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Optimizing Support Systems for Railway Tunnel Based on Rock Mass Classification and Ground Deformation Monitoring in Complex Shiwalik Terrains of Lesser Himalayas - A Case Study

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ABSTRACT

The construction of Tunnel 8 in the Bhanupali-Bilaspur-Beri Rail Project (BBBRP) demonstrates the effective application of advanced geotechnical solutions to overcome complex geological challenges in the Shiwalik Himalaya. The tunnel traverses through highly unstable formations, including weathered siltstone, fractured sandstone, and clay-rich zones, leading to severe engineering challenges such as tunnel convergence due to ground squeezing, groundwater ingress of up to 400 L/min, and cavity formation. To address these issues, a New Austrian Tunneling Method (NATM) approach combining geological mapping, rock mass classification (RMR), staged excavation, and instrumented deformation monitoring was adopted. Rock bolts, lattice girders, multi-layered shotcrete, and pipe umbrella systems ensured immediate and long-term stability. Additionally, cementitious grouting carried out through pipe roofing systems and Self-Drilling Anchor (SDA) rock bolts effectively filled the existing cracks and fissures in the rock mass, thereby improving ground integrity, enhancing load-bearing capacity, and significantly reducing deformation and instability risks. Instrumentation and monitoring facilitated adaptive construction management, allowing for timely design modifications based on ground response. This study is based on extensive field investigations, including direct geotechnical data collection, ground deformation monitoring, and decision-making for support optimization in tunneling through complex geotechnical environments.

Keywords: NATM, Geotechnical challenges, Rock reinforcement, Support systems, Monitoring

1. INTRODUCTION

Tunneling in the Himalaya presents significant challenges due to complex geology, high seismic activity, and extreme ground deformation. The region's weak, highly fractured rock masses, fault zones, and variable lithology lead to stability issues such as squeezing, rock bursts, and water ingress (Singh & Goel, 2011). Adaptive support systems of New Austrian Tunneling Method (NATM), like rock bolting, steel ribs, and wiremesh-reinforced shotcrete have proven effective in stabilizing tunnels under high-stress conditions (Goel & Jethwa, 2008). Continuous monitoring using 3D targets, borehole extensometers, and microseismic analysis has helped to assess deformation patterns in the field. By integrating systematic geological monitoring with optimized reinforcement techniques, engineers have enhanced tunnel safety and longevity in the challenging Himalayan terrain (Calvi, 2025).

The Bhanupali-Bilaspur-Beri New Rail Project is a crucial infrastructure initiative aimed at enhancing rail connectivity in Himachal Pradesh, India. Spanning approximately 63 km, this broad-gauge railway line is being developed by Indian Railways to improve transportation for passengers, freight, and military logistics. The project is expected to boost regional economic growth by facilitating trade and tourism while also providing a strategic transport link to the Himalayan region. The railway alignment passes through a challenging hilly terrain, requiring the construction of multiple tunnels, bridges, and embankments. Given its location in the Shiwalik Himalaya, the project demands advanced engineering solutions to navigate complex geological conditions. Once completed, the Bhanupali-Bilaspur-Beri Rail Line will significantly reduce travel time, improve accessibility, and support the socio-economic development of the region, making it a transformative addition to India's railway network.

The Shiwalik Himalaya present significant challenges for the Bhanupali-Bilaspur-Beri Rail Project, primarily due to weak and unstable geological formations. The region consists of loosely compacted sandstone, siltstone, and clay, which are highly prone to erosion, weathering, and landslides. Tunnel construction in these formations is difficult, as soft rock layers can collapse under excavation-induced stress, leading to squeezing and deformation. Additionally, the high seismic activity in the Himalaya increases the risk of ground displacement and structural instability, necessitating the use of seismic-resistant designs (Yousuf and Bukhari, 2019a&b). Another major challenge is groundwater ingress, as the presence of water-bearing strata can weaken tunnel linings and increase excavation difficulties. Overcoming these challenges requires a combination of modern tunneling techniques and continuous geological assessments, making this project a benchmark in Himalayan railway construction.

In this study, engineering challenges encountered during the construction of Tunnel 8 (T8) of the Bhanupli-Bilaspur-Beri Rail Project (BBBRP) are highlighted. The tunnel's alignment traverses the geologically complex Himalayan terrain, primarily composed of the Lower and Middle Shiwalik formation characterized by soft sandstone, siltstone, clay, and boulder conglomerates. The terrain exhibits highly fractured and weathered rock masses, variable overburden thickness, and groundwater ingress, necessitating the implementation of a robust geotechnical support system Additionally, the presence of major thrust faults, including the Bhakra, Barsar, and Himalayan Frontal Thrust (HFT), along with seismic activity and potential slope instability, underscores the critical need for systematic geological assessments and deformation monitoring. This study is based on systematic direct field observation and deformation monitoring carried out during the construction of Tunnel 8. Direct field monitoring refers to on-site geological mapping of exposed tunnel faces after each excavation round, recording of rock mass characteristics (lithology, joint condition, groundwater condition, RQD estimation), in-situ rock mass classification (RMR), and continuous measurement of tunnel deformation using installed instruments such as 3D convergence targets, multipoint borehole extensometers (MPBX), and load cells. In addition, daily site inspections, support performance assessments, and documentation of groundwater inflow were undertaken to evaluate ground response to excavation. The geotechnical data collected from these field observations and instrumented measurements were systematically analyzed to assess rock mass behavior and to modify the excavation sequence and support density wherever required, ensuring safe and efficient tunneling through the complex Shiwalik geological conditions.

2. GEOLOGICAL AND GEOTECHNICAL SETTINGS

2.1 Geological Conditions

The Bhanupali-Bilaspur-Beri Rail Project (BBBRP) is located in the northwestern Himalayan foothills, where the geological setting is primarily controlled by the Shiwalik Group formation (Figure 1). This region has undergone significant geological transformation due to active tectonics, faulting, and sedimentary deposition, leading to a complex geological environment. The Shiwalik Group, which dominates the project area, represents fluvial sediment deposits from the middle Miocene to early Pleistocene, primarily consisting of sandstone, claystone, siltstone, and conglomerates. These formations were deposited by ancient river systems under varying energy conditions and have been subsequently deformed due to Himalayan orogenic processes (Bhargava and Singh, 2021).

The Shiwalik formation is further divided into three formations, Lower, Middle, and Upper. The Lower Shiwalik formation predominantly comprises fine to medium-grained micaceous sandstone interbedded with red and purple clays. The sandstone within this formation is moderately compact, while the claystone layers exhibit significant shearing and fracturing, particularly in regions influenced by faulting and thrust movements. This formation is frequently encountered near fault zones, where extensive deformation has occurred over time. In contrast, the Middle Shiwalik formation consists of grey to greenish-grey fine to medium-grained sandstone interbedded with yellow and brown clays. The sandstone here is moderately strong but exhibits moderate-to-high jointing, making it more susceptible to weathering and slope instability, which pose additional challenges for tunnel stability and support design. The Upper Shiwalik formation, which is generally weaker than the lower units, comprises grey, friable sandstone interbedded with clay, siltstone, and conglomerates. This unit is often covered with slope wash material, particularly along tunnel alignments, and poses significant tunneling challenges due to its weak composition, requiring stabilization measures during construction. These units are often highly sheared and fractured, particularly in zones associated with intra-formational thrusts like the Bhakra/Soan, Barsar, Nalagarh, and Jwalamukhi thrusts, which displace Lower Shiwalik units over Middle and Upper Shiwalik. Additionally, quaternary deposits, found mainly in the southwestern (SW) sections of the alignment, consist of river gravels, terrace deposits, and alluvial sediments. These poorly consolidated materials can impact foundation stability, particularly for bridges and other infrastructure elements (Bhargava and Singh, 2021).

