoe-shaped tunnel and is situated in the lesser Himalayan region with meta-sedimentary rock formations. Likewise, the KH project has a 7.88 km long inverted D-shaped headrace tunnel bounded by Main Central Thrust (MCT). The MM project has a 5.3 km long headrace tunnel and is also situated in the lesser Himalayan with meta-sedimentary rock formations. All three projects were excavated with drill and blast methods. The Plastic deformations were analyzed in terms of both instantaneous (time-independent) and long-term (time-dependent) plastic deformation [4].

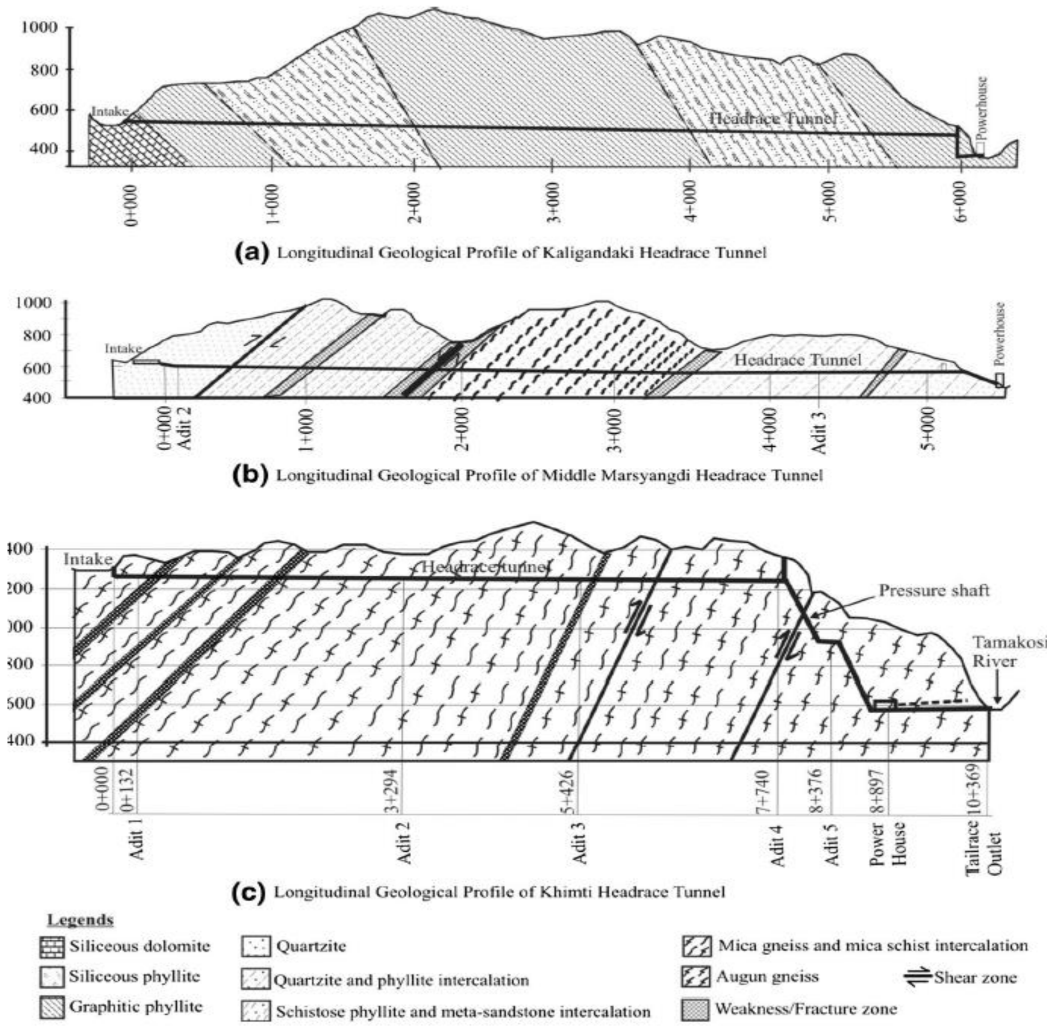

Figure 5: Longitudinal geological profile of headrace tunnel [4]

4.1.2 Challenges encountered and remedial measures

In these cases, the rock mass strength was determined by back calculation. The authors found that tunnel contour deformation was significantly influenced by anisotropic in-situ stress, rock mass deformability, and support pressure. The deformation due to the stress anisotropy was established by support pressure, vertical stress, shear modulus of rock mass, tunnel strain, and stress ratio.

