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Chapter 3

Tunnels in the Himalaya

R.K. Goel[1] *& Bhawani Singh*[2]
[1]*Chief Scientist & Professor (AcSIR), CSIR-CIMFR Regional Centre, Roorkee, India*
[2]*Former Professor, Department of Civil Engineering, IIT Roorkee, Roorkee, India*

"Learn from yesterday. Live for today. Hope for tomorrow.
The important thing is not to stop questioning."

– Albert Einstein

1 INTRODUCTION

Generally, a tunnel layout is first prepared and the tunneling operations are started after collecting adequate geological information of the area. Success of the tunneling operations depends upon the reliability of these geological predictions. It is easier to collect geological details when the tunnel is shallow, the terrain is flat and the rock mass is not much disturbed. In such regions the ideal approach would be first to make a quick geophysical exploration to identify such features as major faults, shear zones, sand pockets, water bodies etc. Once the presence or absence of such features is established, conventional geological exploration should be planned for detailing. Such an approach would optimize the time and cost of exploration efforts besides providing useful geological information and reducing the chances of surprises.

The need for the geological information becomes all the more valid in a tunnel where the cover is high, the terrain is inaccessible and the rock mass is highly disturbed tectonically below a thick forest cover. Under such conditions, drilling up to the tunnel grade is costly and sometimes impossible. Attempts to infer the geology up to the tunnel grade by extrapolating meager surface data and to plan the layout of a major tunneling project on the basis of such geological projections often lead to serious unforeseen problems. These problems sometimes lead to time and cost over-runs.

Major tunneling problems in India are encountered in the young Himalayan regions, particularly the lesser or lower Himalaya, where the geology is difficult, the rock masses are weak and undergoing intense tectonic activities resulting into major faults, folds and other discontinuities. Compared to this, in the peninsular (southern part) India where the rocks are strong and less disturbed, tunneling problems are rarely encountered.

A number of hydroelectric projects are located in the lower Himalaya. In addition, rail and road tunnels are also being constructed on a mass scale. These projects lie in the Himalayan states of Assam, Himachal Pradesh (H.P.), Jammu & Kashmir (J&K), Manipur, Uttarakhand, etc. in India. Detailed geological exploration work for all the projects throughout the country has been mainly undertaken by Geological Survey of India (GSI). Despite the best efforts of the geologists, inadequacies in the prediction of nature of the rock masses, at the tunnel grade were observed in most of the tunnels in

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the Himalaya. These inadequacies led to different tunneling problems like water-in-rush, roof falls, cavity formation, face collapse, swelling, support failure, gas explosion etc. In addition, squeezing ground conditions in the weak rock masses of Himalaya have also created considerable construction problems.

Experience of TBM tunneling in the Himalaya, so far, is not encouraging. But, the success of TBM in a recently completed head race tunnel of Kishanganga hydroelectric project has certainly encouraged the morale of designers and engineers in favor of TBM. The key issues for the success of TBM in the Himalaya are highlighted.

Since the chapter is on 'Tunnels in Himalaya', a brief geology of the Himalaya is presented at first.

2 THE HIMALAYA

The Indian subcontinent is surrounded in the north by a lofty mountain chain known as the Himalaya. The Himalayan range with NW-SE general trend was formed, according to the theory of 'Continental Drift', by the collision of the Indian Plate with the Eurasian Plate. The Indian plate is known to be moving toward north at a rate of approximately 5cm per year. This collision, which began with the first contact about 40 million years ago, caused the sediments of the intervening Tethys Sea and the Indian Shield to be folded and faulted into the lofty peaks and outliers visible in the lesser Himalaya. Since the northward shift of the Indian plate is still continuing, the mountain building process is still continuing and the zone is still active seismically. On the basis of its average height from mean sea level (MSL), from south to north, the Himalayan range and the rock formations are divided as per Table 1 and shown in Figure 1. Similarly, geographically from west to east, the Himalaya is divided as given in Table 2.

Table 1 Rock formations and average height above mean sea level of Himalaya (Goel *et al.*, 1995).

Rock Formation (Broadly)	*Average Height from Mean Sea Level (MSL)*	*Popular Name*
SOUTH		
Indo-Gangetic Planes		
---------- ***Main Frontal Thrust*** ----------		
Soft, loose and easily erodible rocks, *e.g.* sandstone, siltstone, mudstones, clays and or claystones, conglomerates	Up to 1000m	Sub-Himalaya or Shiwaliks
---------- ***Main Boundary Fault*** ----------		
Sedimentary formations, *e.g.* slates, dolomites, quartzites, shales, claystones etc. Metamorphic formations, *e.g.* phyllites, quartzites, schists, gneisses etc.	1000m to 4000m	Lesser or Lower Himalaya
---------- ***Main Central Thrust*** ----------		
Weak sedimentary formations *e.g.* shales sandstones, siltstones, conglomerates and strong metamorphic formations *e.g.* gneisses, migmatites schists, marble etc.	> 4000m	Greater or Higher Himalaya
NORTH		

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Table 2 Sub-divisions of Himalaya from west to east.

WEST
Punjab Himalaya
Kumaon Garhwal Himalaya
Nepal Himalaya
Sikkim Bhutan Himalaya
Nefa
EAST

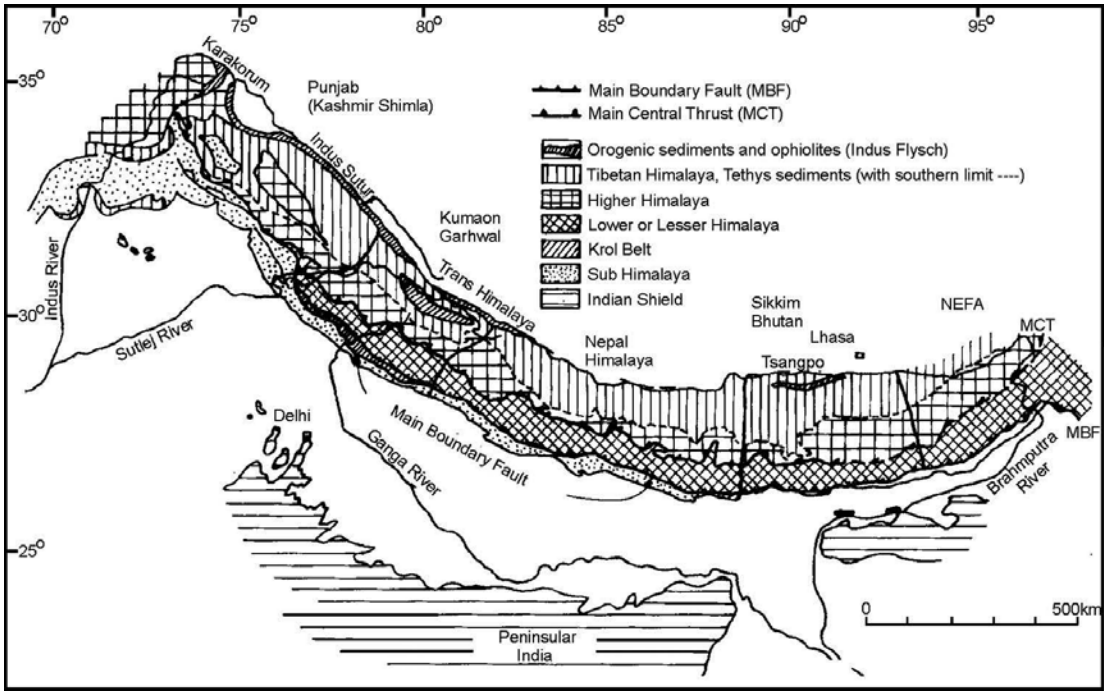

Figure 1 Longitudinal and transverse sub-divisions of the Himalaya (after Gansser, 1964).

2.1 Geology of the Himalaya

Moving from south to north, main frontal thrust (MFT) separates the Shiwaliks from the Indo-Gangetic planes (Table 1).

2.1.1 The Shiwaliks

The Shiwalik rocks constitute the southern foothills of the Himalaya. With an average height of about 1000m from mean sea level (MSL), these are generally covered with thick forests and comprise the youngest rocks in the Himalayan range. The soft, loose, and easily erodible rocks are represented by sand rocks, sandstones, siltstones, claystones, mudstones and conglomerates. Water penetrates into these rock masses along the fractures and joints and sometimes creates flowing ground conditions (*e.g.* Khara

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tunnel, Udhampur-Katra rail tunnel, etc.). Since the Shiwalik rocks are less resistant to weathering, the engineering behavior of these rocks is likely to vary with time.

2.1.2 The Main Boundary Fault (MBF)

Separating the Shiwalik Formations of the Sub-Himalaya from the older rocks of Lesser Himalaya lying to their north, the Main Boundary Fault is a major structural plane discernible throughout the length of the Himalaya. Hitherto regarded as a steep north dipping fault, it is more likely a thrust which flattens with depth. The MBF, originally defined as the tectonic feature separating the Shiwalik from the pre-Shiwalik Tertiaries, is exposed only in the extreme western sector in the Kumaon Himalaya, roughly between the Yamuna and Tons valleys. On East of the Yamuna, the higher Krol Thrust has overlapped the Eocene Subathu and has completely concealed the MBF. The only exception is seen near Durgapipal in the east where a narrow belt of Subathu is exposed between the Shiwalik and overthrust Krol rocks. Secondary faults or thrusts branch off the MBF, as for instance in southern Punjab. These secondary fault zones always diverge in a westward direction and merge with the MBF toward the east. The irregularity and sinuosity of the fault trace is evidence of a highly inclined plane. The older rocks of the lesser Himalaya are thrust over the Shiwaliks along a series of more or less parallel thrust planes. The Main Boundary Fault is a thrust fault with large-scale movements and is still very active.

2.1.3 The lower or lesser Himalaya

The lower Himalaya are separated from the Shiwaliks by the main boundary fault (MBF). The lower Himalaya are rugged mountain region having an average height of about 4000m from mean sea level. Like Shiwaliks, these are also covered with thick forests. The lesser Himalaya is made of sedimentary and metamorphic rocks. The sedimentary formations vary from weak slates to massive and thickly bedded dolomites. Limestones, quartzites, shales and claystones are also present. These are intensely folded and faulted. The low grade metamorphic rocks in the lesser Himalaya are phyllites, quartzites, schists and gneisses. The metamorphic formations are also folded and faulted (*e.g.* Chhibro-Khodri tunnel, Giri-Bata tunnel, Loktak tunnel, Maneri Stage I & II tunnels, Salal tunnel and Tehri tunnels, etc.).

2.1.4 The Main Central Thrust (MCT)

The main central thrust (MCT), marking the boundary between the lesser and higher Himalaya, is a zone of more or less parallel thrust planes along which the rocks of the Central Crystallines have moved southwards against, and over the younger sedimentary and metasedimentary rocks. It is a major feature.

2.1.5 The higher Himalaya

The higher Himalaya are separated from the lesser Himalaya by the Main Central Thrust (MCT). The topography is rugged and the average height above mean sea level

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is about 8000m. These Himalayan ranges remain covered with snow. The formations are divided into two units (a) The Central Crystallines, comprising of competent and massive high grade metamorphic rocks such as gneisses, migmatites, schists and marbles and (b) The Tibetan- Tethys Zone, composed of incompetent rocks such as shales, sandstones, siltstones and conglomerates. The rocks of higher Himalaya are also intensely folded and faulted.

The above geological description clearly indicates that the Himalayan rocks are tectonically disturbed, weak and the terrain is inaccessible. Tunnels in Himalaya have high overburden because of its great heights from MSL. Because of these features, various tunneling problems were encountered while excavating tunnels through the Himalaya.

2.1.6 Great earthquakes in the Himalaya

Many major earthquakes of differing size that have occurred during the past centuries dominate the seismicity of the Himalayan region. The major ones among them are: 1897 earthquake associated with the rupture in the south of Himalaya beneath the Shillong plateau (and the formation of the Shillong plateau, M=8.7); the 1905 Kangra earthquake (M=8.6); the 1934 Bihar–Nepal earthquake (M=8.4) and the 1950 Assam earthquake (M=8.7). In addition to these, a few more earthquakes of magnitude M > 7 have occurred during the years 1916, 1936 and 1947. During the 1991–2000 decade, three significant and damaging earthquakes with M > 6.5 have occurred in Himalaya in 1988 (M=6.6), 1991 (M=6.6), 1991 (M=6.6) and 1999 (M=6.3). An earthquake of magnitude 7.9 has stuck recently on 25th April 2015 in the north-west of Kathmandu, Nepal having widespread devastating effects.

The Himalaya is tectonically active region with number of earthquakes in the past. Weak and fragile rocks, with regional and smaller structural features have made the tunneling in the Himalaya a challenging task.

In the words of Dr. V. M. Sharma, an eminent engineer, "It is difficult to think of India, more so of Rock Mechanics in India without the great Himalaya. On the one hand, it provides an enormous source of renewable energy, and on the other, it poses some of the most difficult challenges to the Rock Engineers".