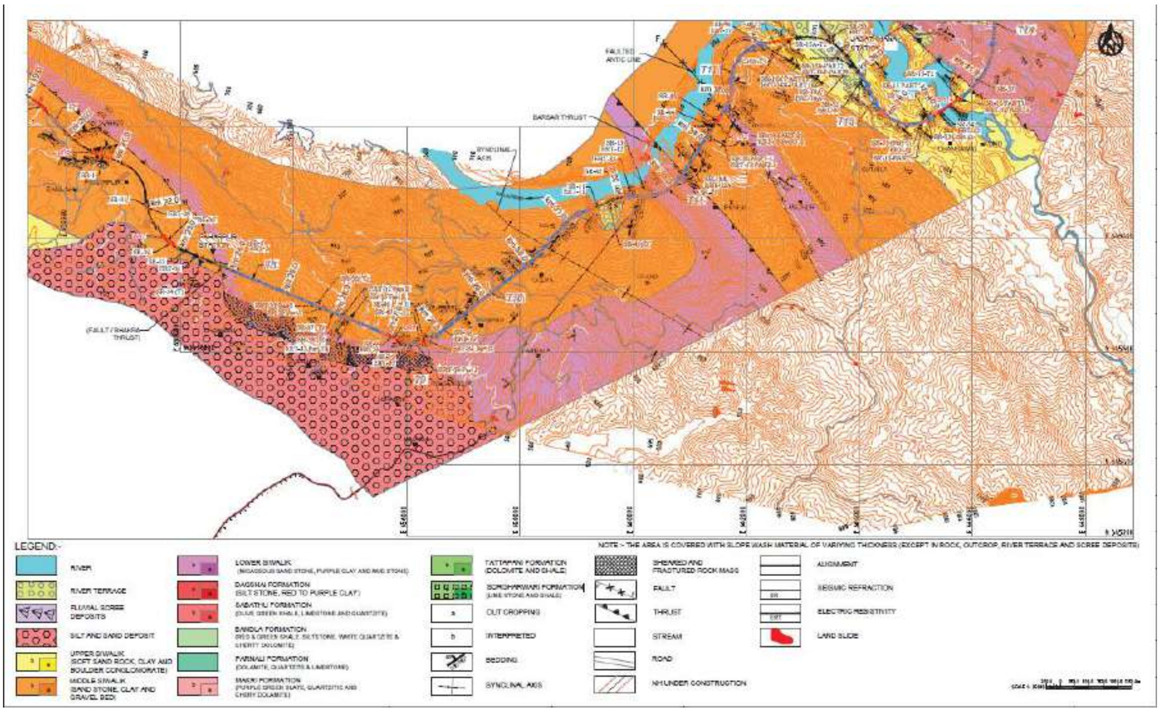

Figure 1- Geological cross section along the tunnel alignment, T8

Tunnel 8 of the BBBRP spans a length of 2,932.817m, extending from chainage 24+054.315m to 26+987.162m. Tunnel 8 is expected to be primarily housed within the Lower to Middle Shiwalik formation (Figure 2), which consists of soft sandstone, siltstone, and clay with occurrences of boulder conglomerate. The excavated tunnel size is 9.06m in diameter and 9.516m in height, with a construction tolerance of 150mm. The finished tunnel size is 7.60m in diameter and 6.481m in height, having a horseshoe profile. Excavation was carried out using NATM in staged sequences comprising heading, benching, and invert. The theoretical excavation quantities are 36.59m³/m for heading, 19.58m³/m for benching, and 16.28m³/m for invert. Both mechanical excavation and drill-and-blast methods were adopted depending upon rock mass conditions. The tunnel gradient is 1% from portal 1 (formation level RL 470.107m) to portal 2 (formation level RL 500.423m). The maximum rock cover above the tunnel roof is 127m at chainage 26+750m, while the minimum rock cover is 1.89 m at Chainage 26+980m. Excavation commenced in March 2022 and was completed on 26 June, 2025. The secondary lining consists of a 300 mm thick RCC lining and Steel Fibre Reinforced Concrete (SFRC) adopted in designated stretches as per design requirement.

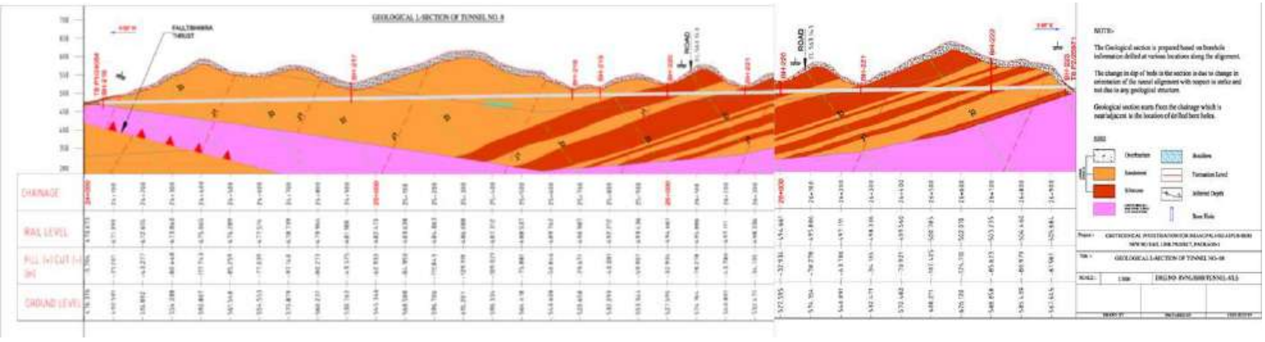

Figure 2- Geological L-section of Tunnel 8

Eight boreholes (BH-1 to BH-8) were drilled along Tunnel 8 between Chainage 24.064 km and 26.940 km based on detailed geological mapping and anticipated lithological variations, particularly at portal zones and expected weak stretches (Table 1). The boreholes were planned along the tunnel alignment at standard investigation intervals. Borehole depths range from 30 m to 95 m and were extended below the tunnel invert level to adequately characterize the strata influencing tunnel stability. The predominant lithology encountered is sandstone, with interbedded siltstone in BH-5, BH-6, and BH-8, while BH-7 shows multiple sandstone bands indicating variable stratification. At Portal 1, the formation level is RL 470.107 m and the tunnel roof RL is 477.318 m, while at Portal 2, the formation level is RL 500.423 m, and the tunnel roof RL is 507.634 m. Overall, the borehole data indicate a weak and fractured Shiwalik formation with localized groundwater presence (notably in BH-6), necessitating systematic support and groundwater control measures during excavation.

Table 1 - Borehole investigation summary along T8

BH. No	Depth (m)	Chainage (km)	Overburden Depth (m)	Rock type	RL (m)	RQD (%)	Ground Water Table (m)
BH 1	30	24.064	3	Sandstone (3-30)	472	Nil-36	27
BH 2	55	24.915	19.5	Sandstone (19.5-55)	480.5	Nil-20	15
BH 3	30	25.673	3	Sandstone (3-30)	490.5	Nil-25	11
BH 4	30	25.769	4.5	Sandstone (4.5-30)	492.5	Nil-66	10
BH 5	40	26.000	3	Sandstone (3-6), Siltstone (6-40)	495	Nil-10	12

BH 6	31.5	26.266	3	Siltstone (3-31.5)	497.5	10-99	1.3
BH 7	95	26.700	13.5	Sandstone (13-19.5, 25.5-43.5, 55.5-95, multiple bands)	501	Nil-99	3.5
BH 8	30	26.940	16.5	Siltstone (16.5-30)	504.5	Nil-99	3.5

2.2 Geophysical Survey for Subsurface Profiling

Seismic Refraction Tomography (SRT) was conducted along the tunnel alignment to characterize subsurface stratification and rock mass variability. A linear geophone array with 5m spacing and approximately 120m spread length was deployed, with overlapping profiles to ensure continuous coverage across the portal and critical zones. First-arrival travel times were processed using tomographic inversion to generate P-wave velocity sections. The derived velocity ranges were correlated with borehole lithology and geological mapping, and the thickness values presented in Table 2 represent averaged stratigraphic layers along the alignment. Interpretations are based on correlation with borehole data. No deformation assessment has been made solely from velocity measurements.