After all, the authors observed that plastic deformation was significantly influenced by the shear modulus of highly anisotropy rock mass. Likewise, deformation in the roof was less than in the tunnel walls. This was due to the higher value of vertical stress components as compared to the magnitude of horizontal stress. Afterward, the authors recommended time-independent and time-dependent relations to determine tunnel plastic deformation. The authors concluded that the tunnel is the most challenging work if it crosses through weak and schistose rock mass with high anisotropy stress conditions [4].

During the excavation period, instantaneous tunnel deformation was mitigated by the installation of different primary rock supports such as steel ribs, fully grouted rock bolts, and steel fiber-reinforced shotcrete. On the other hand, large plastic deformation was mitigated by concrete lining as a final support system in KGA and MM headrace tunnels [4].

conditions is very challenging for rock engineers [3]. Figure 7 shows the longitudinal geological profile along the tunnel.
BBDM project is one of the most important projects in Nepal, which was planned to be constructed for irrigation and electricity generation (48 MW) purposes. The headrace tunnel of length 12 km with an internal diameter of 4.2 m crosses Babai Thrust in the southern part and Bhari Thrust in the middle part of the project area. Also, HRT alignment passes through the lower Siwalik (LS) and middle Siwalik (MS) formations, where the main rock types are medium to fine-grained sandstone, mudstone, and conglomerates. The starting portal of 150m was excavated by drill and blast method. After that, the machine was set up and the tunnel was excavated by TBM method [23].

4.1.3 Challenges encountered and remedial measures

Ingress of water is one of the challenges that the TBM tunnel excavation experienced. According to Panthi [23], the water ingress occurred on 27 December 2017 and 6 January 2018 at chainage of 1.175 km and 1.337 km, respectively. As a mitigation measure, the excavation process was slowed down and continuous pumping and drainage of water and intensified back-filling and plugging were done [23].
On 15 October 2018, the TBM machine was shifted and deviated by around 131 mm from the original alignment and became jammed at chainage 8.589 km. In this section, rock mass strength on the right side of the wall is harder than left-hand side. In addition, this chainage is located between the boundary of LS and MS and the alignment of HRT is parallel to the strike of the bedding plane. Due to these effects, the machine was unable to maintain proper alignment in different geological rock mass conditions and was difficult to control itself. As a result, the machine was jammed with a high thrust of 18,500 KN almost for five days, so it was unable to move. To resolve this situation, firstly bypass passage was excavated at the right side of the tunnel alignment up to the machine cutter head presented in Figure 7 (right), and after that TBM machine was removed. Also, the cutter head was jammed at the chainage 8.606 km due to high-pressure water ingress through porous sandstone ground. This type of problem was resolved by injecting the 1287 kg of polyurethane at the crown of the headrace tunnel by using a 16 m long probe hole [23-24].

4.2 Kishanganga Hydroelectric Project

4.2.1 Project Background

Kishanganga hydroelectric (hereafter referred to as KHE) project is a run-of-river hydroelectric scheme located in the Bandipora district of Jammu and Kashmir state, India. This project generates electrical power of 330 MW. The length of the headrace tunnel is 23.65 km with an overburden of up to 1400 m. The drill and blast method were used to excavate about 8.9 km and TBM was used to excavate the remaining 14.75 km length [13]. The project is situated on the western part of the Himalayan Mountain range and the Main Boundary Thrust (MBT) and Panjal. Thrust to the south (Figure 8). The rock mass has four to five joint sets along with the foliation plane [26]. Also, types of rock along the tunnel are siltstone, andesite, metasandstone, and phyllitic quartzite with overburden ranging from 400 m to 1400 m. TBM excavated tunnel portion was commenced in April 2011 and completed in June 2014 with an average tunnel progress of 400m per month [13].

4.2.2 Challenges encountered and remedial measures

The HRT crosses fault, fold, and shear zones, which were made the most challenging tunneling work.