Prof. J. A. Hudson (Editor-in-Chief, Int. J. Rock Mech. Min. Science & Geomech. Abstract, 1994) once mentioned in his Editorial the importance of Rock Mechanics activities in India with these words, "To those of us who appreciate the romance and passion of Rock Mechanics, there can be no more exciting work than building structures in the Himalaya with the huge scales, the tectonic activity and the weatherability of the rocks ……. Having travelled on just a few of the roads in the foothills of the Himalaya, I have experienced the romance of this work carried out at great heights, low temperatures and in adverse conditions….."

3 TUNNELING PROJECTS IN THE HIMALAYA

The Himalaya has the tunnels and other underground openings mainly for hydroelectric projects, roads and railways. Some of the important tunneling projects are listed in Table 3. The rock type, major tunneling problems and the remedial measures are highlighted against each project in Table 3.

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Table 3 Major tunneling projects with problems faced and possible remedial measures.

Name of the Project, Name of Tunnel, Length, Size	*Rock Type*	*Tunneling Problems*	*Remedial Measures & Supports*
Ranganadi Hydroelectric Project, NE Himalaya: HRT – 8.5 km long, 6.8m dia.	Schist, gneiss, shiwalik sandstone besides mica chlorite/mica schist, granitic gneiss, carbonaceous shales and soft sandstone	Intake portal collapse, squeezing ground, intra-thrust zone, methane gas, chimney formation, roof falls and over breaks, crushed rock and flowing water from roof	Forepoling and drainage then tunnel driving, steel supports in squeezing grounds, shotcreting and rock bolting etc., changed tunnel alignment to cross main central thrust (MCT)
Dul Hasti Hydroelectric Project, J&K: HRT – 10.6km long, 7.5m dia. circular/ horse-shoe	Schist/gneiss on west, quartzite/phyllite on east; Kishtwar fault separating the two lithological units	Water ingress, cavity formation, TBM did not succeed in a smooth manner	Advanced probe holes and use of conventional DBM, use of 20mm wiremesh at crater location to stop the muck flow, filling of cavities with concrete, drainage
Nathpa Jhakri Hydroelectric Project, H.P.: HRT – 27.4km long, 10.15m dia, maximum tunnel depth up to 1300m.	Intrusive igneous & metamorphic rocks like gneiss/augen gneiss, quartz mica schist	No serious problem in tunneling except crossing a hot water zone of 52–53°C with large quantity of water	Aeration, ice in large quantities, ties at face; short shift operation; special precautions in concreting for lining; shotcreting and rock bolting
Uri Hydroelectric Project, J&K: HRT-10.5 km long, 8.4m dia.	Phyllites, graphites, shales, limestones and marble; saturated condition	Highly squeezing ground	Feeler holes ahead of drilling for advance drainage; grouting and spiling in saturated horizons; steel fiber reinforced shotcrete in layers
Tehri Project, Uttarakhand: HRT (4 nos.) – 1km long each, 8.5m dia.	Thinly/thickly bedded phyllites of various grades, sheared phyllites	Minor tunneling problems generally in sheared phyllites	Steel rib supports with final concrete lining in HRTs.
Yamuna Project, Uttarakhand: Ichhari-Chhibro tunnel-6.2km long, 7.0m dia.; Chhibro-Khodri Tunnel – 5.6km long, 7.5m dia., tunnel depth > 600m	Quartzite, slate, limestone, shale, sandstone and clays; recurrence of thrust in Chhibro-Khodri tunnel	High overbreaks, water-in-rush, Squeezing conditions, high tunnel deformations, abnormal rock loads	Heading and benching and multi-drift method; shotcreting, perfo-bolting, forepoling flexible lining
Beas Sutlej Link Hydroelectric Project, H.P.: Pandoh-Baggi Tunnel- 13.12km long, 7.62m dia	Granite with schistose bands and kaolinised pockets and phyllites	Overbreak, cavity formation, flowing and squeezing ground conditions, abnormal load, twisting of steel ribs, heavy water inflow	Forepoling, distressing by drilling advance holes at heading and benching for draining rock behind the face

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Table 3 (Cont.)

Name of the Project, Name of Tunnel, Length, Size	*Rock Type*	*Tunneling Problems*	*Remedial Measures & Supports*
Sundernagar-Slapper Hydroelectric project, H.P.: HRT-12.23km long, 8.5m dia.	Limestone, dolomite	Overbreak, cavity formation, flowing ground, heaving ground, heavy water inflow	Forepoling, draining water from behind the heading face, changing tunnel alignment
Giri Hydroelectric Project, H.P.: HRT-7.0km long, 3.7m dia.,	Slates with boulder beds, phyllite, shale, clay, sandstone	Overbreak, Squeezing conditions, high tunnel deformation and support pressure, twisting of steel ribs, occurrences of gases	Shotcreting with perfo-bolting, flexible lining, excavating tunnel of large diameter to allow deformation before final supporting, use of gas detectors
Maneri Stage-I Hydroelectric Project, Uttarakhand: HRT-8.56km long, 5.0m dia., circular, maximum tunnel depth 800m	Quartzite, metabasic, Chlorite schist, quartzite with minor slate; fault and recurrence of folds	Water-in-rush, cavity formation and high pressure because of squeezing condition leading to support failure	Tunneled through alternate alignment to avoid water-charged zone; formation of grouted zone around tunnel to tackle the highly jointed and crushed metabasics and quartzites in cavity area; heavy steel rib supports with steel lagging to tackle squeezing condition; secondary support of concrete lining
Maneri Stage-II Hydroelectric Project, Uttarakhand: HRT-16.0km long, 6.5m wide horse-shoe, tunnel depth > 1000m	Quartzite, gneisses, phyllites, greywackes, slates, limestone, epidiorite; Srinagar thrust and faults	The lithological contacts were sheared, squeezing, high pressure and deformation, flowing ground condition	Forepoling, grouting to tackle the crushed and weak rocks; cavity was grouted using bulkhead and inserting the bolts, steel rib supports with concrete backfill; excavation of bypass drift to release the water pressure
Loktak Project, Manipur: HRT- 6.25km long, 4.6m dia., circular, maximum tunnel depth 800m	Terrace and lake deposits, sandstone, siltstone and shale	Squeezing and swelling grounds, abnormal support pressures, methane gas explosion	Perfo-bolting, shotcreting and use of NATM; larger excavation size to allow deformation, excavation diameter in squeezing condition was 5.4m
Ranjit Sagar Hydroelectric Project, Sikkim: HRT- 3km long, 4.5m dia.	Phyllitic zone, intake portal at slope-wash /talus	Number of shear zones with flowing conditions	Cold bend rib supports, precast lagging, forepoling and backfill concrete

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Table 3 (Cont.)

Name of the Project, Name of Tunnel, Length, Size	*Rock Type*	*Tunneling Problems*	*Remedial Measures & Supports*
Khara Hydroelectric Project, Uttarakhand: HRT –10.2km long, 6.0m dia.	Sandstone, clay and conglomerates	Overbreak and flowing ground conditions, saturated with water	Creation of bulkhead, grouting, excavation and heavy supports of steel ribs with final concrete lining
Parbati Stage-II Hydroelectric Project, H.P.: HRT- 31.5km long, 7.0m dia., circular, maximum tunnel depth 1300m	Granite, gneissic granite and quartzite; folded, faulted and jointed	Mild rock burst, water inundation from probe holes flooding the tunnel and TBM, work stopped from TBM side, likely to resume soon.	High capacity steel rib supports were installed in drill and blast excavated section; secondary concrete lining; rock bolts, wiremesh shotcrete and hexagonal precast concrete segments were installed in TBM excavated section
Pir Panjal Rail Tunnel, J&K: 11.2km long, 8.4m wide horse-shoe, maximum tunnel depth 1100m	Silicified limestone, andesite, basalt, quartzite, sandstone, limestone, shale and agglomerates	Squeezing in shales, high deformation, roof falls,	NATM, rock bolts, shotcrete and lattice girder supports were used; forepoles in weaker rocks; thorough monitoring to evaluate the performance of supports; secondary concrete lining
Tala Hydroelectric Project, Bhutan: HRT-22.4km long, 6.8m dia., horse-shoe, maximum tunnel depth 1100m	Gneiss, quartz mica schist, chlorite, sericite schist, quartzite; rocks are folded, faulted and highly jointed; contact of two rock types sheared	Cavity formation, flowing ground, squeezing condition, other adverse geological conditions	Face supported by bamboo bolts, shotcrete, rock bolts, self-drilling anchors, steel rib supports, forepoles; face plugged to grout the roof cavity; systematic drainage; secondary support by concrete lining
Udhampur-Katra Rail Tunnel, J&K: Tunnel T1 – 3.1km long, 6.5m wide, D-shape, maximum tunnel depth 320m	Softs sandstone, siltstone and claystone; highly jointed; claystone has swelling minerals; at places the rock mass is charged with water	Squeezing and swelling conditions; high deformation and support pressure; floor heaving; steel rib supports buckled and twisted	Double steel rib supports with invert and backfill up to inner flange; Secondary support by concrete lining
Chenani-Nashri Highway Tunnel, J&K: 9.0km long, 6.0mwide horse-shoe escape tunnel and 12m wide horse-shape main tunnel, maximum tunnel depth 1200m	Sandstone, siltstone and claystone; minor shear zones	High deformation for longer period; roof falls at places	Tunneling by NATM; longer rock bolts and additional layer of shotcrete along with lattice girders have been used as primary support with final concrete lining

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Table 3 (Cont.)

Name of the Project, Name of Tunnel, Length, Size	*Rock Type*	*Tunneling Problems*	*Remedial Measures & Supports*
Rohtang Highway Tunnel, H.P.: 8.9km long, 10.0m wide horse-shoe, Maximum tunnel depth 1900m	Uniformly dipping alternate sequence of quartzites, quartzitic-schist, and quartz-biotite schist with thin bands of phyllites; Seri nala fault passes through the tunnel	Roof collapse; loose rock falls at various places; squeezing; high deformations of roof; Seri nala fault flooded tunnel with rock debris	NATM was used. Shotcrete and rock bolt supports was strengthened; longer rock bolts were used; yieldable steel rib supports are planned in poor rock conditions; DRESS technology was used to tackle the fault zone; the concrete lining will be used as final support

Notations: HRT – Head race tunnel; H.P. – Himachal Pradesh (an Indian state); J&K – Jammu and Kashmir (an Indian state); NATM – New Austrian tunneling method; DBM – Drilling and blasting method; TBM – Tunnel boring machine; DRESS – Drainage, reinforcement, excavation and systematic support

Apart from the variation in geology, the Himalayan tunnels have posed all type of challenges of tunneling, such as, face collapse, cavity formation, water-in-rush, hot water spring, gas explosion, flowing ground condition, squeezing, swelling, rock burst, etc. (Table 3). Thus, for researchers, engineers and geologists, the Himalaya provides the best field laboratory in the world where the experience and knowledge of Rock Mechanics and Tunneling Technology can be tested and established with greater confidence.

4 VARIOUS TUNNELING ISSUES AND LESSONS LEARNT

4.1 Variation in predicted and actual geology

In the Himalaya, drilling up to the tunnel grade sometimes is not possible because of high rock cover (or high tunnel depth), thick vegetation on surface, difficult and inaccessible terrain. The rocks are severely folded and faulted due to tectonic activities. Because of these reasons, the geological investigations are limited to portal areas or around the low cover zones. Hence, in number of projects, it has been observed that the geology encountered during the tunneling vary from that predicted or anticipated geology. This results in variation of excavation planning and methodology, supports type and density, etc. For example, the Chhibro-Khodri tunnel (Figs. 2a & 2b) and the Giri-Bata tunnel (Table 4).

In Chhibro-Khodri tunnel of Yamuna hydroelectric project, recurrence of Krol and Nahan thrusts have resulted in changing geology along the tunnel alignment (Figs. 2a & 2b). This resulted in the problems of water-inrush and squeezing ground conditions during tunneling through the intra-thrust zone, which delayed the project.