Table 2- Seismic velocity with thickness and interpreted lithology from geophysical survey along Tunnel 8

Seismic P-Wave velocity range (m/s)		Thickness range (m)		Interpreted lithology from borehole & geophysical survey
From	To	From	To	
389	1096	0.1	6.7	The top layer comprises unconsolidated soil (loose in nature) mixed with boulders
1096	1154	4.5	23.4	Compacted strata comprising highly weathered sandstone or claystone
2036	3550	5	30	Moderately to slightly weathered sandstone/ claystone, zone of highly to moderately weathered sandstone/ shear zone

2.3 Geotechnical Investigation

The laboratory tests had been performed to characterize the on-site rock, soil, and water in boreholes, and to obtain geotechnical parameters for engineering analysis and design (Table 3). The tests were performed in accordance with the Indian standard code.

Table 3- Summary of test results on soil and rock samples of T8

Sr. No.	Tests Conducted	Tunnel 8			Numbers of samples
		Min.	Max.	Average	
1.	Dry Density (g/cc)	2.07	2.69	2.49	20
2.	Specific Gravity	2.55	2.73	2.67	8
3.	Bulk Density (g/cc)	2.28	2.68	2.56	15
4.	Moisture Content (%)	3.67	12.79	12.73	2
5.	UCS (MPa)	43.43	102.32	74.67	10

6.	Point Load Index (MPa)	0.99	60.87	27.88	13

Based on the geological and geotechnical investigations, the tunnel alignment was divided into stretches according to the expected rock mass class. The corresponding tunnel lengths for each rock mass class are presented in Table 4, and a description of ground type and its RMR range in Table 5, which formed the basis for preliminary NATM support design and excavation planning.

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Table 4- Ground type classification along Tunnel 8 as per design ONORM B2202 (af. Oct,94)

Tunnel 8 Chainage (m)		Ground Type (GT)	Behaviour Types (ONORM)
From	To		
24+054	24+300	GT5, GT2	C3,C4,B2,B3
24+300	25+200	GT3, GT5	C3,C4,A2,B1,B2
25+200	25+750	GT3, GT2, GT5	A2,B1,B2,B3,C3,C4
25+750	26+200	GT4,GT5, GT2	B3,C2,C3,C4,B2,B3
26+200	26+600	GT4, GT5, GT3	B3,C2,C3,C4,A2,B1,B2
26+600	26+971	GT4, GT5, GT2	B3,C2,C3,C4,B2,B3

Table 5- Description as per design ONORM B2202 (af. Oct,94) used for rock mass classification with ground type behavior

Ground Type (GT)		ONORM B 2023 (af. Oct, 94)	
		RMR	Behavior Type
GT 1	Claystone with thin bands of sandstone	100-80	A1 (Stable)
		80-65	A2 (Slightly stable)
GT 2	Sandstone (thinly bedded)	65-58	B1 (Friable)
		58-47	B2 (Very friable)
GT 3	Sandstone (Bedded)	47-29	B3 (Rolling)
		29-20	C1 (Rock bursting)
GT 4	Siltstone (micaceous fine grained)	20-15	C2 (Squeezing)
		15-10	C3 (Heavily Squeezing)
GT 5	Conglomerate, scree deposits, RBM deposits	10-5	C4 (flowing)
		<5	C5 (swelling)

3. DESIGN OF SUPPORT SYSTEM

The tunnel design was established before construction based on detailed geological and geotechnical investigations. Tunnel 8 was designed following the New Austrian Tunneling Method (NATM), considering the surrounding rock mass as a load-bearing component. Expected rock mass classes along the alignment were determined from borehole data, geological mapping, RQD, and RMR evaluation, and the corresponding tunnel length versus predicted rock mass class is presented in a separate table (Tables 4 & 5). The initial support systems (Table 6) were derived using empirical correlations between ONORM-based ground type classification and NATM support guidelines, supplemented by established design charts and regional tunneling experience. Support measures, including pipe roofing, rock bolts, wiremesh, shotcrete, and steel ribs, were assigned based on the anticipated rock mass class and overburden conditions. The designed primary support capacity was based on composite action between shotcrete, rock bolts, 3-chord lattice girders (LG 130×25×25 mm), as shown in Figure 3, and the surrounding rock mass.

During excavation, supports were reviewed and modified based on face mapping and in-situ RMR assessment after each round of excavation. NATM governed the overall design philosophy, while RMR served as the quantitative basis for selection and adjustment of supports in the field.

Table 6- Design of T8 supports for different rock mass class/support class

Rock Mass Class	Piperoof/Forepole	Rock Bolts	Wiremesh	Shotcrete	LG/Rib
III (RMR31-40)	32 mm ϕ SDA Forepole, 6m length, 32 no. @300 mm c/c, 20 degree upward	SDA / 32 mmϕ / 4m / 1.5×1.5m c/c staggered	Double layer wiremesh 150*150*6mm	250mm	130*25*25 @1.5m
IV (RMR 21-30)	114.3mmϕ, 6.3mm thick / 15m / 23 no./@420 mm c/c/ 3.34º upward	SDA / 32mmϕ / 4m / 1.5×1.5 m c/c staggered	Double layer wiremesh 150*150*6mm	300mm	130*25*25 @0.75m
V (RMR<20)	114.3mmϕ, 6.3mm thick / 15m / 27 no./ @350 mm c/c/ 3.34º upward	SDA / 32mmϕ / 6m / 1.0×1.0 m c/c staggered	Double Layer wiremesh 150*150*6mm	300mm	130*25*25 @0.5m
Portal Class	114.3mmϕ, 6.3mm thick /15m/ 32 no./ @300mm c/c/ 3.5º upward	SDA / 32mmϕ / 6m / 1.5×1.5 m c/c staggered	Double layer wiremesh 150*150*6mm	250mm	ISMB 150 @1.0m