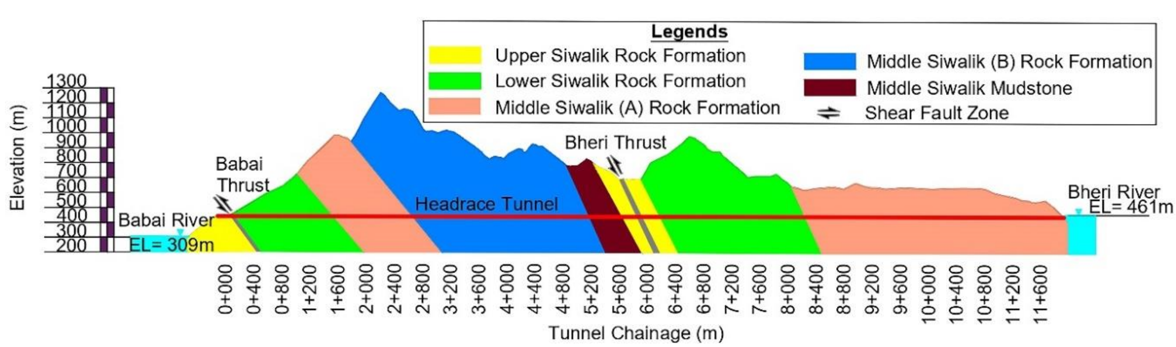

Figure 7: Geological profile along the head race tunnel of BBDMP [Revised from 23]

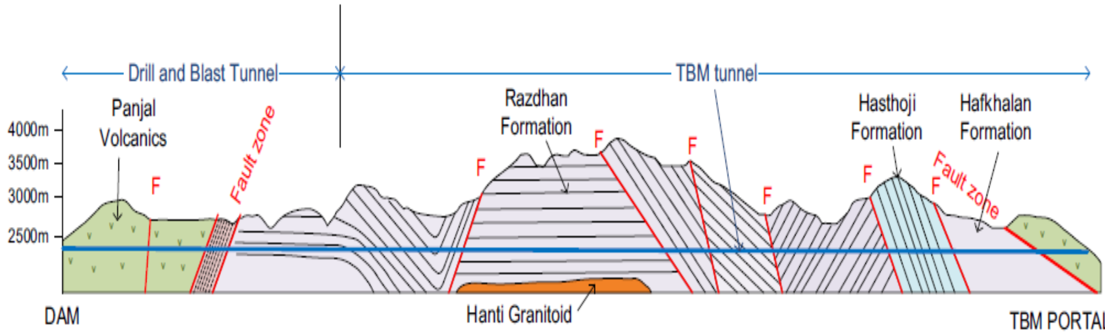

Figure 8: Geological longitudinal section of the KHE project [26]

After excavation of 1.5 km length, the machine encountered very difficult ground conditions with rock mass class V (RMR less than 20), which caused the TBM machine to jam. A bypass tunnel was constructed to release the jammed TBM. Furthermore, HRT crosses several fault zones where squeezing occurred at areas with high overburden. The stabilization was done by using consolidation grouting and the HRT excavation work was completed as per the schedule [13].

5. Discussion

In the Himalayas, the stability of tunnel excavation is mainly influenced by complex geology, stress anisotropy, and active tectonic movement. Proper geological site investigation, alignment selection, prediction of rock mass quality, and rock support requirement are considered key factors for the successful planning and designing of tunnel projects. Thus, proper planning can reduce anomalies between the predicted and actual rock mass conditions [3]. Consequently, efficient planning can optimize the overall cost and completion time for the underground project.

This review has exposed major stability problems (rock bursting and squeezing) encountered in the headrace tunnels. In relatively hard rock mass, stress anisotropy, high rock stress environment, and geological structures significantly increase the possibility of rock bursting/spalling. Rock bursting/spalling in the tunnel can be mitigated by the installation of rock support consisting of steel fiber shotcrete, rock bolting, and the use of steel ring beams and steel mess. Detailed descriptions of three rock-bursting cases are presented in Table 1.