Table 4 shows the difference in the predicted and the actual geology along the Giri-Bata tunnel. The difference was mainly in terms of the position of faults and

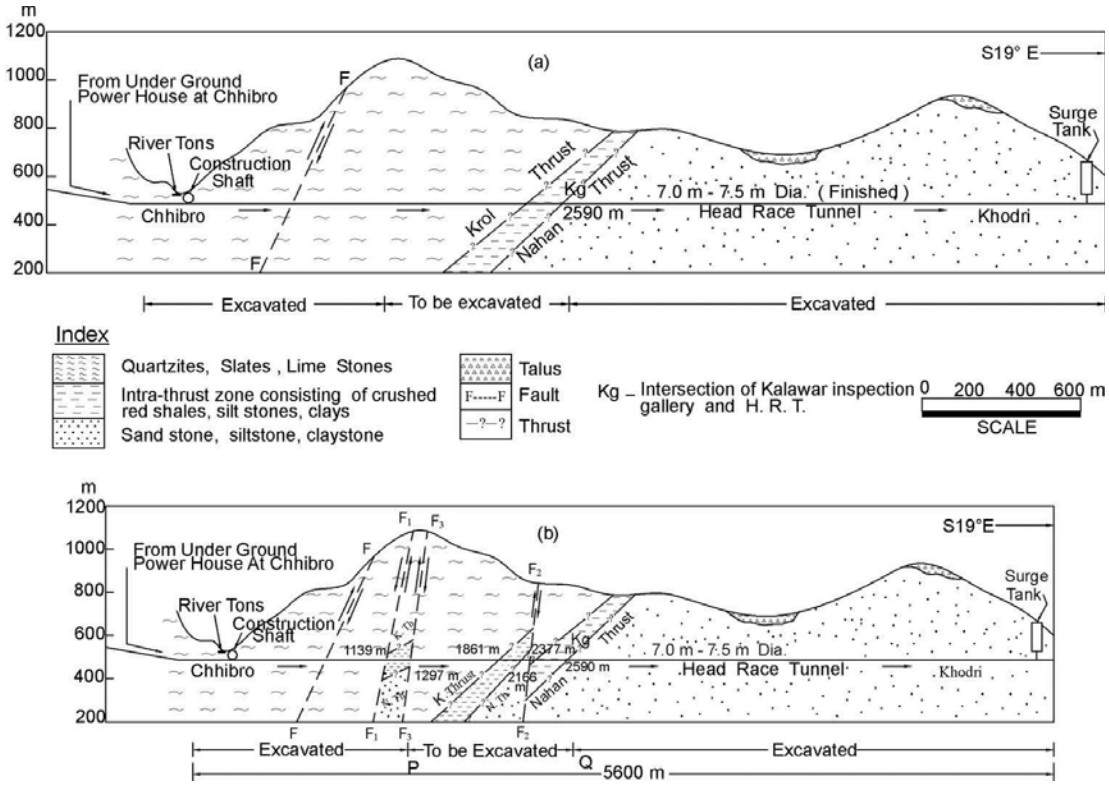

Figure 2 Geological cross-sections along Chhibro-Khodri tunnel (a) original before starting tunneling (b) actual encountered during tunneling (after Jethwa *et al.*, 1980).

Table 4 Predicted and actual geology, Giri-Bata tunnel (Dube, 1979).

Geological Features	*Predicted*	*Actual*	*Difference (m)*
1. Krol Thrust	R.D. 2780m	R. D. 3360m	580
		R. D. 2980m	200
2. Nahan Thrust	R.D. 3405m	R. D. 3520m	115
3. Sile Branch Fault	R.D. 3350m	R. D. 3196m	154
		R. D. 3236m	14
		R. D. 3266m	84
4. Marar Fault	R.D. 4959.5m	R. D. 3360m	169.5
		R. D. 2980m	89.5
5. Length of Blaini Formation	1710m	1312m	398
6. Length of Infra-Krol	1070m	1660m	590
7. Length of Dadahus	625m	384m	241
8. Length of Nahans	3710m	3593m	115

thrusts, which were struck as surprise and resulted in the delay in completion of tunnel.

Rohtang highway tunnel project in H.P. state, India is a challenging project through the higher reaches of Himalaya. The tunnel is being excavated at an altitude of more than 3000m and has the rock cover of upto 1.9km above the tunnel. While tunneling from south end, the Seri nala fault was encountered about 300m before the expected

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Figure 3 Flow of clay matrix along with highly disintegrated and weathered rock mass from tunnel face at Ch. 2077m (Rao & Sharma, 2014).

location. As per the investigations, it was extrapolated to be encountered between Ch. 2.20 and 2.80km from south end. But, during the tunneling, the Seri nala fault was struck at Ch. 1.90km, about 300m before the predicted location. At Ch. 1.918km the fault line was visible on the tunnel face where left half face is weak strata charged with water and the right half of the face is undisturbed strata.

No probe hole could be drilled to ascertain the location of Seri nala fault. Generally, it is advised to have number of probe holes in different directions to know the location of such important features. In this tunnel, as the excavation from south end progressed, Seri nala fault adversely affected the tunnel excavation and created difficult conditions for tunneling as shown in Figure 3. Finally, the tunnel through the fault zone was excavated using the DRESS method, which is found to be useful to excavate tunnel through soft, weak and water charged strata (Rao & Sharma, 2014). The DRESS (**D**rainage, **R**einforcement, **E**xcavation and **S**ystematic **S**upport) method has systematic pre-drainage ahead of face, reinforcement of ground, use of forepoles to form umbrella, pre-grouting if required, excavation in small steps by mechanical means and finally the systematic supports. The DRESS method is found to be useful to excavate tunnel through such soft, weak and water charged strata (Rao & Sharma, 2014).

The variation in the predicted and actual geology sometimes makes it impossible to tunnel along the planned alignment as discussed below.

4.2 Change in tunnel alignment

In earlier hydroelectric projects through the Himalayan rocks, in absence of the geological investigations up to the tunnel grade, the straight tunnel alignment between the inlet and the outlet location were chosen. The straight alignment, quite often, had to be changed while constructing the tunnel because of difficult and adverse ground conditions. *For example*, in the head race tunnel (HRT) of Maneri stage-I project, India the tunnel had to be diverted because of the water-in-rush and chimney formation. Three alternative tunnel alignments were considered as shown in Table 5. But,

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Table 5 Alternate tunnel alignments between Heena and Tiloth, Maneri stage-I project, India (Goel *et al.*, 1995b).

S.No.	*Proposed Layout*	*Total Length between Heena and Tiloth (m)*	*Tunnel Length through Water Charged Quartzites (m)*	*Increase in Tunnel Length (m)*
1	Original	5065	1200	–
2	Alternative I	5940	920	875
3	Alternative II	7170	–	2105
4	Alternative III	5535	920	470
5	New Alignment	5207	1600	142

finally alternative at S.No. 5 in Table 5, *i.e.*, 'New Alignment' was followed to complete the tunnel. This problem had led to time and cost over-runs. Thus, selection of a trouble free tunnel alignment is of great importance to complete the project within stipulated time and budget.

Almost similar problem was faced in the Chhibro-Khodri tunnel while tunneling in the intra-thrust zone (Jethwa *et al.*, 1980).

It has been experienced that a delay of about 20 per cent in time results in cost escalation by 35 to 40 per cent. Therefore, detailed geological and geo-physical investigations of the area are must and shall be carried out in the area where the geology is varying. In addition, the provision of probe holes ahead of the tunnel face shall also be mandatory in the contract.

4.3 Mixed and fragile geology

Experiences related to the Murree formation of the Himalaya is highlighted here to show the effect of mixed and fragile geology on tunneling. The Murree formation is represented by a sequence of argillaceous and arenaceous rocks that includes interbedded sandstone, siltstone, claystone/mudstone beds. These are also affected by minor shears.

The bands of sandstone, siltstone, claystone/mudstone of varying thickness have been frequently encountered during tunnel excavation. There is no fixed pattern of the bands of these rocks. Figure 4 shows an exposure of different rocks on tunnel face. In fact the bands of mixed rocks, for example, intermixed siltstone & sandstone and intermixed siltstone & claystone are also encountered frequently. The uniaxial compressive strengths of freshly obtained rock samples of sandstone, siltstone and claystone are 70–120MPa, 25–40MPa and 8–15 MPa respectively.

The claystone lies in the category of soft rocks. The claystone rock specimen, if left exposed to atmosphere, degrades and crumbles to small pieces in about a week's time. The freshly excavated claystone on tunnel face sometimes give deceptive appearance of massive rock or one or two joints, but after a day, it starts giving way. *Siltstone are also creating the problems where the joints in siltstone have erodible clay fillings, reducing the shear strength of the rock mass*.

Rock mass behavior because of tunneling in mixed rock masses is different from tunneling through only the poor rocks or through only the good rocks. In the Murree formations having mixed rocks, the sandstone layers are usually separated from each other by weaker layers of siltstone or claystone. Hence, rock-to-rock contact between

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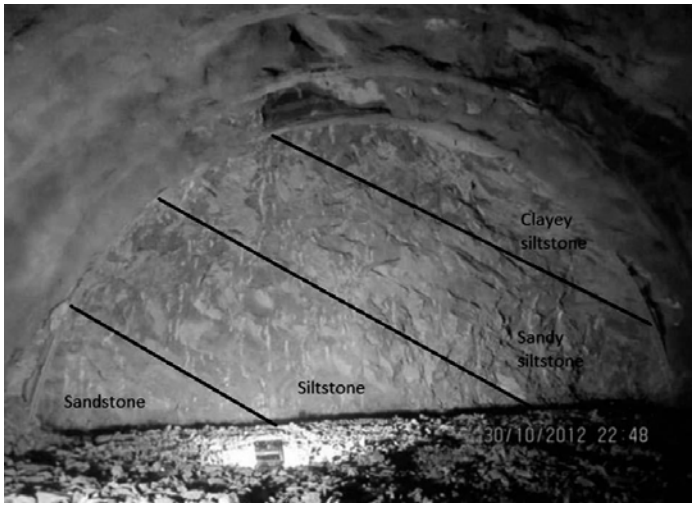

Figure 4 Photo showing exposure of different rocks on tunnel face, ch. 1546m, Chenani-Nashri main tunnel, south end.

blocks of sandstone is limited. Therefore, it is not appropriate to use the properties of the sandstone to determine the overall strength of the rock mass. On the other hand, using the 'intact' properties of the siltstone or claystone only may be conservative since the sandstone skeleton certainly contributes to the rock mass strength.

Murree formations of the Himalaya are comparable with the flysch rocks of the Alps. In order to know the uniaxial compressive strength of mixed rocks, Marinos & Hoek (2001) have proposed that a 'weighted average' of the intact strength properties of the strong and weak rock layers should be used.

Barton's rock mass quality Q (Barton *et al.*, 1974) and Bieniawski's rock mass rating RMR (Bieniawski, 1994) have wide range for different rock masses being encountered in the tunnel through layered mixed rocks. The variation in the values of Q is mainly because of the variation in RQD, J_a and SRF, whereas variation in RMR is because of variation in RQD, UCS and joint condition. In most of the cases there are three joint sets including the bedding plane plus random. In case of mixed rocks, since the rock mass behavior will vary as mentioned in above paragraphs, it is understood that the Q and RMR values shall be influenced by the per cent of different rocks. *This highlights the need of a new engineering rock mass classification for characterizing the mixed (layered) rocks.*

4.4 Squeezing ground condition

Commission on Squeezing Rocks in Tunnels of International Society for Rock Mechanics (ISRM) has published *Definitions of Squeezing* as reproduced here (Barla, 1995).

"Squeezing of rock is the time-dependent large deformation, which occurs around a tunnel and other underground openings, and is essentially associated with creep caused by (stress) exceeding shear strength (limiting shear stress). Deformation may terminate during construction or continue over a long time period".

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This definition is complemented by the following additional statements:

- Squeezing can occur in both rock and soil as long as the particular combination of induced stresses and material properties pushes some zones around the tunnel beyond the limiting shear stress at which creep starts.
- The magnitude of the tunnel convergence associated with squeezing, the rate of deformation, and the extent of the yielding zone around the tunnel depend on the geological conditions, the in situ stresses relative to rock mass strength, the ground water flow & pore pressure and the rock mass properties.
- Squeezing of rock masses can occur as squeezing of intact rock, as squeezing of infilled rock discontinuities and / or along bedding and foliation surfaces, joints and faults.
- Squeezing is synonymous of over-stressing and does not comprise deformations caused by loosening as might occur at the roof or at the walls of tunnels in jointed rock masses. Rock bursting phenomena do not belong to squeezing.
- Time dependent displacements around tunnels of similar magnitudes as in squeezing ground conditions, may also occur in rocks susceptible to swelling. While swelling always implies volume increase due to penetration of the air and moisture into the rock, squeezing does not, except for rocks exhibiting a dilatant behavior. However, it is recognized that in some cases squeezing may be associated with swelling.
- Squeezing is closely related to the excavation, support techniques and sequence adopted in tunneling. If the support installation is delayed, the rock mass moves into the tunnel and a stress re-distribution takes place around it. Conversely, if the rock deformations are constrained, squeezing will lead to long-term load build-up on rock support.

Squeezing ground conditions through weak and highly jointed rock masses under high overburden pressure (in situ stress) is quite common in the fragile Himalaya. Squeezing is mainly experienced in the argillaceous rock masses, such as, phyllites, shales, clays, soft gougy material, etc. having uniaxial compressive strength < 30MPa and the overburden pressure is high (tunnel depth is more). In most of the Himalayan tunnels, the squeezing ground condition has been experienced where the tunnel floor heaving is also common (Fig. 5).

Figure 5 Floor heaving in a railway tunnel in J&K, India.

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The support pressure developing far behind the tunnel face in a heavily squeezing ground depends considerably on the amount of support resistance during the yielding phase. The higher the yield of the support, the lower will be the final load. A targeted reduction in support pressure can be achieved not only by installing a support that is able to accommodate a larger deformation (which is a well-known principle), but also through selecting a support that yields at a higher pressure. Furthermore, a high yield pressure reduces the risk of a violation of the clearance profile and increases safety level against roof instabilities (loosening) during the deformation phase (Cantieni & Anagnostou, 2009).