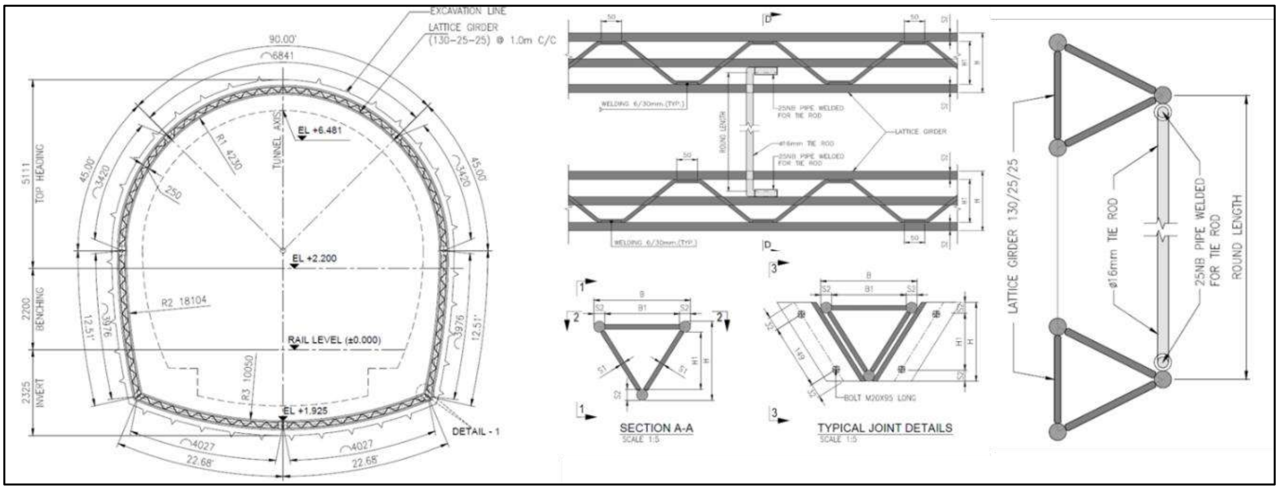

Figure 3- Tunnel 8 cross-section with lattice girder details

In Class IV and V zones and portal stretches, a 114 mm diameter pipe-roof umbrella was provided within the 120° crown zone as pre-excavation support. The umbrella length was 15 m with 3 m overlap between successive rounds and an inclination of approximately 3.5° above horizontal. Installation of one umbrella round required approximately 24-36 hours, depending on ground conditions. Rock bolts of 32 mm diameter (4–6 m long) were not placed in the crown where pipe-roofing or forepole was adopted; they were installed only below the pipe-roofed area, except for about 120° of the crown in Class IV and V rock class, where additional reinforcement was necessary. Rock bolts were arranged in a staggered pattern between lattice girders to form a continuous reinforced support ring. Double-layer wiremesh in Classes IV, V, and Portal Class was provided to enhance surface confinement in highly fractured rock. At the portal zones, consolidation grouting (cementitious) was carried out before excavation to improve ground conditions. The portal class length at both portals was determined based on geological mapping, weathering depth, and shallow overburden conditions, extending up to the transition into competent rock mass. This length was finalized during detailed design and confirmed during initial excavation. Within this stretch, primary support was installed as per Table 6, including ISMB 150 ribs spaced at 1.0 m to ensure stability under portal and shallow cover conditions.

Support installation followed by NATM sequencing: pre-excavation support (pipe umbrella where required), controlled excavation, immediate sealing layer of shotcrete, erection of lattice girders, fixing of first-layer wiremesh, application of first-layer shotcrete, installation of second-layer wiremesh where specified, completion of shotcrete to the total designed thickness, and installation of SDA/rock bolts. The shotcrete thickness specified in Table 6 represents the total thickness applied in layers.

4. ROCK MASS CLASSIFICATION OF TUNNEL 8

During the construction of Tunnel 8, the Rock Mass Rating (RMR) system proposed by Bieniawski (1989) was adopted for geological characterization and stability assessment. RMR evaluation was carried out during excavation based on six parameters: (i) UCS of intact rock, (ii) RQD, (iii) joint spacing, (iv) joint condition, (v) groundwater condition, and (vi) joint orientation adjustment. The tunnel alignment was classified into Support Classes III, IV, and V, along with Portal Class zones as per the encountered rock mass in the field. Weak stretches, particularly near portals and shear-affected zones, recorded RMR values between 18-28 (Class V-IV), associated with low RQD, closely spaced weathered joints, and damp to wet conditions. Comparatively competent stretches exhibited RMR values of 31-40 (Class III). The chainage-wise distribution of rock mass classes, along with corresponding UCS, RQD, and derived shear strength parameters (cohesion *c* and friction angle *φ*), is presented in Table 7, and spatially illustrated in the geological longitudinal section (Figure 4).

Table 7- RMR, support class and shear parameters obtained during T8 excavation

S. No.	Chainage (m)		Support class	RMR	UCS (MPa)	RQD	Shear Parameters (range)	
	From	To					Cohesion (c, MPa)	Friction Angle (φ, °)
Tunnel T8P1								
1	24054.32	24062.07	False Portal	NA	NA	NA	NA	NA
2	24062.07	24118.32	Portal Class	NA	<1	NA	0.091-0.020	25-37
3	24118.32	24464.32	SC-V	<20	1-25	<25	0.091-0.020	25-37
4	24464.32	24500.32	SC-IV	21-30	5-50	25-50	0.096-0.170	37-47
5	24500.32	24632.32	SC-V	<20	1-25	<25	0.091-0.020	25-37
6	24632.32	24668.32	SC-IV	21-30	5-50	25-75	0.096-0.170	37-47
7	24668.32	25028.32	SC-V	<20	1-25	<25	0.091-0.020	25-37
8	25028.32	25055.4	SC-IV	21-30	5-50	25-50	0.096-0.170	
Tunnel T8P2								
1	26987.3	26982.3	False Portal	NA	NA	NA	NA	NA

2	26982.3	26948.28	Portal Class	NA	<1	NA	0.096-0.170	25-37
3	26948.28	26876.3	SC-V	<20	1-25	<25	0.091-0.020	25-37
3	26876.3	26804.3	SC-IV	21-30	5-50	25-50	0.096-0.170	37-47
4	26804.3	26679.8	SC-V	<20	1-25	<25	0.091-0.020	25-37
5	26679.8	26612.3	SC-IV	21-30	5-50	25-50	0.096-0.170	37-47
6	26612.3	26565.32	SC-III	31-40	25-100	50-100	0.0262-0.270	42-53
7	26565.32	26294.3	SC-IV	21-30	5-50	25-50	0.096-0.170	37-47
8	26294.3	26016.6	SC-V	<20	1-25	<25	0.091-0.020	25-37
Tunnel T8P3								
1	25677.4	25652.4	Portal Class	NA	1-25	<25		25-37
2	25652.4	25208.4	SC-V	<20	1-25	<25	0.091-0.020	25-37
3	25208.4	25124.4	SC-IV	21-30	5-50	25-50	0.096-0.170	37-47
4	25124.4	25055.4	SC-V	<20	1-25	<25	0.091-0.020	25-37
Tunnel T8P4								
1	25702.6	25727.6	Portal Class	<20	1-25	25-50	0.091-0.020	25-37
2	25727.6	25859.6	SC-V	<20	1-25	<25	0.091-0.020	25-37
3	25859.6	25883.6	SC-IV	21-30	5-50	25-50	0.096-0.170	37-47
4	25883.6	26016.6	SC-V	<20	1-25	<25	0.091-0.020	25-37

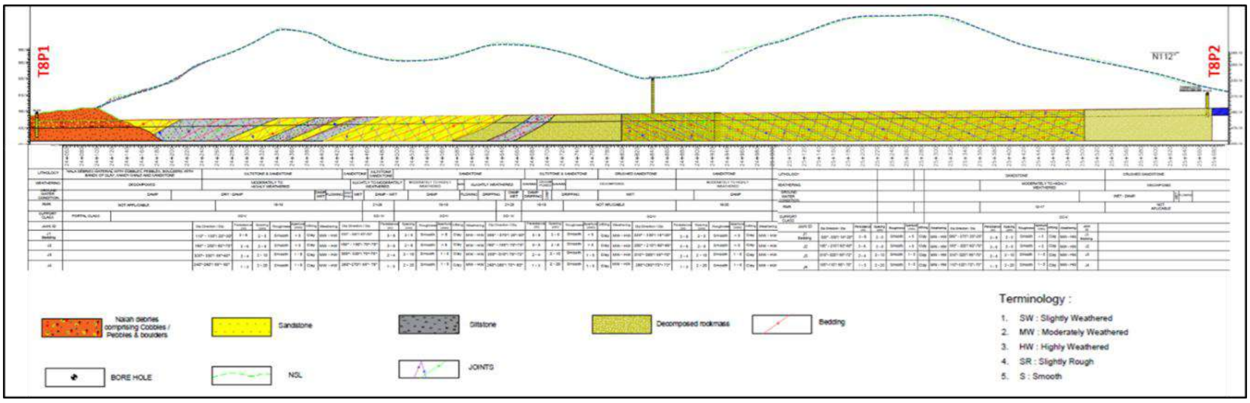

Figure 4- Geological L-section of Tunnel 8 as per encountered rock mass during excavation