Table 1: Summary of different tunnel projects that faced rock-bursting challenges with different excavation methods and their stabilization solution in the Himalayan region

Project Name	Geological location	Excavation Method	Challenges and their causes	Remedial measures
PHPP-II	Lesser Himalayan (Rock sequences are biotite schist, carbonaceous phyllite, Manikaran quartzite, and schistose granite gneiss	-DB	-Rock burst at tunnel roof due to hard rock mass at high overburden	-High awareness of the project team with selecting appropriate rock support (steel fiber shotcrete, rock bolt) methodology
		-TBM	-Splitting along springing line due to hard rock mass with high overburden -Tunnel collapse with a high inflow of water	-Installation of steel ribs with spacing 0.4m, rock bolting, fore poling, and concrete backfilling the overbreak zone -Concrete filling and applying consolidation grouting with ordinary Portland cement
NJHEP	Sub-Himalayan (with interbedded sandstones, siltstones,	-DB -TBM	-Several rock bursting (due to high horizontal stress & geological a	- Construction of vertical and horizontal relief hole - Excavation of pilot tunnel

	and mudstones)		nomalies) damages the TBM	by Drill and Blast method to release high-stress
TVHP	Higher Himalayan (Rock types: gneisses, quartzite, Augen gneiss, and mica-schists)	DB and double shield TBM method	-Rock wedge failure with ingress of high-pressure water -Cutter head damaged and blocking the TBM	-Excavation of D-shaped (2m*2m drift) bypass tunnel (BPT)

In the rock formations where weak and highly schistose rock mass plastic deformations (squeezing) are a major challenge regarding tunnel stability in the Himalayan region. For stabilization of large plastic deformation, construction of bypass tunnel, and reshaping to the tunnel with the installation of different types of rock supports (steel ribs, fully grouted rock bolts, steel fiber reinforce shotcrete, wire mesh system) show quite good solutions. Also, the tunnel stability can be advanced by forepoling with grouting and installation of a dowel bar. Detailed descriptions of six squeezing cases are presented in Table 2.

Table 2: Summary of different tunnel projects that faced the squeezing challenges with different excavation methods and their stabilization solution in the Himalayan region

Project Name	Geological location	Excavation Method	Challenges and their causes	Remedial measures
Three projects KGA, KHP, and MMHP	Himalayan regions (weak and schistose rock mass with high-stress anisotropy conditions)	DB	-Tunnel squeezing due to rock mass deformability, support pressure, and high degree of in situ stress anisotropy	-Primary rock support system (steel ribs, fully grouted rock bolts, and steel fiber reinforced shotcrete) used to control instantaneous deformation - Large plastic deformation controlled by using fully concrete lined used in KGA and MMHP, wire mesh system installation in KHP
CHP	Lesser Himalayan zone (Rock types: dolomite intercalated with slate, phyllite, black shale phyllite)	-DB in a major portion -For Poor rock mass: Conventional & sequential method	-Severe squeezing, wall convergence (1m to 2 m) & collapse of support due to high overstress and discrepancies in ground conditions	-Reshaping of the tunnel section and installation of the final lining was done after stopping the squeezing effect. -Support type R5 (with 30 cm concrete lining) and support type R6 (with 40 cm concrete lining) applied -Change the existing shape of the tunnel into circular (with steel ribs and shotcrete)
BBDMP	Siwalik region (Rock types: sandstone, mudstone, and conglomerates)	-TBM -DB used for portal excavation	-TBM machine deviated (around 131 mm) & jammed due to different rock masses in the sides of the tunnel face	-Excavation of bypass passage at right side tunnel -Injecting 1287 kg of polyurethane at the crown of the headrace tunnel by using a 16 m long probe hole
KHEP	Himalayan Mountain range (Rock types: meta siltstone, andesite, meta sandstone, and phyllitic quartzite)	-DB -TBM	-Collapsing and squeezing the tunnel -Blocking the cutting head & jamming TBM due to fault, fold, & sheared zone	-Excavation of hand-excavated bypass tunnel to remove the TBM machine -Squeezing & challenging fault zone completely stabilized by using the extensively consolidated grouting with high alertness of tunnel crew

6. Conclusions

In this review, different hydropower projects located in the Himalayan region have been presented. As highlighted, the key features like rock mass quality, in-situ stress of rock, and the presence of groundwater significantly influence the stability of underground openings. Himalayan geology is very complex which makes it difficult to observe rock mass quality conditions accurately enough during the planning and designing phase. As a result, most of the project encounters high discrepancies between predicted and actual rock mass conditions. These high anomalies increase tunnel instabilities. Consequently, a stepwise geological investigation should be performed to minimize these discrepancies. Appropriate tunnel excavation methods should be selected as per the specific project site conditions. Furthermore, a technically qualified tunnel crew and appropriate remedial solutions are required to preserve tunnel stability.

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