4.4.1 Tunnel size and squeezing ground condition

In Chhibro-Khodri tunnel, in 1970s the main tunnel of 9m diameter was divided into three tunnels of 4.5m diameter each to avoid the squeezing purely on the qualitative consideration. Thus, by reducing the tunnel size, the squeezing condition was avoided. But, in Maneri Stage-II head race tunnel, in 1980s the main tunnel of 6.0m diameter experienced the squeezing ground condition. To bypass the problematic squeezing condition zone, a smaller size drift (2.5m) was excavated. But, this 2.5m wide drift had also experienced some squeezing ground condition. These two cases qualitatively showed that there is some effect of tunnel size on ground condition for tunnels and encouraged to develop approach for predicting the ground condition (see section 5.2.1).

4.4.2 Effect of tunnel depth on support pressure

According to the elasto-plastic theory, failure of the rock mass around an opening under the influence of depth pressure forms a broken zone called "coffin cover". The failure process is associated with volumetric expansion of the broken rock mass and manifests itself in the form of squeezing into the opening (Labasse, 1949; Daemen, 1975). The "characteristic line" – or the "ground reaction curve" – concept explains that the support pressure increases with depth, provided that the tunnel deformations are held constant. Further, *large tunnel deformations associated with expansion of the broken zone lead to reduced support pressures* (Fig. 7).

Higher tunnel deformations and support pressures observed in the red shales at a depth of 600 m at Chhibro-Khodri tunnel, as compared to those observed at a depth of 280m in the same tunnel, were explained by Jethwa *et al.* (1977) with the help of the elasto-plastic theory. They employed an empirical relation given by Komornik & David (1969) to estimate the swelling pressure and considered that the support pressure was the arithmetic sum of elasto-plastic (squeezing plus loosening) and swelling pressures. Later, Singh (1978) emphasized the interaction between the swelling and squeezing pressure and suggested that only the greater of the two should be considered. The average elasto-plastic pressures, estimated according to the suggestions of Singh (1978), are close to the observed values. As such, the empirical approaches, developed to estimate support pressure for tunnel support design, must be amended to include the effect of tunnel depth in order to obtain reliable results

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under squeezing rock conditions. The correction factor for overburden (or tunnel depth) for estimating support pressure using Q, as suggested by Singh *et al.* (1992), is accepted now. Equation 6 also shows the effect of tunnel depth (H) on support pressure.

4.4.3 Loose backfill with steel arch supports

In a deep tunnel under squeezing ground conditions, the supports are likely to attract high loads unless substantial tunnel deformations are allowed. An ideal support system for such conditions would be the one which absorbs large deformations while maintaining tunnel stability. Use of flexible supports in a slightly over-excavated tunnel provides a possible solution to such a problem. The thickness of backfill should be decided from the considerations of its compressibility and desirable tunnel deformations.

Although flexible steel arches were not used, loosely thrown tunnel muck behind steel ribs provided an element of flexibility in the Chhibro-Khodri tunnel. It was observed that the support pressure reduced to a large extent with such a loose backfill (Fig. 6a & 6b after Jethwa *et al.*, 1980).

4.5 Roof collapse and cavity formation

It has been experienced that because of frequently changing geology and presence of shear zone, support has either been inadequate or it has not been installed soon enough, which has resulted in deterioration of the rock mass quality and roof falls and cavity formation.

For example, in Maneri stage-I project head race tunnel a major cavity was formed during excavation between ch. 5038m and 5050m in highly jointed and folded quartzites. The tunnel crossed a shear zone at ch. 5050m. The crushed quartzite was also charged with water. Therefore, the crushed rock debris was continuously flowing from the roof, which formed a cavity. The total volume of the cavity was estimated as 813m^3. The face was sealed after forepoling with rolled steel joists as shown in Figure 7. A bulkhead was constructed at the tunnel face leaving 2 to 3 pipes for grouting the muck and debris. Drainage holes were provided to release the hydrostatic pressure. A side drift was also excavated to release the water pressure. The cavity above the forepoles was then filled with concrete using the pipes inserted in the cavity for this purpose. The muck below the forepoles and behind the face was grouted using a cement water slurry to check the water flow and to consolidate the muck. The tunnel then was excavated.

In yet another incident in the same tunnel, the sheared and crushed zone between metabasics and quartzites was tackled by creating grouted plug ahead of the tunnel face all around the tunnel (Fig. 8). This was then excavated and supported leaving 5m grouted zone by following the steps shown in Figure 8.

Such collapses can be avoided by pre-grouting the rock mass ahead of the tunnel and or installing the effective supports timely close to the tunnel face. Invert supports, to complete the support ring, must be used in soft and weak rock masses, thick fault gouges and shear zones.

In the lower Himalaya, it has been observed that the contact of two rock formations invariably is sheared, which generally leads to support failure and collapse. Hence, this

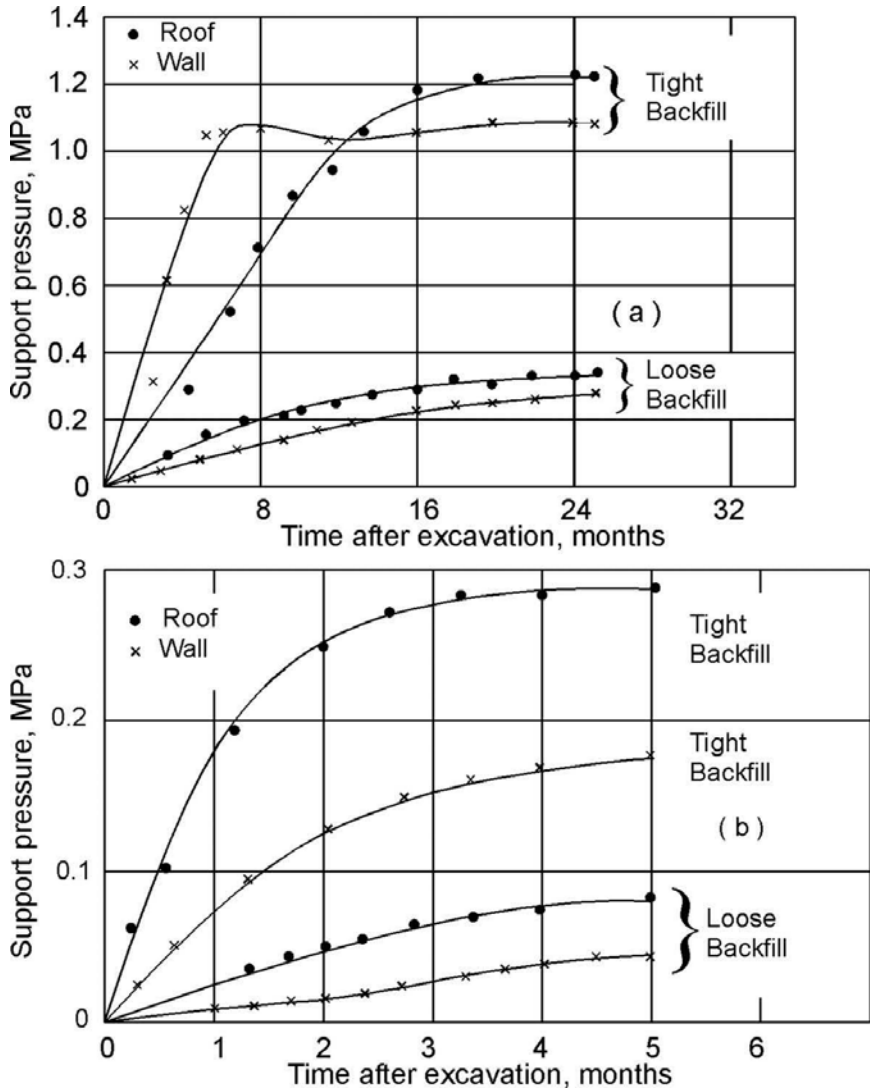

Figure 6 Effect of compressible loose backfill on support pressure in: a – black clays; b – red shales (after Jethwa *et al.*, 1980).

should be known to the geologists and site engineers so that timely preventive steps can be taken.

4.6 Shear zone treatment

There are number of small or big shear zones and faults present in the lower Himalayan rocks. These shear zones and faults are sympathetic to regional main boundary fault and main central thrust. It is generally said that in the tunnels through lower Himalayan rocks if no fault or shear is seen for 100m it means this has been missed. The contact of two rock types is also found to be generally sheared in lower Himalaya (Goel *et al.*, 1995a).

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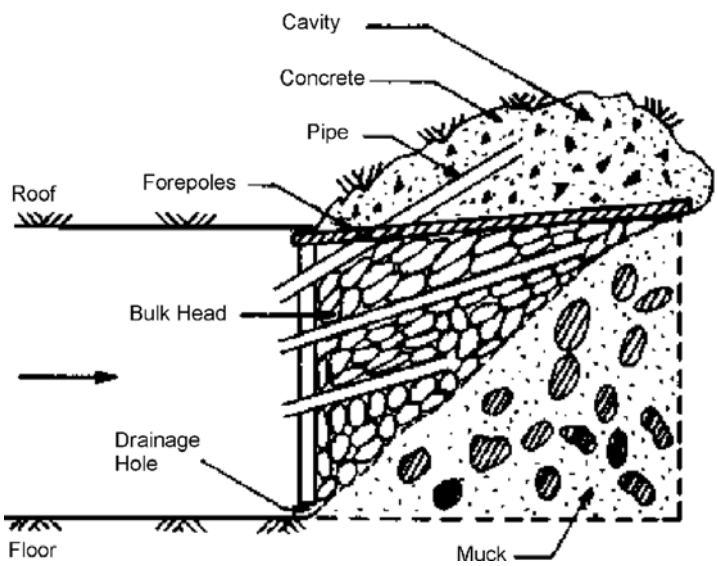

Figure 7 Method for tackling the problem of cavity formation (Goel *et al.*, 1995b).

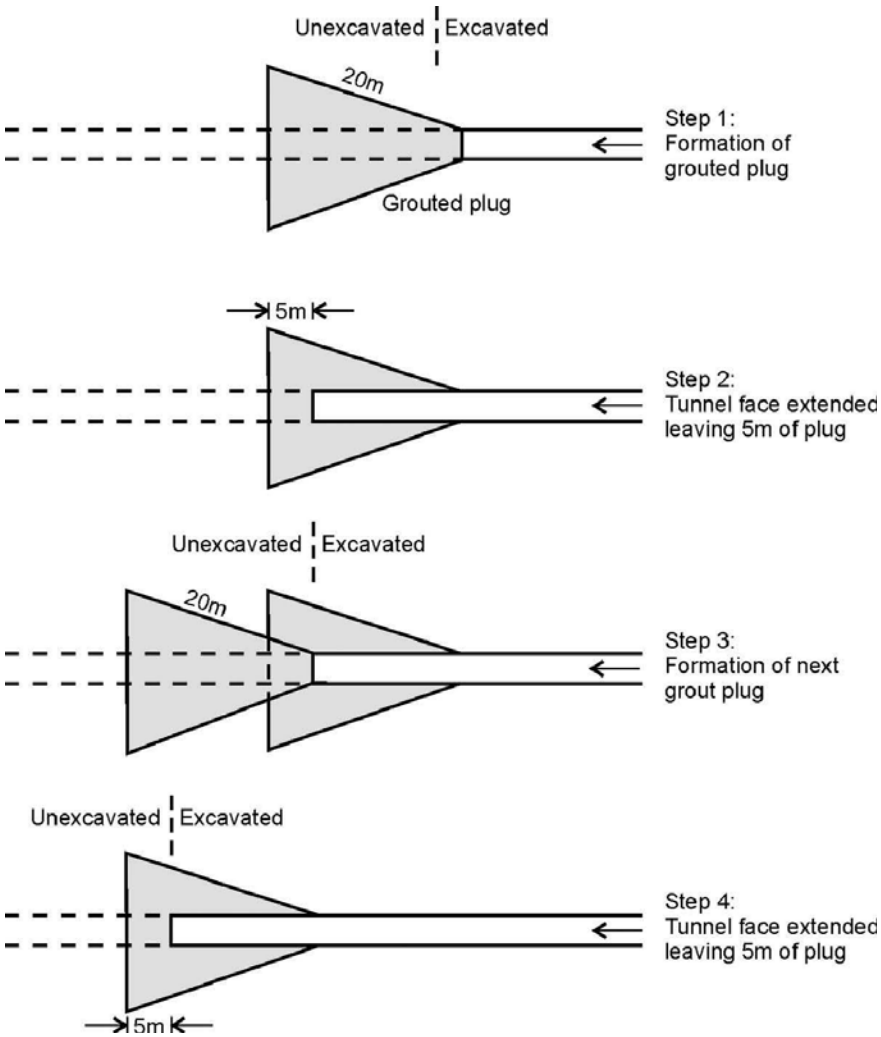

Figure 8 Steps to tackle the problem of sheared contact zone of metabasic and quartzite (Goel *et al.*, 1995b).