The NATM-based support design was directly correlated with RMR classification. RMR provided the quantitative basis for identifying rock mass class at the excavation face, while NATM governed the staged excavation and support philosophy. Initial support types were pre-defined for each RMR class, and after each excavation round, face mapping and RMR reassessment were performed. Any reduction in RMR due to increased fracturing, groundwater ingress, or adverse joint orientation triggered modification of support measures such as reduced spacing of ribs, increased shotcrete thickness, or installation of forepoling. Thus, RMR enabled systematic field-based adjustment of supports within the NATM framework.

Unlike the Q-system, which primarily relates support to rock mass quality through stress-reduction factors, RMR was preferred for its direct applicability to face mapping and ease of rapid field estimation under variable Himalayan geology. The integration of RMR classification with NATM allowed site-specific adaptation to heterogeneous Shiwalik formation, including weathered sandstone, claystone bands, and localized shear zones, thereby ensuring stability control, optimized support deployment, and safe excavation under complex geological conditions.

4.1 Expected vs Encountered (Actual) Geological Conditions

During the design phase of Tunnel 8, the Geological Baseline Report (GBR) predicted predominantly moderately weathered sandstone and siltstone of the Shiwalik Group with interbedded clay layers and localized fault zones. The anticipated RMR ranged between 25–37 (Class III–IV), with UCS values expected between 40–75MPa in competent stretches (Figure 5). Groundwater levels were estimated at depths of 10–99 m, requiring standard drainage provisions. However, excavation revealed greater geological variability than predicted. Extensive stretches, particularly near portals and shear-affected zones, exhibited highly fractured and blocky rock mass conditions corresponding predominantly to Class V (RMR < 20). The encountered UCS ranged from <1 MPa to 25 MPa, significantly lower than GBR estimates, as illustrated in the graph below. RQD values were frequently below 25%, indicating poor rock mass quality.

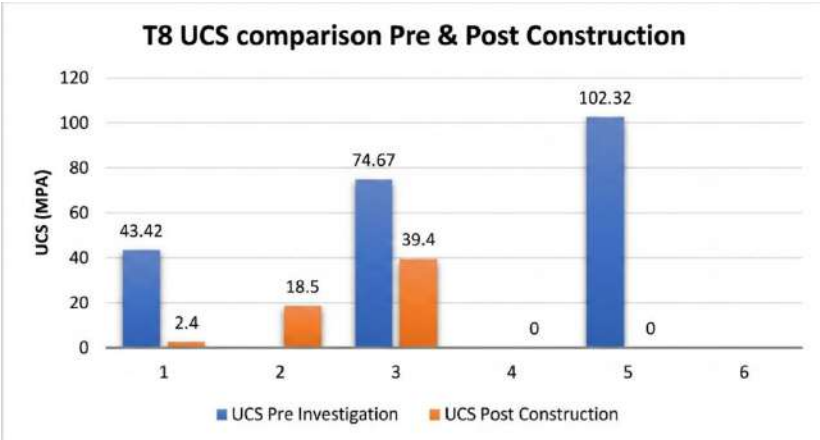

Figure 5- Pre and post-excavation UCS comparison of Tunnel 8

Groundwater conditions also deviated from predictions, with localized higher inflows and comparatively shallower water table conditions in faulted zones, necessitating enhanced drainage and systematic cementitious grouting. Structural discontinuities were more persistent and closely spaced than anticipated, with larger apertures contributing to block instability and reduced stand-up time.

Overall, the encountered geological conditions were weaker and more heterogeneous than predicted in the GBR. This necessitated continuous geological face mapping, systematic deformation monitoring, and adaptive modification of support measures within the NATM framework to maintain stability and construction safety in the variable Shiwalik Formation.

5. CONSTRUCTION METHODOLOGY

The construction of Tunnel 8 was carried out using the New Austrian Tunneling Method (NATM), which utilizes the load-bearing capacity of the surrounding rock formation and permits staged support installation based on observed ground behavior and monitoring. The excavation strategy was developed from prior geological and geotechnical investigations and implemented in accordance with predicted rock mass classes and overburden conditions. A staged excavation sequence comprising heading, benching, and invert was adopted. The heading was excavated first, followed by the bench, and subsequently the invert to achieve early ring closure. In weak rock mass zones (Classes IV and V), invert closure was carried out at the earliest feasible stage to ensure formation of a complete structural ring and to enhance confinement, thereby limiting deformation in the heterogeneous and shear-affected Shiwalik formation.

Excavation of Tunnel 8 was initiated from Portals T8P1 and T8P2 under moderate overburden conditions, enabling systematic implementation of the NATM approach with sequential installation of primary supports, while at the junction section excavation proceeded simultaneously from Portals T8P3 and T8P4 to optimize construction time; in shallow cover stretches near the junction, the cut-and-cover method was adopted to ensure stability and control surface deformation. Mechanical excavation was adopted for the majority of the tunnel, achieving an average advance rate of approximately 1.0–1.5 m/day in comparatively weaker Class IV–V rock mass conditions due to the requirement of intensive pre-support and stabilization measures; however, in relatively harder Class III rock mass, the advance rate further reduced to about 0.75–1.0 m/day owing to increased excavation resistance. In contrast, controlled drill-and-blast excavation was applied in relatively competent stretches of Class III and IV rock mass, achieving an average progress of around 1.5–2.0 m/day, depending on blasting cycle efficiency, ventilation clearance, and support installation requirements.

Figure 6- Construction process & support system (a) Piperoof Installation (b) Face Excavation & Mucking (c) Lattice Girder Erection (d) Shotcrete Application

The construction sequence under NATM included: (i) installation of pre-support such as pipe-roof umbrella where required, (ii) controlled face excavation and mucking, (iii) erection of lattice girders, (iv) installation of rock bolts in a staggered arrangement to develop a reinforced support ring, (v) placement of wiremesh, and (vi) staged application of shotcrete to the specified total thickness (Figure 6). This systematic approach ensured controlled deformation, structural stability, and safe advancement under variable formation and overburden conditions.

6. INSTRUMENTATION

In accordance with the principles of the New Austrian Tunneling Method (NATM), a systematic instrumentation and monitoring program was implemented during the excavation of Tunnel 8 to support the observational design approach.

3D Monitoring Targets were installed at closer spacing in weaker rock masses to ensure higher data resolution in potentially unstable zones. The spacing was 15 m in Class V, 20 m in Class IV and 25 m in Class III. Multi-point borehole extensometers (MPBX) were installed at approximately 500 m intervals, or as required by site-specific geological conditions, to monitor subsurface deformation at depths ranging from 0–2 m, 2–5 m, 5–10 m, and 10–20 m. Vibrating wire load cells mounted on selected rock bolts were placed at about 300 m intervals to assess axial load development, while pressure cells were installed at approximately 500 m spacing in weak zones, low cover stretches, and shear-affected sections to monitor stress redistribution in the lining and surrounding rock mass.