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It is envisaged that the rock mass affected by a shear zone is much larger than the shear zone itself. Hence, the affected rock mass must be down-graded to the quality of the shear zone so that a heavier support system than a regular one can be installed. A method has been developed at NGI (Norwegian Geotechnical Institute) for assessing support requirements using the Q-system for rock masses affected by shear zones (Grimstad & Barton, 1993). This has also been used in some Himalayan tunnels in India. In this method, weak zones and the surrounding rock mass are allocated their respective Q-values from which a mean Q-value can be determined, taking into consideration the width of the weak zone/shear zone. Equation 1 may be used in calculating the weighted mean Q-value (Bhasin *et al.*, 1995).

$$\log Q_m = \frac{b.\ \log\ Q_{wz}\ +\ \log Q_{sr}}{b+1} \tag{1}$$

where,

Q_m = mean value of rock mass quality Q for deciding the support,
Q_{wz} = Q value of the weak zone/shear zone,
Q_{sr} = Q value of the surrounding rock, and
b = width of the weak zone in meters.

The strike direction (θ) and thickness of weak zone (b) in relation to the tunnel axis is important for the stability of the tunnel and therefore the following correction factors have been suggested for the value of b in the above Equation 1.

if θ = 90° – 45° to the tunnel axis then use 1b,
if θ = 45° – 20° then use 2b in place of b,
if θ = 10° – 20° then use 3b in place of b, and
if θ < 10° then use 4b in place of b.

Hence, if the surrounding rock mass near a shear zones is downgraded with the use of the above equations, a heavier support should be chosen for the whole area instead of the weak zone alone.

Figure 9 shows a typical treatment method for shear zones (Lang, 1961) in the roof of tunnel. First the shear zone is excavated with caution up to some depth. After excavation, immediately one thin layer of shotcrete with wire mesh or steel fiber reinforced shotcrete (SFRS) shall be sprayed. The weak zone is then reinforced with inclined rock bolts and finally shotcrete with wiremesh or SFRS (preferably SFRS) should be sprayed ensuring its proper thickness in weak zones. This methodology is urgently needed if NATM or NMT (Norwegian Method of Tunneling) is to be used in the tunnels of the Himalayan region, as seams/ shear zones/ faults/ thrusts/ thin intra-thrust zones are frequently found along tunnels and caverns in the Himalaya. *Stitching is perhaps the terminology that best suits this requirement*.

In case of a thick shear zone (b>>2m) with sandy gouge, umbrella grouting or rock bolting is used to enhance the strength of roof and walls in advance of tunneling. The excavation is made manually. Steel ribs are placed closely and shotcreted until the shear zone is crossed. Each (blasting) round of advance should be limited to 0.5m or

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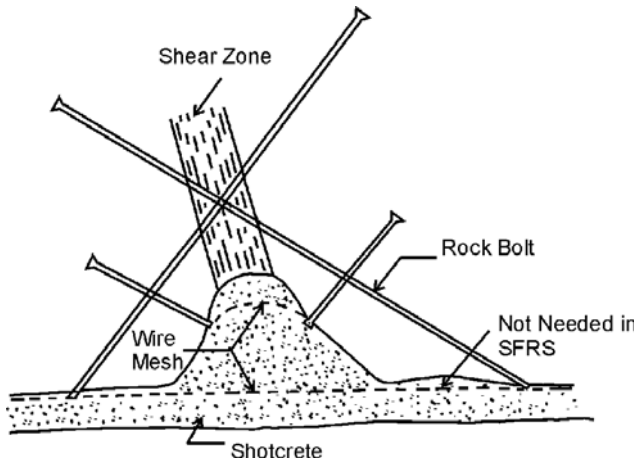

Figure 9 Shear zone treatment in an underground opening (Lang, 1961).

even smaller depending upon the stand-up time of the material and fully supported before starting another round of excavation.

4.7 Water-in-rush

In the tunnels in Himalaya, it has been experienced often that the rock mass above the shear zone is water-charged. This may be because of the presence of impermeable gouge material in the shear zone. Hence, one should be careful and be prepared to tackle the water problem in the tunnels through shear zone having impermeable gouge material.

For example, in the case of Maneri stage-I project head race tunnel in Uttarakhand state, India, because of tunneling, the trapped water in quartzites above impervious shear zone rushed in to the tunnel causing roof collapse and debris flow flooding the tunnel (Fig. 10).

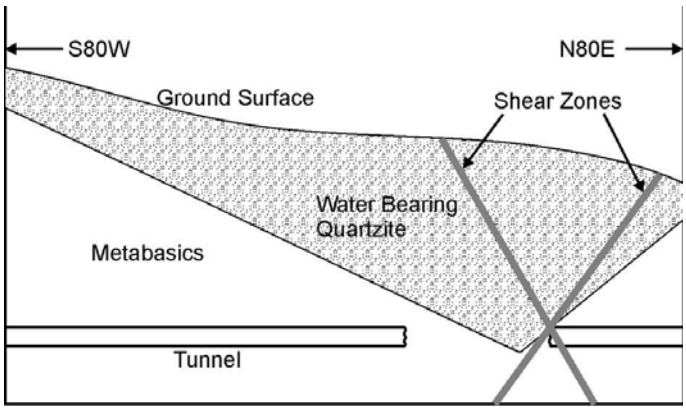

Figure 10 Schematic view of causes of water inrush in head race tunnel of Maneri stage-I project (Goel *et al.*, 1995a).

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Figure 11 Water-in-rush in a rail tunnel in J&K, India.

Intersection of water-charged zones while tunneling is also a common feature. However, when the normal seepage turns into free flowing conditions, particularly with material outwash, tunneling problems attain serious dimensions. If caught unawares, these problems are capable of completely disrupting tunneling activity and influencing the time schedules involved. Such cases call for state of art tunneling techniques like ground stabilization using special grout admixtures or freezing which saves draining arrangements running into several cumec capacity, advance probing, and so on.

In a rail tunnel in J&K state, the water started flowing from the tunnel face with a discharge of more than 500 liters per minute. The tunnel was being constructed through the terrace deposits. It was thought that the discharge rate will subside with time. But, to our surprise, it remained continued with a rate of about 300 liters per minute. The tunnel was completed by pipe-roofing technique and by channelizing the water with the help of drainage holes.

It is highlighted here that once the water is stuck in the tunnel, it is difficult, costly and time consuming to tackle it and tunnel through. *Therefore, it is generally advised to take steps to probe and divert water as soon as first sign of water is seen.*

4.8 Probe holes

An important step to prevent the geological surprise is to drill the probe holes in different directions, if required, from the tunnel face to get the geological information. Having 50m deep probe holes of at least 50 to 75mm diameter ahead of the tunnel face in the regions of highly changing geology could provide valuable advance information of geology. Lesser depth of probe holes can also be drilled depending upon the requirement and availability of drilling equipment and material. The ground conditions, the support pressures, etc., can also be ascertained as per the geological details obtained from probe holes. Accordingly, the construction and supporting techniques may be modified. Such probing is now becoming popular and being used in some projects. *For example*, in Chenani-Nashri highway tunnel project, J&K probe holes were drilled to

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know the water condition ahead of the tunnel face from north end in the escape tunnel. The probe holes have shown that the water pressure is reducing and accordingly the tunnel activity was planned.

Probe holes sometimes can be disastrous also. For example, Parbati stage-II head race tunnel in Nov. 2005 faced a problem when approximately 12000m^3 of silt and fine sand flowing out of a probe hole buried the TBM and half of the bored tunnel with 7000 lit per min water inflow. Hence, it is essential to keep close watch on the variation of geology and water seepage with the excavation before and during drilling the probe holes.

Probing ahead of the tunnel face using geophysical means like tomographic analysis and radar is also becoming popular but is comparatively expensive and cannot be used as a regular or on a routine basis. Seismic profiling is another methodology being adopted to probe the geology ahead of tunnel face.

Probe holes planning, drilling and monitoring shall be carried out in the supervision of experienced geologists to get the desired information and results.

4.9 Tunnel Boring Machine (TBM) in the Himalaya

Use of Tunnel Boring Machine (TBM) has not been very discouraging so far in Himalayan tunneling because of varying geology and water-charged formations. To highlight, three cases of TBM are briefly presented.

The work of 6.75km long Dulhasti project head race tunnel of 8.3m excavated diameter was started with gripper type hard rock TBM. The rock mass was predominantly hard and highly abrasive quartzites. While tunneling, the TBM was inundated with a water inflow of over 1000litres/sec. This 'inrush' occurred at a minor shear zone aquifer (fractured quartzite) within impermeable interbedded phyllites and included 4,000m^3 of sand and quartzite pebbles. The inflows fell to 150 liter/sec within 100 days and five years later inflows of 100 liter/sec were still being recorded. The TBM could bore only 2.86km and finally abandoned. This experience in Himalayan geology was not encouraging. The project has subsequently been completed by conventional excavation. The project was commissioned in 2007, after a delay of about 19 years.

At the head race tunnel for Parbati Stage-II project, H.P. state of India, an incident similar to Dulhasti project tunnel occurred in May 2007 when routine probing ahead of a 6.8m diameter refurbished Jarva open TBM tunnel in sheared and faulted quartzite at 900m overburden cover punctured a water bearing horizon which resulted in inflows of water of over 120 l/sec containing about 40% sand and silt debris. The inflow was sudden and occurred at a high pressure which could not be contained. Eventually over 7500m^3 of sand and silt debris buried the TBM. The project supposed to be commissioned in 2007, was delayed for about 10 years (Sengupta *et al.*, 2008).

National Thermal Power Corporation (NTPC) is constructing the Tapovan-Vishnugad hydroelectric power project (TVHEP) with installed capacity of 520MW (4x130MW). The project has HRT of length approximately 12.1 km, of which 8.6 km has been planned to excavate using a double shield TBM by a Joint Venture (JV) of Larsen & Toubro Ltd., India, and Alpine, Austria. The remaining 3.5km of the HRT is being excavated conventionally. The tunnel passes below the steep hills of Himalaya near Joshimath, India. The tunnel depth at places is more than 1.0km. This is the first

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time the double shield hard rock tunnel-boring machine (ordered from Herrenknecht, Germany) has been used for a hydel power project tunnel in India. The TBM has an excavation diameter of 6.575 m for an internal finished diameter of the HRT being 5.64 m (Saxena, 2013).

Soon after TBM excavation started, it became clear that there are also groundwater-bearing, approximately NS striking, steeply inclined faults and fracture zones associated with quartz-rich lithologies such as quartzite, quartzitic gneiss and augen gneiss. These steeply inclined fracture and fault structures cut across the main foliation joint, which means that there is a high level of interconnection between the joint systems, allowing for the development of potent and high-pressure aquifers. The rock types are gneisses and quartzites (Brandl *et al.*, 2010).

During the excavation, the TBM encountered a large fault zone. A major portion of rock detached and dented the shield of the TBM and the TBM trapped. Subsequently, there was heavy ingress of ground water into the tunnel, commencing at the tail skin area of the TBM and progressing rapidly, through the ungrouted section of the annulus, some 160 m back along the tunnel. The water pressure was very high carrying the rock material and debris which resulted in more damages to TBM (Brandl *et al.*, 2010). Work remains standstill for quite some time. Subsequently, a bypass tunnel was excavated to recover the buried TBM. The TBM has been recovered, repaired and again put to use in the same tunnel.

While the excavation using TBM has been quite successful in other parts of India, *e.g.*, Delhi metro, Srisailam left bank canal tunnel and Bombay Malabar hill tunnel, the Himalaya remain a major challenge. The experience suggests that many of the problems can be avoided if sufficient advance information ahead of the face is available. Following key issues have been identified by Goel (2014) for the success of TBM in the Himalaya.

4.10 Key issues for TBM success in the Himalaya

The Himalaya pose the most challenging ground conditions for tunneling. One of the prime reasons is that they are the youngest of the mountain chains and are still tectonically active. The difficulties of tunneling at depth through high mountainous terrain pose major challenges not just for tunnel boring machines (TBM) but also for the use of drill and blast (D&B).

The big investment in a sophisticated TBM and the expectation of mostly rapid advance rates can be spoiled by the unexpected delays caused by unexpected ground. Only a few percent of the total length of a tunnel may be unexpected, yet these few percent could double the construction time in some cases.

Tunneling in adverse ground is significantly less tolerant of the limitations of the tunneling approach than in good ground. Generally, the more difficult the ground, the more flexibility is also needed. Tunneling in the Himalaya, the Andes and until recently the Alps has shied away from TBM use due to perceived inflexibility and the likelihood of the machines getting trapped by adverse ground conditions, either as a result of squeezing or spalling/bursting conditions or because of ground collapses associated with rock falls or with running or flowing ground within faults. Any of these situations can lead to problematic tunneling at best and collapses and abandonment at worst.

Following are the key issues for the success of TBM in the Himalaya.

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4.10.1 Geological investigations and probe holes

The more challenging the ground, the greater the pre-planning that is required before tunneling. This challenge is not just of tackling adverse ground, stress state and/or groundwater conditions, it is also often about logistics. For deep tunnels in mountainous regions, problematic geologic zones often are at significant distance from the nearest portal and at such significant depth that surface pre-treatment is generally impractical (Carter, 2011).

Experience suggests that many of the problems associated with the TBM in the Himalaya can be avoided if sufficient geological and geohydrological information is available in advance. Faced with cost and time constraints, detailed investigations before selecting a tunnel alignment are often compromised, resulting in encountering very disturbed geological conditions. It is essential that detailed exploration work is carried out before the start of the project and exploration ahead of the face is undertaken on a continuous basis.

In particular for a TBM driven tunnels – which are not as flexible as a conventionally driven tunnels – forward probe drilling from the tunnel face is certainly not an alternative to an adequate pre-investigation. But, regular cautious probe drilling during cutter change and maintenance shifts could largely remove the unexpected; especially if performed with two slightly diverging probe holes (Barton, 2000). It must be highlighted here that while drilling the probe holes, to avoid the blow outs, the groundwater conditions shall be closely watched and an attempt shall be made to carry out geophysical investigations. It is needless to mention here that the probe holes shall be drilled under close supervision of an experienced engineering geologist.

4.10.2 Selection of TBM and add-on-features

Selection of a TBM is the key decision. Complications in the decision-making process, in general, relate to the timing when making this decision, as it needs to be made 12–18 months in advance of actually starting tunneling, so that sufficient lead time is available for building the machine. However, often detailed project site investigations are incomplete, still ongoing or sometimes not even started when this key decision is to be made. Furthermore, once the contract is awarded to the contractor, generally after a long tendering process, almost always insufficient time and/or funds have been allocated to allow the contractor any opportunity for additional customized exploration to support his own excavation technology selection procedures before initiating equipment procurement.

The choice of TBM also needs critical analysis at the planning stage. In the absence of accepted standards for the design and construction of a type of TBM and the fact that no TBM can be designed for every type of geological condition, the design and special construction characteristics of each TBM need to be carefully, project-specifically designed. The shielded TBM has a definite edge over the open TBM as it is not as sensitive to the instability phenomena of the excavation walls owing to the presence of precast concrete or steel lining inside and the protection of the shield (Saxena, 2013).

TBM can be designed with add-on-features as per the site conditions, *e.g.* probe hole drilling, forepoling, shotcrete spraying, rock bolting, pre-grouting/grouting,

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steel rib erection etc. As per the expected requirements, these features can be incorporated in the TBM.

In mountainous terrain, when considering a decision on whether or not to use a TBM, and which type of TBM to use for a deep tunnel, it must be appreciated that, historically, three types of ground conditions have proved to be the most problematic from the viewpoint of halting tunnel advance. In order of severity, case records suggest faults with gouge filling, heavy water and major stress, individually and/or in combination, constitute the most problematic ground conditions. The three elements which control the trouble-free tunnel excavations at significant depth are, stress state, groundwater conditions and the rock or the medium. Adverse characteristics of any of these three elements can, on its own, compromise drill and blast (D&B) or TBM tunneling, but it usually takes a combination of all three being adverse to trap a machine or halt a D&B drive to the extent that a bypass becomes necessary (Carter, 2011).

Hence, TBM shall be selected after detailed analysis of stresses, ground water and the expected rock masses and ground conditions.

4.10.3 Expert TBM crew

Even with the best possible TBM, the progress required may not be achieved. Experienced and dedicated TBM crew is very important for the success of TBM. It is the expert crew, which can take the right decision at the right time and implement it properly. Success of TBM in Kishanganga hydroelectric project in India is one such example.

Bieniawski (2007) also highlighted the influence of TBM crew and suggested an adjustment factor for the influence of TBM crew on its performance in rock mass excavatability index for TBM.

4.10.4 Timely decision and action

Extreme ground conditions present major contrasts to tunneling, so much so that they inevitably demand use of flexible rock engineering solutions for the tunnel to progress. The fact that conditions within the Himalaya can be expected to be as bad as has ever been encountered elsewhere means there has to be the ability while tunneling to allow changes in excavation procedures and in pre- and post-excavation support approaches. It has been experienced that the delays in decision have enhanced the problems. For tunnel to be completed successfully, *the rock is not going to wait*. Hence, timely decision and action is important. There may be situations where the flexibility in the designs is required. This is possible only when the engineers-in-charge are given decision and risk taking authority.

4.10.5 Risk sharing

"Engineers have to take a calculated risk, persons become wiser after an accident. If they were really wise, it was their duty to point out mistakes in the design to engineers" – Karl Terzaghi

In more difficult ground conditions, such as those encountered in the Himalaya, with minimal investigation comes more risk of the TBM getting trapped – either as a result of

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squeezing or spalling/bursting conditions or because of ground collapses associated with rockfalls or with running or flowing ground within faults. These cases are further complicated by heavy water inflows. To reduce these risks considerably more investment must be made in the design process in these complex mountainous regions. Significant reduction of real risk can only be achieved through more investigative effort, not through design refinement. Cost and schedule analysis of past case records suggests that for complex ground conditions, *some 5% of the engineer's estimate of capital expenditure is required to be spent on investigating ground conditions to push the process in the right direction* (Carter, 2011).

Hence, if the investigations are insufficient, whatever is the reason, various problems are bound to be encountered as a surprise (mostly not at the expected location). There should be provision of risk sharing between the client and the contractor in the contract document. Otherwise these surprises result in the time and cost over-runs and litigations.

As mentioned earlier the contract document shall have the flexibility also to accommodate the unforeseen conditions/events. The site engineers shall be given the responsibility of allowing and approving the use of newer techniques and material required for tackling the unforeseen conditions. The contractor, once allowed by the site engineer, shall get full payment after executing the job to the satisfaction of the site engineers.

The contract should include (i) clause for compensation to contractor for an unexpected geological conditions or surprises, (ii) clause on innovations by contractors and engineers on the basis of mutual agreements, (iii) clauses for first and second contingency plans for the preparedness and (iv) penalty for delays in construction. Obviously contract is not a license for injustice to any party. Injustice done should be corrected soon (Singh & Goel, 2006).

4.11 General observations

Following are the general observations from various tunneling and underground projects constructed so far.

(i) The alignments of long power tunnels have not been fixed after proper and purposeful geological exploration. The surface geological features have proved misleading. Consequently, many disastrous problems were faced in tunneling.

(ii) In squeezing grounds, the selection of size of power tunnel is important. Initially one big tunnel was excavated, as it became difficult to drive, three tunnels of smaller diameter were then driven (Mitra, 1991). This was because there were fewer supports, bridge action period was greater and heaving of the floor was limited.

(iii) The underground power house was located at one side of the river and not too high above the water level in the river. Consequently during flood, water entered into the cavern through the tail race tunnel while its excavation was going on. The rock masses in the roof and walls were submerged for a couple of weeks before water could be pumped out. Fortunately the cavern remained stable.

(iv) The seepage through dam foundation is increased after a major earthquake, as permeability of the jointed rock mass increases drastically during shearing. On the contrary permeability of micro-fissured rock is reduced due to

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deformation of the joints beneath the foundation due to thrust on the reservoir filling, thereby making the grout curtain redundant which led to a dam failure.

(v) The study (on the basis of 11 years of monitoring of Chhibro underground cavern of Yamuna hydro-electric project) has shown that ultimate roof support pressure for water-charged rock masses with erodible joint filling may rise up to 6 times the short-term support pressure. The damage to the support system during an earthquake of 6.3 magnitude is not appreciable except near faults with plastic gouge material (Mitra, 1991).

(vi) Very high support pressures may be generated by reduction in the modulus of deformation due to saturation of the rock mass around HRT, TRT and penstocks etc. (Verman, 1993). Mehrotra (1992) observed that the modulus of deformation is actually very low after saturation compared with that in dry conditions in the case of argillaceous rocks (claystone, siltstone, shale, phyllite etc.).

5 CONTRIBUTIONS IN THE STATE-OF-THE-ART

5.1 Squeezing ground condition

The over-stressed zone of rock mass fails where tangential stress (σ_θ) exceeds the mobilized uniaxial compressive strength (UCS) of the rock mass. The failure process will then travel gradually from the tunnel boundary to deeper regions inside the unsupported rock mass. The zone of the failed rock mass is called the 'broken zone'. This failed rock mass dilates on account of the new fractures. A support system after installation restrains the tunnel deformation and gets loaded by the support pressure.

It is evident from the ground reaction curve that the support pressure decreases rapidly with increasing tunnel deformation. *Hence, significant tunnel deformation shall be allowed to reduce the cost of support system. This is the secret of success in tunneling through squeezing ground condition.*

In case one chooses to install very stiff support system, it may be seen from Figure 12 that stiff support system will attract high support pressure as it will restrict the tunnel deformation. If a flexible support system is built after some delay, it will attract much less support pressure. This is ideal choice. However, too late and too flexible support system may attract high support pressure due to excessive loosening of rock mass in the broken zone. Yet the squeezing ground comes to equilibrium after years even in severe squeezing ground condition. Although the final deformations may be undesirable and so corrective measures are required.

The data suggests that support pressure jumps up after tunnel deformation of about 5 to 6 percent of tunnel size. Then there is sympathetic failure of entire brittle rock mass within the broken zone, rendering its residual cohesion $c_r = 0$ in the highly squeezing ground. The theoretical ground response curve is shown in Figure 12 on the basis of this hypothesis. The sympathetic failure is in fact unstable and wide spread fracture propagation in the entire failure zone, starting from the point of maximum shear strain. This brittle fracture process may be taken into account in the elasto-plastic theory by assuming $c_r = 0$ after critical tunnel deformation of 6%. *Thus, it is recommended that tunnel deformation shall not be permitted beyond 4 percent of tunnel radius to be on safe side.*

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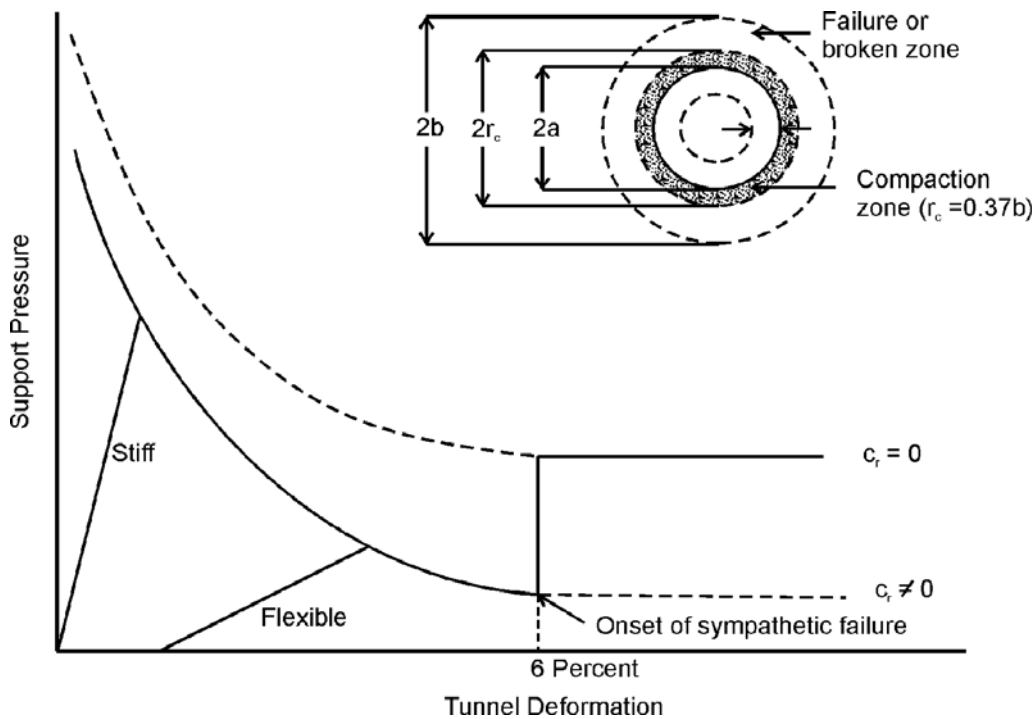

Figure 12 Effect of sympathetic failure of rock mass on theoretical ground response curve of squeezing ground condition. Support reaction curve of stiff and delayed flexible supports are superposed.

10.5.1.1 Compaction zone within broken zone

From the study of steel rib supported tunnels, Jethwa (1981) observed that the values of coefficient of volumetric expansion (K) are negative near the tunnel and increased with radius vector. Thus, he postulated the existence of compaction zone within the broken zone (Fig. 13). The radius of the compaction zone (r_c) is estimated to be approximately equal to,

$$r_c = 0.37\ b \tag{2}$$

Thus, compaction zone will not develop where b is equal to a/0.37 or 2.7a. This is the reason why compaction zone was not observed in some of the European tunnels in the squeezing ground conditions as b was perhaps less than 2.7a.

In an ideal elasto-plastic rock mass, compaction zone should not be formed. The formation of the compaction zone may be explained as follows. A fragile rock mass around the tunnel opening fails and dilates under the influence of the induced stresses. The dilated rock mass then gets compacted due to the passive pressure exerted by the support system in order to satisfy the ultimate boundary condition that is zero rate of support deformation with time. The development of support pressure with time would reduce the deviator stresses ($\sigma_\theta - \sigma_r$) within the compaction zone which in turn will undergo creep relaxation manifested by the negative K values.