Table 8- Tunnel 8 instrumentation and monitoring record format

FREQUENCY OF MEASUREMENT UPON TUNNEL EXCAVATION		
WEEK 1 WEEK 2 WEEK 3 & 4 After 4 WEEKS	Once per Day Twice a week Once per Week Once Every Two Months	
INSTRUMENTATION INTERVALS FOR DIFFERENT CLASSES		
Rock Class	Interval for Instrumentation	
	Standard Monitoring by Targets	Main Monitoring with Extensometers
Class I Class II Class III Class IV Class V	50m 30m 25m 20m 15 m	500 m (or as per site conditions)
Pressure cell spacing for different classes: 500m (or as per site conditions)		
Vibrating wire load cell mounted on rock bolts: 300 m (or as per site conditions)		

Tunnel convergence monitoring was carried out using optical targets (Model AIS 1700) and high-precision total stations. The 3D displacement components (x, y, z) of crown, springline, and benching points were recorded. Radial deformation was determined directly from inward displacement toward the tunnel centerline. Tangential deformation (ovalization) was evaluated from relative displacement between opposing targets (crown–benching and springline–springline), allowing assessment of asymmetrical stress redistribution and shear-induced distortion, which are common in heterogeneous Himalayan formations. The instrumentation and monitoring data were recorded as per the format shown in Table 8.

A three-tier "Review Level" system was adopted to define trigger criteria for dynamic support adjustment. For 3D radial deformation, the Threshold Level (Alert) ranged from 30–75 mm depending on rock mass class, requiring a 100% increase in monitoring frequency. The Allowable Level (Alarm), ranging approximately from 40–120 mm, required detailed investigation and review of support adequacy. The Design Level (Action), generally 100 mm for Class III–V and up to 150 mm in portal zones, mandated immediate stoppage of excavation and implementation of remedial measures such as increased shotcrete thickness, reduced ISMB rib spacing, or additional bolting. Similarly, stress limits for support elements were strictly governed; for Self-Drilling Anchor (SDA) rock bolts, an alarm value of 210 kN was established as the maximum tensile capacity before structural intervention. Pressure cell trigger, alert, and alarm limits were defined in the range of 2–7.5 MPa to assess lining stress conditions. This structured, threshold-based monitoring framework provided objective criteria for support modification and excavation control, ensuring tunnel stability and safe advancement under variable geological conditions.

7. PROBLEM FACED DURING EXCAVATION

7.1 Squeezing Ground Conditions

Squeezing ground conditions were encountered between Ch. 24+714 m and 24+745 m in Tunnel 8, corresponding to Rock Class V with RMR ≈ 18 and an overburden ranging from 90.5 m to 86.0 m above the tunnel crown. The large cover in this stretch contributed to elevated in-situ stresses, estimated at approximately 3.74 MPa, comprising vertical stress from overburden weight and corresponding horizontal stress due to lateral confinement.

The rock mass in this section primarily comprised highly weathered sheared sandstone, siltstone, and clay-rich gouge material, all exhibiting weak mechanical properties and low resistance to deformation. Faulted rock and shear zones further exacerbated the issue by reducing rock competence and facilitating uncontrolled inward movement and excessive convergence. High moisture content and the swelling potential of clay-rich material further weakened the rock mass, promoting plastic deformation and increasing the risk of tunnel convergence. At Ch. 24+730 m (Figure 7), heading excavation was completed on 04-02-2024, and 3D target monitoring commenced on 23-02-2024. The benching section was excavated on 10-05-2024, and deformation data for targets T6 and T7 (bench portion) were recorded from 15-05-2024 onward. Targets T1 to T5 were installed in the heading portion. The monitored resultant radial displacement in the heading reached approximately 60 mm between 23-02-2024 and 19-04-2024. However, a further increase of about 28 mm was observed after 19-04-2024, which is significant and is attributed to stress redistribution and disturbance caused by subsequent benching excavation. Thereafter, deformation stabilized, with negligible additional movement recorded up to 26-07-2024. The maximum recorded total deformation in TP4-TP5 reached 118.1mm. The average closure rate during the initial monitoring phase was approximately 5-8 mm/day, based on incremental displacement trends. Figure 7 indicates that maximum chord convergence occurred between targets T4 and T5 (horizontal axis), whereas crown settlement remained comparatively small. This behavior reflects dominant horizontal squeezing governed by lateral stress redistribution and shear deformation within weak, sheared rock mass. The deformation pattern is therefore consistent with anisotropic stress redistribution rather than uniform radial closure.

7.1.1 Ground stabilization and support strategies

Squeezing conditions in Tunnel 8 were managed through a systematic NATM-based approach incorporating staged excavation (top heading, bench, and invert) to enable controlled stress redistribution and early ring closure. Immediate primary support was provided after each excavation round using 100–150 mm thick shotcrete and Self-Drilling Anchors (SDA), 32 mm diameter and 9m length, installed at 1.0m × 1.0m staggered spacing around the tunnel periphery to enhance confinement and shear resistance of the weak rock mass. In Class V stretches, systematic pre-support and grouting measures (as specified in Table 3) were implemented. Cementitious grouting (w/c ≈ 0.45) improved rock mass competence by filling fractures and increasing stiffness, while strategically placed drainage boreholes reduced pore pressure and prevented softening of clay-rich zones.

Support optimization was strictly governed by instrumentation data and predefined trigger levels. 3D monitoring targets measured radial and chord convergence at the crown, spring level, and benching locations. When radial displacement approached the Threshold (≈60 mm), the monitoring frequency was increased. At the Allowable level (≈80 mm), support was strengthened by installing additional SDA bolts and increasing shotcrete thickness (up to 100 mm additional layer). At the Design (≈100 mm; higher in portal zones), excavation was temporarily halted, and early invert closure was executed to mobilize full ring action.