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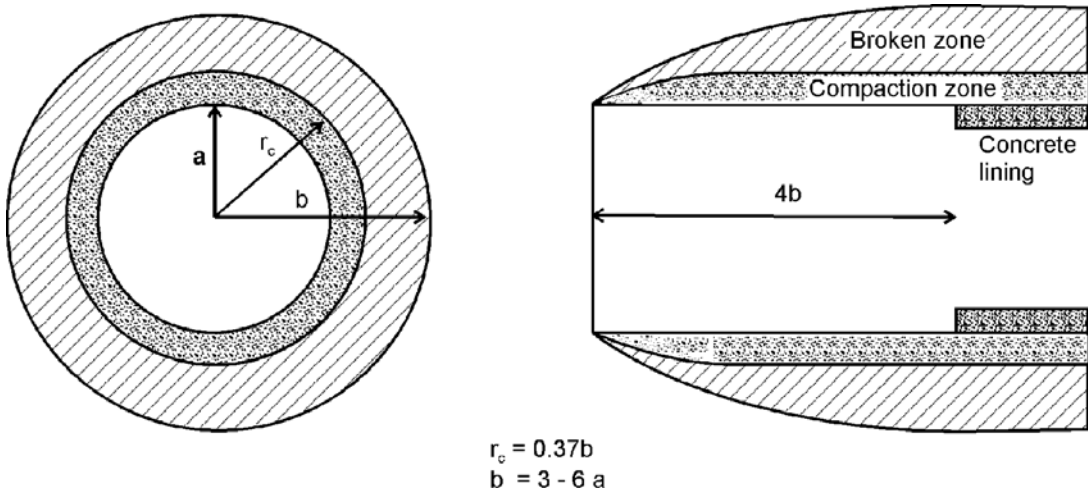

Figure 13 Compaction zone within broken zone in the squeezing ground condition (Jethwa, 1981).

Table 6 Overall coefficient of volumetric expansion of failed rock mass (K) within broken zone (Jethwa, 1981; Goel, 1994).

S.No.	*Rock Type*	*K*
1.	Phyllites	0.003
2.	Claystones / Siltstones	0.01
3.	Black clays	0.01
4.	Crushed sandstones	0.004
5.	Crushed shales	0.005
6.	Metabasics (Goel, 1994)	0.006

5.1.2 Coefficient of volumetric expansion

Jethwa (1981) has found the overall values of K from instrumented tunnels in Himalaya. These values are listed in Table 6. Actually K varied with time and radius vector. K was more in roof than in walls. So only peak overall values of K are reported and considered to have stabilized to a great extent with time (15 – 30 months). It is heartening to note that the value of K for crushed shale is the same at two different projects. It may be noted that higher degree of squeezing was associated with rock masses of higher K values.

5.2 Prediction of ground conditions

5.2.1 Empirical approach for predicting the ground conditions

To avoid the uncertainty in obtaining appropriate SRF ratings in rock mass quality Q of Barton *et al.* (1974), Goel *et al.* (1995c) have suggested rock mass number N, defined as follows, for proposing the criterion of estimating ground conditions for tunneling.

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$$N = [Q]_{SRF} = 1 \tag{3}$$

Equation 3 suggests that N is Q with SRF = 1.

Other parameters considered are the tunnel depth H in meters to account for stress condition or SRF indirectly, and tunnel width B in meters to take care of the strength reduction of the rock mass with size. The values of three parameters – the rock mass number N, the tunnel depth H and the tunnel diameter or width B were collected covering a wide variety of ground conditions varying from highly jointed and fractured rock masses to massive rock masses.

All the data points were plotted on a log-log graph (Figure 14) between rock mass number N and $H.B^{0.1}$. Figure 14 shows zones of tunneling conditions/hazards depending upon the values of $HB^{0.1}$ and N. Here H is the overburden in meters, B is the width of the tunnel in meters and N is rock mass number.

The equations of various demarcating lines have been obtained as shown in Table 7. Using these equations, one can predict the ground condition and then plan the tunneling measures. These equations can also be used to plan the layout of the tunnel to avoid the squeezing ground condition.

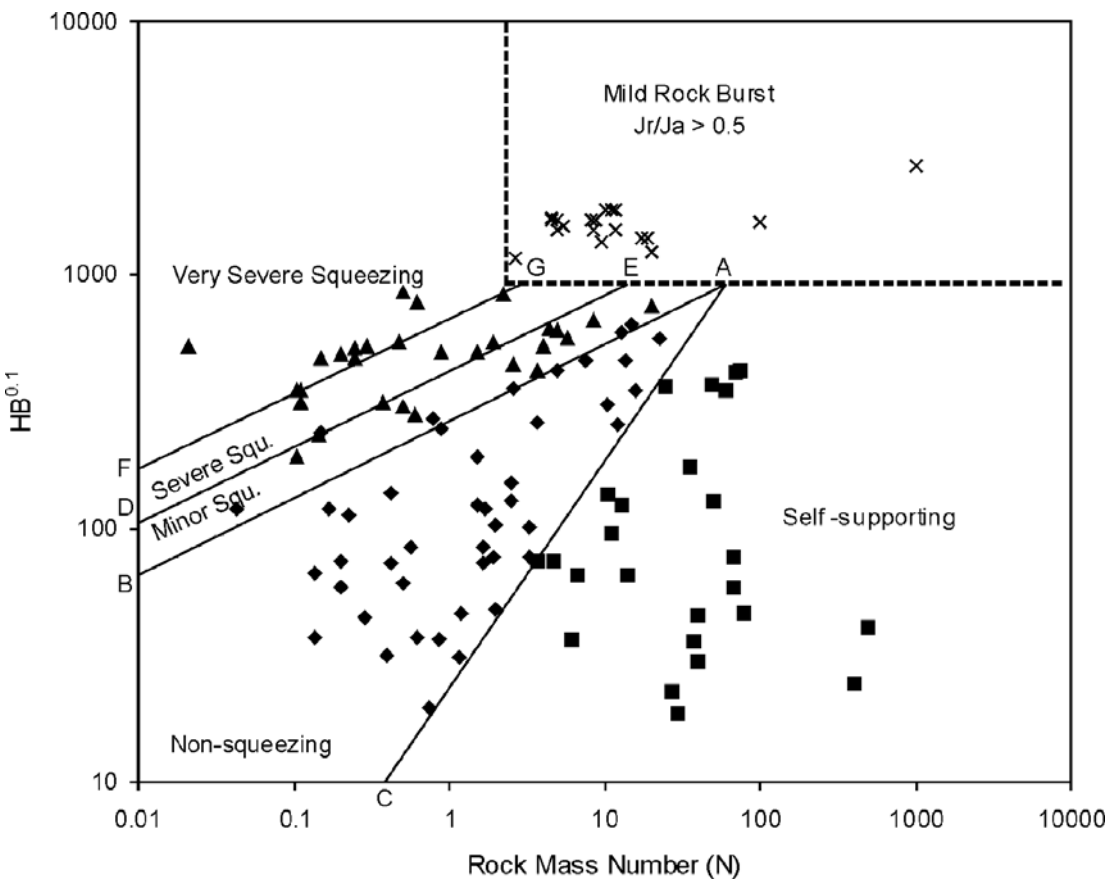

Figure 14 Plot between rock mass number N and $HB^{0.1}$ for predicting ground conditions (Singh & Goel, 2011).

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Table 7 Prediction of ground condition using N (Singh & Goel, 2011).

S. No.	*Ground Conditions*	*Correlations for Predicting Ground Condition*
1.	Self-supporting	$H < 23.4\ N^{0.88}.\ B^{-0.1}$ & $1000\ B^{-0.1}$ and $B < 2\ Q^{0.4}$ m (for ESR=1) (Barton *et al.*, 1974)
2.	Non-squeezing (< 1% deformation)	$23.4\ N^{0.88}.B^{-0.1} < H < 275N^{0.33}.\ B^{-0.1}$
3.	Mild or minor squeezing (1–3% deformation)	$275\ N^{0.33}.\ B^{-0.1} < H < 450N^{0.33}.\ B^{-0.1}$ and $J_r / J_a < 0.5$
4.	Moderate or severe squeezing (3–5% deformation)	$450\ N^{0.33}.\ B^{-0.1} < H < 630N^{0.33}.\ B^{-0.1}$ and $J_r / J_a < 0.5$
5.	High or very severe squeezing (>5% deformation)	$H > 630N^{0.33}.\ B^{-0.1}$ and $J_r / J_a < 0.25$
6.	Mild rock burst	$H.B^{0.1} > 1000$m and $J_r/J_a > 0.5$ and $N > 2$

Notations: N = rock mass number (Barton's Q with SRF = 1); B = tunnel span or diameter in meters; H = tunnel depth in meters; J_r and J_a = Parameters of Q-system; and ESR= Barton's excavation support ratio.

Few cases of spalling and rock burst conditions have also been plotted in Figure 14. These data points are lying in the upper right hand side corner of Figure 14. The enclosure is shown by dotted line because more data points are needed before reaching to a firm conclusion. However, it may be highlighted here that spalling and rock burst have been encountered in the rock masses having $J_r/J_a > 0.5$, N>2 and $H.B^{0.1}>1000$. Hence, the rock mass having J_r and J_a ratings in category b and c in the Tables of J_r and J_a in Q-system shall not experience the rock burst condition.

Equations given in Table 7 are found to be quite useful in planning and design. In addition, the N based approach of estimating/predicting the ground condition has been found as complimentary to Q-system in respect of identifying the ground conditions and then select SRF rating to get Q-value. The, Q-value thus obtained has been used for the support design in tunnels using the Grimstad *et al.* (2003) chart.

5.2.2 SRF rating in Q-system for spalling and rock burst conditions

Kumar *et al.* (2006) have studied the spalling and rock burst condition in the Nathpa-Jhakri Project tunnels. They have experienced that the Barton's stress reduction factor (SRF) ratings in case of 'competent rock' should be different for massive rocks and for moderately jointed rocks. Accordingly they have proposed new rating of SRF for competent moderately jointed rocks as shown in Table 8. The support pressure values obtained using these new suggested SRF ratings are found to be matching with the observed values (Kumar *et al.*, 2006).

High value of J_r/J_a leads to high angle of internal friction along joints. Consequently, the intermediate principal stress (σ_2) which is the in situ stress along tunnel axis increases the rock mass strength enormously where overburden is more than 1000m. The net effect is that the rock burst condition is not serious as anticipated from Barton *et al.* (1974) and Grimstad & Barton (1993). In other words the pre-stressing effect of σ_2 on rock mass strength is similar to reduction in SRF values. Application of the polyaxial failure criterion shall be studied and Table 8 needs further research.

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Table 8 Stress reduction factor (SRF) rating for jointed rock for categories L, M and N (Kumar *et al.*, 2006).

Category	*Rock Stress Problem*		q_c/σ_1	σ_θ/q_c	*SRF (Old)*	*SRF (New)*
H	Low stress, near surface open joints		>200	<0.01	2.5	2.5
J	Medium stress, favorable stress condition		200–10	0.01–0.3	0.5–2.0	1.0
K	High stress, very tight structure (usually favorable to stability, may be unfavorable to wall stability)		10–5	0.3–0.4	0.5–2.0	0.2–2.0
L	Moderately slabbing after > 1hr	Massive rock	5–3	0.5–0.65	5–9	5–50
		Moderately jointed rock			–	1.5–2.0
M	Slabbing and rock burst after a few minutes	Massive rock	3 – 2	0.65–1.0	9–15	50–200
		Moderately jointed rock			–	2.0–2.5
N	Heavy rock burst (strain burst) and immediate deformations	Massive rock	< 2	< 1	15–20	200–400
		Moderately jointed rock			–	2.5–3.0

5.3 Deformation of tunnel walls

It is common knowledge that the rock failure is associated with volumetric expansion due to creation and progressive widening of new fractures. Consequently all the points within the broken zone in a circular tunnel shift almost radially toward the opening because the expanding rock mass is kinematically free to move in radial direction only. Labasse (1949) assumed that the volume of failed rock mass increases at a constant rate (called coefficient of volumetric expansion, K).

The displacement at the boundary of broken zone (u_b) is negligible compared to that at the opening periphery (u_a). So the coefficient of volumetric expansion is defined as follows:

$$K = \frac{\pi\, b^2 - \pi\,(a - u_a)^2 - \pi\,(b^2 - a^2)}{\pi\, b^2 - \pi\, a^2} \tag{4}$$

Equation 4 can be solved as below for obtaining u_a

$$u_a = a - \sqrt{a^2 - K(b^2 - a^2)} \tag{5}$$

Using Equation 5 and the K-value from Table 6 it has become possible to estimate the tunnel wall/roof deformation.

5.4 Estimating support pressure in squeezing ground condition

First time in 1994 it has been empirically established that in the tunnels with arch roof through squeezing ground, the support pressure in steel rib supported tunnels varies with the tunnel size as per the following Equation 6 (Goel, 1994).

$$p_v(sq) = \left[\frac{f(N)}{30}\right] \cdot\ 10^{\left[\frac{H^{0.6} \cdot\ a^{0.1}}{50 \cdot\ N^{0.33}}\right]}, \quad \text{MPa} \tag{6}$$

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Table 9 Correction factor f(N) for tunnel deformation in Equation 6 (Goel, 1994).

S.No.	*Degree of Squeezing*	*Normalized Tunnel Deformation %*	*f (N)*
1.	Very mild squeezing ($270\ N^{0.33}.\ B^{-0.1} < H < 360\ N^{0.33}.\ B^{-0.1}$)	1 – 2	1.5
2.	Mild squeezing ($360\ N^{0.33}.\ B^{-0.1} < H < 450\ N^{0.33}.\ B^{-0.1}$)	2 – 3	1.2
3.	Mild to moderate squeezing ($450\ N^{0.33}.\ B^{-0.1} < H < 540\ N^{0.33}.\ B^{-0.1}$)	3 – 4	1.0
4.	Moderate squeezing ($540\ N^{0.33}.\ B^{-0.1} < H < 630\ N^{0.33}.\ B^{-0.1}$)	4 – 5	0.8
5.	High squeezing ($630\ N^{0.33}.\ B^{-0.1} < H < 800\ N^{0.33}.\ B^{-0.1}$)	5 – 7	1.1
6.	Very high squeezing ($800\ N^{0.33}.\ B^{-0.1} < H$)	>7	1.7

Notations: N = rock mass number; H = tunnel depth in meters; and B = tunnel width in metres.
Note: Tunnel deformation depends significantly on the method of excavation. In highly squeezing ground condition, heading and benching method of excavation may lead to tunnel deformation > 8%.

where,

$p_v(sq)$ = short-term vertical roof support pressure in squeezing ground condition in MPa,
f(N) = correction factor for tunnel deformation obtained from Table 9, and
H & a = tunnel depth & tunnel radius in meters respectively (H < 600m).

Equation 6 has been developed using the measured value of support pressures from number of tunnels experiencing the squeezing ground conditions in the Himalaya.

Subsequently, Bhasin & Grimstad (1996) have proposed Equation 7 to estimate the support pressure in tunnels through poorer qualities of rock masses. Equation 7 also shows the effect of tunnel size on ultimate roof support pressure (p_{roof}).

$$p_{roof} = \frac{40\ D}{J_r.Q^{1/3}}\ ,\ \text{kPa} \tag{7}$$

where D is the diameter or span of the tunnel in meter.

It may be highlighted here that Equation 6 has been successfully used for estimating the support pressure in some tunnels including the Udhampur-Katra rail link project tunnels in the Himalaya.

5.5 Effect of tunnel size on support pressure

Effect of tunnel size on support pressure is summarized in Table 10.

It is cautioned that the support pressure is likely to increase significantly with the tunnel size for tunnel sections excavated through the following situations:

(i) slickensided zone,
(ii) thick fault gouge,
(iii) weak clay and shales,
(iv) soft plastic clays,

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Table 10 Effect of Tunnel Size on Support Pressure (Goel *et al.*, 1996).

S. No.	*Type of Rock Mass*	*Increase in Support Pressure Due to Increase in Tunnel Span or Dia. from 3m to 12m*
A. Tunnels with Arched Roof		
1.	Non-squeezing ground conditions	Up to 20 percent only
2.	Poor rock masses / squeezing ground conditions (N = 0.5 to 10)	20 – 60 percent
3.	Soft-plastic clays, running ground, flowing ground, clay-filled moist fault gouges, slickensided shear zones (N = 0.1 to 0.5)	100 percent
B. Tunnels with Flat Roof (irrespective of ground conditions)		up to 100 percent

(v) crushed brecciated and sheared rock masses,
(vi) clay filled joints, and
(vii) extremely delayed support in poor rock masses.

5.6 Principles of planning

The principles of planning tunnels and underground structures are summarized as follows:

(i) The electrical power is generated in proportion to product of the river water discharge and the hydraulic head between reservoir water level and the turbines.

(ii) So the size of machine hall depends upon the generation of electrical power and number of turbines. The machine hall should be deeper in the hill for protection from landslides. The power house may be located on a stable slope in the case of hard rocks, if it is not prone to landslides and snow avalanches. The pillar width between adjacent caverns should be at least equal to the height of any cavern or the average of widths (B) of caverns whichever is larger.

(iii) The size of the desilting chamber or settling basin (rock cover H > B) depends upon the size of particles which we need to extract from the sediment from intake. Some rivers like Ganga in Himalaya have very high percentage of feldspar and quartz particles (of hardness of 7) which damage the blades of the turbines (of weldable stainless steel) soon. In such cases desilting basins may be longer. The (pillar) spacing of desilting chambers is of critical importance. Turbines are closed when silt content is more than the design limit in the river during the heavy rains.

(iv) The rock cover should be more than three times the size of openings (power tunnels). The overburden pressure should counter-balance water head in head race tunnel and penstocks etc. However, the rock cover should be less than 350 $Q^{1/3}$ m for $J_r/J_a < 0.5$ to avoid squeezing conditions. Further the overburden should also be less than 1000m for $J_r/J_a > 0.5$ to avoid rock burst conditions.

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(v) Designers have to select velocity of flow of water (say 4–6 m/sec) in head race tunnel. Higher velocity means more head loss and more abrasion of concrete lining. Less velocity, on the other hand, leads to bigger diameter of head race tunnel which increase cost and the time of construction. Thus an economical diameter of the head race tunnel is found out by trial and error approach. There is no need of hoop reinforcement in spite of high hoop tensile stresses in lining within the strong rocks as cracks (< 3mm) are self-healing. The center to center spacing of HRTs and TRTs (of width B) should be 2B in (competent) non-squeezing grounds, and preferably 6B in squeezing ground conditions for stability of the rock pillars. The width of the tunnels should preferably be limited to 6m in severe squeezing conditions, as support pressure increases with width (for Q < 4). Unlined HRTs and TRTs may be planned in unerodible and hard rock masses ($B < 2\ Q^{0.4}$ m and $H < 275\ N^{0.33}\ B^{-0.1}$ m, N = Q with SRF = 1, B is the tunnel size in meters), where it is more economical than the tunnels lined with PCC.

The PCC lining and the steel liner in penstocks should be designed considering actual rock sharing pressure and water hammer along with maximum head of water. The thickness of steel liner should be adequate to withstand external seepage water head when penstock is empty. The steel liner should have stiffeners to prevent its buckling when empty.

(vi) Alignment of underground structures is planned according to geology and rock mass quality. If there is one band of sound strata, we try to align the underground structures along this sound strata (with generally Q>1, E_d >2GPa, except in shear zones; but $H<350Q^{1/3}$ m). The penstocks may be turned along sound rock strata and need not be along the head race tunnel. The general alignment should preferably be perpendicular to shear zones/faults/thrusts zones. The thickness of intra thrust zone should be minimized if any. The caverns should not be parallel to strike of continuous joints but be parallel to major horizontal in situ stress if possible. The displacement of the walls of machine hall should not be high at haunches (say > 6 or 7 cm) so that gantry crane can travel freely for maintenance of machines. We should also try to avoid swelling ground which may contain clay like montmorillonite etc.

(vii) In fact designers cannot plan in the beginning as a few drifts or drill holes give no idea of complex geology in tectonically active regions. We know complete geology about a project only when project is over. The important thing is to have proper documentation of each project.

(viii) A good reservoir site is located where width of the reservoir expands upstream and preferably downstream of confluence of rivers so that volume of river water storage is maximum. Steep gorges such as in Himalaya need high dams for reasonable reservoir storage (~10,000 MCM). High dam stores water in rainy season. High Tehri dam absorbed a huge flood of 3500cumecs and saved devastation of down-stream during hydrological disaster in 2013 in Uttarakhand state. Engineers are planning run-of-the-river schemes along rivers with steep bed slopes but they do not store more water for power generation but are good ecologically (in China, Himalaya). The life span and capacity of a reservoir increases with the height of a dam and so plays an important role in planning of river valley projects (Swamee, 2001).

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(ix) The whole plan should be feasible geologically and economical. Several layouts are made and costs compared. In 21st century time of completion of project is very important as time means money (monthly profit). Cost of construction time should be added to total cost of the project for optimization of the overall cost. So engineering geology and rock mechanics studies are very important to reduce the time of construction of underground structures, especially the very long tunnels (> 5 km) under high overburden in weak rocks. In three gorges dam project (17,000 MW) in China, lot of money and construction time was saved because of properly planned rock/geotechnical investigations.

(x) Topography or slope of the hill river bed matters. The longer head race tunnel means more hydraulic head which gives more power.

(xi) Underground structures are preferred in the highly seismic area or landslide prone area or in strategic area as they are more stable, durable and invisible. Surface structures are prone to damage by landslides (such as in Himalaya and Alps).

(xii) Major tectonic features like tectonically active thrust and fault zones (*e.g.* MCT, MBT etc. in Himalaya) should be avoided, otherwise articulated or flexible concrete lining may be provided to deform without failure during slip along these active thrust zones in the entire life span of the project. It is interesting to know that the observed peak ground acceleration increases with earthquake magnitude up to 7M (on Richter's scale) and saturates at about 0.70 g up to 8 M (Krishna, 1992).

(xiii) Major shear zone should be treated by the dental concrete upto adequate depth beneath a concrete dam foundation. Abutment should be stabilized by grouted rock bolts or cable anchors if wedge is likely to slide down. In fact roller compacted concrete (RCC) dam may be more economical than conventional gravity concrete dam on weak rocks. Doubly arched dams will be economical in hard rocks and stable strong abutments. Stepped spillways are preferred now-a-days for reducing hydraulic energy of rapidly flowing water. The double rows of grout holes should be used to provide a tight grout curtain in a highly jointed rock mass below a concrete dam near heel.

(xiv) Dynamic settlement of earth or earth and rockfill dams should be less than 1m or one hundredth of dam height whichever is less. The longitudinal tensile strains (stress) should be less than permissible tensile strain (stress) in clay core. It is a tribute to the engineers that no failure of modern high earth and rockfill dams has taken place even during major earthquake (~8M) since 1975 all over the world, even when earth has passed through unprecedented peak of earthquakes. Filter upstream of clay core is also provided to drain seepage water during rapid draw down in many modern earth and rockfill dams.

(xv) The site should not be prone to formation of huge landslide dams (in unstable steep gorges subjected to large debris flow during long rains), causing the severe flash floods upon breaking. Example is temporary huge twin-reservoir created by landslide of Pute hydro-electric station, Ecuador. Nor there should be deep seated landslides near the dam site generating waves which would over-topple the dam as in Vajont dam, Italy. Further stability of villages above reservoir rim is to be looked into for safety of the people.

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(xvi) Approach tunnels, hill roads and bridges should be planned and maintained well in landslide prone mountains (subjected to heavy rains or cloud bursts (> 500 mm per day) or rains for a long time) and snow avalanche prone areas for easy construction of the project. Hill roads and vibrations due to blasting in tunnels should not damage houses upon the ground, if any.

(xvii) Huge contingency funds should be made available upto 30 % of total cost for timely risk management in underground structures. Authorities should prepare risk management plan 1 and 2, in case of unforeseen geological hazards. Alternate huge diesel generators should be installed for emergency power supply in openings.

(xviii) The extensive geodynamic monitoring of displacements and seepage in to landslides along the reservoir rim and all caverns and reservoir induced seismicity etc. is undertaken along the entire hydroelectric project, and connected to the internet for global viewing, using space imageries also. The caverns are fitted with warning systems (connected to extensometers) for safety of the persons. Instrumentation is the key to success in tunneling in weak rock masses.

(xix) Portal of tunnels should be located in the stable rock slopes which should not be prone to severe landslides or severe snow avalanches.

(xx) Steel fiber reinforced shotcrete (SFRS) is found successful in tunneling in the squeezing ground conditions and weak rock masses. The invert should also be shotcreted in squeezing conditions, so that stable rings are formed.

(xxi) Invert struts should be used to enable steel ribs to withstand high wall support pressure in the squeezing ground.

(xxii) *A good bond should be ensured between shotcrete lining or concrete lining and rock masses; so that the bending stresses in the lining are drastically reduced, especially in the highly seismic areas.*

(xxiii) In highway tunnels, the thickness of shotcrete should be increased by 50mm to take care of corrosion due to toxic fumes of vehicles.

(xxiv) In the steep slopes in hard rocks, the half tunnels should be planned along hill roads.

(xxv) In the rock burst prone areas (H>1000m and J_r/J_a >0.5), full-column-resin-grouted rock bolts should be used to make the brittle rock mass as ductile mass which should be further lined by ductile SFRS to prevent rock bursts.

(xxvi) There should be international highway network in Himalaya for its rapid economic development.

6 CONCLUDING REMARKS

Tunneling in the Himalaya is a challenging task because of difficult and varying geology, the rock masses are weak and fragile undergoing intense tectonic activities resulting into major faults, folds and other discontinuities, high in situ stresses, water-charged formations, etc. Each tunnel in the Himalaya poses some problem or another. Majority of the problems are related to water-in-rush, roof falls, cavity formation, face collapse, swelling, squeezing, support failure, gas explosion etc. As such, the experiences of tunneling in the Himalaya have helped in understanding the rock behavior and also developing the state-of-the-art of tunneling.

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