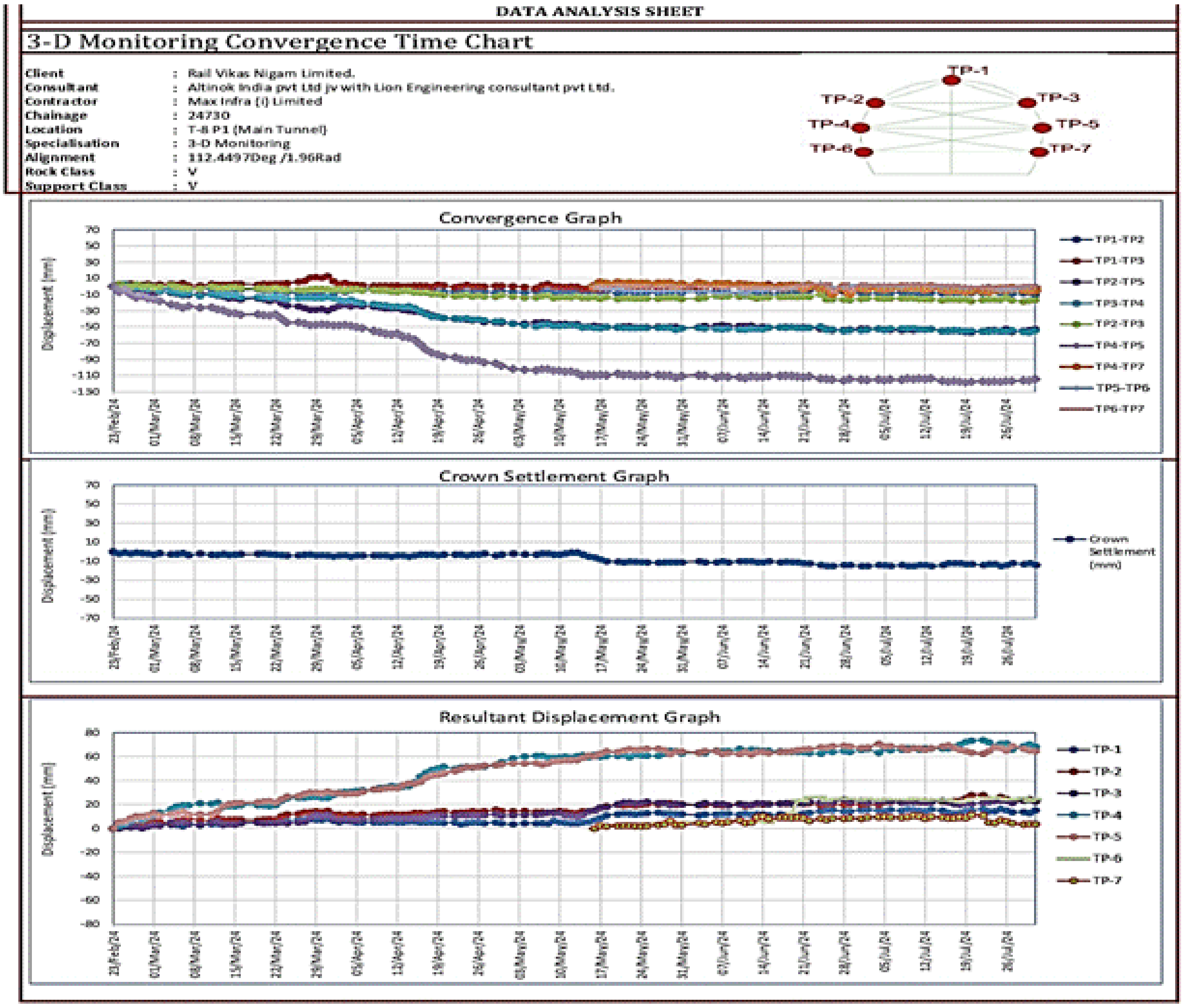

Figure 7- 3D monitoring data chart at ch:24+730.00m

MPBX results indicated that maximum displacement occurred within the shallow anchors (0–2 m and 2–5 m depths), whereas deeper anchors (10–20 m) showed comparatively limited movement. This confirmed that deformation was largely confined to the near-boundary plastic zone and did not extend deeply into the rock mass. Based on this interpretation, strengthening measures focused on enhancing surface confinement rather than modifying deep anchorage length.

Monitoring data, therefore, played a decisive role in construction control. Progressive displacement trends identified active squeezing during heading excavation, while stabilization after support enhancement and invert closure confirmed adequacy of the revised support system. Load cell readings (Figure 8) ensured that bolt stresses remained within permissible limits, and pressure cell data verified lining stress performance. This instrumentation-based feedback mechanism enabled objective, data-driven decisions, ensuring safe excavation and optimized support under variable and weak Himalayan rock mass conditions.

7.2 Cavity Formation

During excavation at Ch. 24+404.315 m of Tunnel T8 (Portal-1), a major cavity developed on the left side of the tunnel face on 14-02-2023 at 10:02 hrs (Fig. 4a). The stretch corresponded to weak rock conditions (RMR=13), classified as Support Class V, with rock cover of 100.20m. The exposed face comprised moderately weathered fine-grained siltstone interbedded with highly fractured sandstone. Geological mapping indicated wet to dripping conditions, with soft clayey infill along joints and closely spaced discontinuities, resulting in low rock mass cohesion and poor stand-up time.

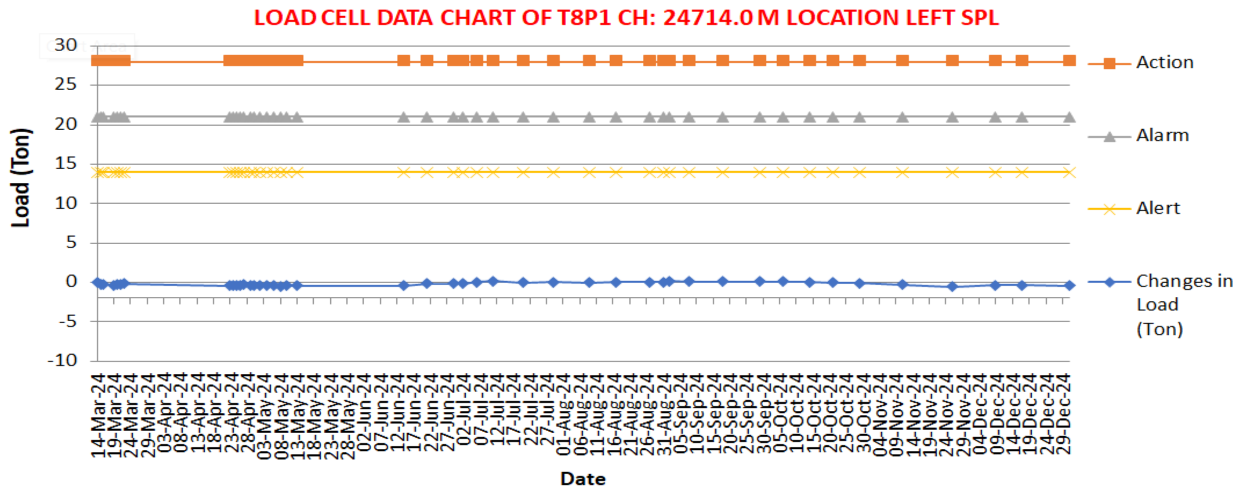

Figure 8- Load cell data chart at ch:24+714.00m of Tunnel 8

As per support provisions discussed in Table 3, the primary support element comprising 32 numbers of 114.3mm diameter, 6.3mm thickness, 15m long pipe roofing @ 300mm spacing at an angle of 3.5° has been installed at Chainage 24+394.315km (Figure 9). During the installation of pipe roofing, considerable water seepage had been recorded. It was noticed that the water seepage from these pipes originated after 12 m. Consequently, discharged water from these pipes had been diverted to one side of the tunnel face, to ensure uninterrupted work activities and further progress of tunnel excavation. The water seepage from these 4 pipes was recorded around 30-50 liter / minute.

Figure 9- Water seepage during pipe roofing at Chainage 24+394.315m and diverted

The excavation of the tunnel has been completed till Chainage 24+404.315m i.e., around 11m from pipe roof chainage, and further implementation of support elements is in progress. Suddenly, from the left side of the face huge water comes and formed a cavity in the left side of the tunnel face. During geological mapping also the face has been recorded in wet to dripping condition only. The heavy water flow around 250-300 Liter / minute from left side of the face has eroded weak rock material and triggered material displacement, reducing the effective shear strength of the rock mass and compromising stability. It has been observed the cavity formed is around 5-7m in height. The failure mechanism was attributed to the combined effects of: (i) highly fractured sandstone with low cohesion, (ii) soft infilling along joint planes reducing effective shear strength, (iii) concentrated groundwater inflow causing erosion and loss of fines, and (iv) lithological contact instability at the sandstone–siltstone interface acting as a preferential plane of weakness.

The excessive groundwater inflow reduced effective stress within the rock mass, triggered progressive loosening, and caused localized detachment at the crown and left shoulder, ultimately resulting in cavity formation. The incident highlights the combined influence of poor rock mass quality (low RMR), structural discontinuities, and concentrated groundwater flow in destabilizing the tunnel face despite pre-installed pipe-roof support.

7.2.1 Remedial measures implemented for cavity stabilization at tunnel T8

To restore tunnel stability and prevent further cavity expansion at Chainage 24+404.3m, a systematic and phased approach was adopted, focusing on immediate stabilization, structural reinforcement, and long-term monitoring.

- **Immediate Stabilization and Drainage Management:** The first step involved halting excavation activities to prevent additional material detachment and collapse. Simultaneously, 12 to 15m long random drainage holes have been installed at angle of 15° to 20° upward to divert the water seepage from face to pipe and prop support has been installed at deformed Lattice girder. The area has been covered with GI sheets for further implementation of other measures like filling of cavity area. The rate of water flow was same however it is diverted to the drainage holes.
- **Cavity Backfilling Strategy:** The cavity, measuring approximately 32m³ and 5-7m in height was backfilled in multiple stages over 4-5 days. The area has been protected by multiple times sealing layer shotcrete consuming $5m^3$ and 32 m^3 quantity of grade M30 A10 has been backfilled or poured through pipes by multiple stages (Figure 10c). The successive shotcrete layers allowed sufficient stand-up time for the backfill material to settle and integrate with the surrounding rock. This staged approach restored structural integrity and prevented further void expansion.
- **Support System Enhancement:** To stabilize the weakened tunnel section, a comprehensive reinforcement system was implemented, integrating multiple support techniques. ISMB 150 steel ribs were installed at 500mm c/c spacing, providing immediate structural stability. Self-drilling anchor (SDA) rock bolts (32mm diameter, 9m length) were placed in a staggered 1.0m x 1.0m c/c pattern, specifically at the crown and left sidewalls, to enhance reinforcement capacity (figure 10d). Shotcrete was applied in successive layers, reinforcing the primary tunnel supports and mitigating deformation risks. Additionally, forepoling and the canopy method, utilizing the piperoof umbrella system, ensured preemptive support for the weak section, facilitating a safer excavation process and maintaining ground stability.
- **Monitoring and Preventive Measures:** To ensure long-term stability, deformation monitoring systems were deployed, including MPBX and convergence meters to track radial and tangential deformation. Monitoring allowed for timely adjustments to the support system as required. Additionally, drain holes were strategically drilled to manage future groundwater seepage, minimizing the risk of similar failures in subsequent excavation zones. This comprehensive and technically robust approach ensured tunnel stability, allowing excavation to progress safely despite the challenging geological conditions encountered.

After cavity filling and installation of the optimized primary support system, redistribution of groundwater flow was observed. A portion of the inflow from the face shifted toward previously excavated stretches where minor seepage had earlier been recorded. The identified seepage zones were located between Ch. 24+394.315–24+382.000 m, 24+143.000–24+162.000 m, 24+181.000–24+221.000 m, and 24+243.000–24+267.000 m. Although seepage in these zones had reduced to damp conditions after initial primary support installation, it subsequently increased to localized dripping through the installed drainage pipes due to hydraulic redistribution within the fractured rock mass.

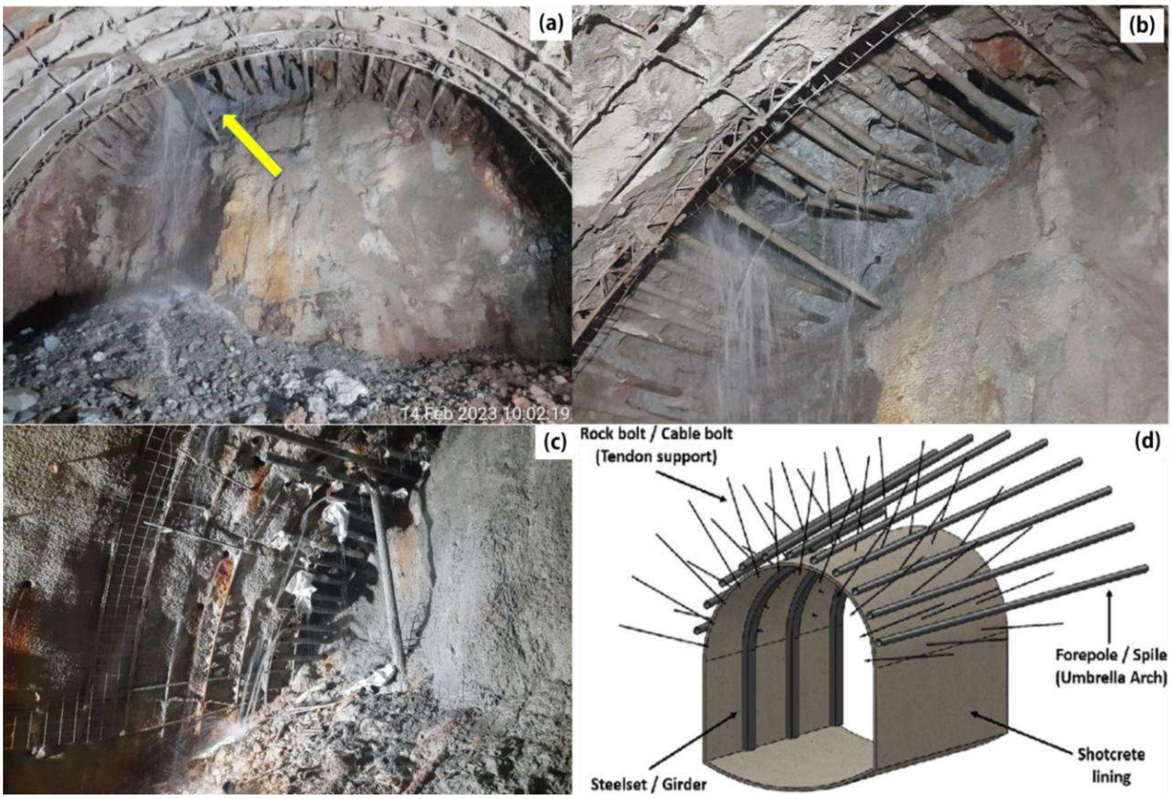

Figure 10- Cavity Formation at Tunnel No. 8 at Ch. 24+404.315m. (a) left side face collapse, cavity formation and water seepage, (b) damage of primary support (c) backfilling of Cavity and grouting (d) techniques employed to restore cavity

Figure 11- Face water seepage diverted in the already identified water seepage zone

To manage this condition, systematic 6 m long drainage boreholes (50 mm diameter) fitted with perforated PVC pipes wrapped in geotextile were drilled in the affected stretches to relieve pore water pressure and intercept seepage paths. Controlled diversion channels were formed to direct water toward designated drainage lines, and a sump with a pumping arrangement was deployed to ensure continuous dewatering and maintain dry working conditions.

For further advancement of excavation, the next round of pipe roofing was installed at Ch. 24+404.315 m to provide pre-support and intercept water-bearing discontinuities ahead of the face. However, seepage continued through canopy pipes in the crown region. The cumulative water discharge at this stage was measured at approximately 300–330 L/min, which was managed through the combined drainage, diversion, and pumping system to enable safe continuation of tunnel excavation.

8. CONCLUSIONS

The construction of Tunnel 8 in the Bhanupali-Bilaspur-Beri Rail Project highlights the effective management of difficult subsurface conditions through innovative geotechnical interventions and flexible excavation practices. The tunnel alignment through the fragile and heterogeneous Shiwalik formations encountered several challenges, including squeezing ground behaviour, groundwater ingress, cavity formation, and weak fractured rock masses. These conditions required continuous assessment of ground response and timely modification of excavation and support measures to maintain stability, safety, and construction progress.

The project demonstrates the importance of integrating detailed geological investigations, systematic monitoring, and adaptive support systems during tunnel excavation in complex terrain. The engineering measures adopted during construction helped control geotechnical risks, minimise delays, and ensure the long-term stability of the tunnel. The experience gained from this project provides valuable guidance for the planning and execution of underground works in similar geological environments and contributes to the development of safer and more efficient tunnelling practices worldwide.

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