Italian Journal of Engineering Geology and Environment
www.ijege.uniroma1.it

EVALUATING TUNNEL STABILITY IN CHALLENGING TERRAINS: A COMPREHENSIVE STUDY OF GEOLOGICAL FACTORS AND SUPPORT SYSTEM IN THE LOWARI TUNNEL, NORTHERN PAKISTAN

Jehanzeb KHAN[(*,**)], Waqas AHMED[(*)], Muhammad WASEEM[(***)], Ihtisham Islam[(****)] & Salman Ahmed KHATTAK[(*****,******,*******)]

[(*)]*University of Peshawar - National Centre of Excellence in Geology - 25130 Peshawar (Pakistan)*
[(**)]*University of Malakand - Department of Geology - 18800 Chakdara, Dir (L), (Pakistan)*
[(***)]*University of Engineering and Technology, Peshawar - 25120 Peshawar (Pakistan)*
[(****)]*Shaheed Benazir Bhutto University - Department of Geology - 18030 Sheringal, Dir Upper, (Pakistan)*
[(*****)]*Institute of Geochemistry - State Key Laboratory for Critical Mineral Research and Exploration - Chinese Academy of Sciences - 550081 Guiyang (China)*
[(******)]*University of Chinese Academy of Sciences - 100049 Beijing (China)*
[(*******)]*The University of Haripur - Department of Earth Sciences - 22620 Haripur (Pakistan)*
Corresponding author: salman@mail.gyig.ac.cn

EXTENDED ABSTRACT

Questo studio esamina i fattori critici che influenzano la stabilità e la sostenibilità di un tunnel in uno sfidante ambiente geologico himalayano, con particolare attenzione al progetto del Lowari Tunnel. Attraverso una mappatura geologica completa, un monitoraggio rigoroso e una valutazione approfondita del sistema di supporto, sono state acquisite diverse informazioni chiave sull'interazione tra le condizioni del terreno e le prestazioni del tunnel. La mappatura dettagliata del fronte del tunnel ha identificato un'ampia gamma di tipologie di roccia (dal granito alle metavulcaniche), stati di alterazione (prevalentemente freschi con alcune zone localizzate di alterazione moderata) e condizioni delle falde acquifere (per lo più secche con alcune zone di gocciolamento verso zone di scorrimento). L'attività di mappatura ha individuato con successo aree potenzialmente problematiche: ad esempio, rumori di fessurazione udibili e lievi scheggiature nella corona sinistra alle progressive ~0+925–0+936 hanno allertato il team sulla presenza di rocce sottoposte a forte stress, e la presenza di un liscione di faglia con breccia alla progressiva ~1+002–1+008 è stata correlata a piccoli distacchi di frammenti. Questi incidenti sottolineano l'importanza di una mappatura geologica precisa e di osservazioni del fronte di scavo per prevedere i problemi di stabilità prima che si aggravino. Valutando costantemente le condizioni del fronte di scavo e adattando gli approcci di scavo, il progetto ha evitato crolli di notevole entità e gestito i rischi associati alla complessa geologia. L'ampia rete di 269 stazioni di monitoraggio delle deformazioni ha consentito una valutazione completa della stabilità del tunnel in tempo reale. Il monitoraggio ha rivelato che la deformazione verso l'interno (convergenza) era la risposta dominante: circa il 93% degli spostamenti misurati era verso l'interno, a indicare che l'ammasso roccioso circostante generalmente comprimeva sul supporto del tunnel anziché allentarsi verso l'esterno.Questo risultato positivo, conferma l'efficacia della strategia NATM e dell'installazione del supporto nel mantenere il tunnel in uno stato di equilibrio compressivo. L'intervallo di convergenza (da 0 a ~36 mm) osservato lungo il tunnel, si è allineato bene con le condizioni geologiche: convergenze maggiori si sono verificate in condizioni di maggiore copertura e in roccia più debole e fratturata (ad esempio, una convergenza di -36,4 mm in una zona di granodiorite altamente alterata), mentre le sezioni superficiali e in roccia resistente hanno visto solo movimenti trascurabili. I rari casi di movimento verso l'esterno sono stati ricondotti a eventi specifici come le rotture di scavo, rafforzando il fatto che non si trattasse di problemi sistemici. Nel complesso, i dati di monitoraggio indicano che il comportamento deformativo del tunnel è rimasto entro limiti di sicurezza durante la costruzione, dimostrando il successo del progetto nel sopportare i carichi del terreno. L'accurata installazione e i rigorosi test dei sistemi di supporto del tunnel hanno confermato la loro efficacia nel mantenere la stabilità in diverse condizioni geologiche. I rivestimenti in spritzbeton hanno raggiunto la resistenza richiesta e hanno fornito un confinamento continuo alla superficie rocciosa. Bulloni, reti metalliche e travi reticolari hanno tutti soddisfatto o superato i criteri di progetto nei test di qualità e, soprattutto, hanno funzionato insieme come un sistema per prevenire qualsiasi crollo anche nelle sezioni più impegnative. Il risultato è stato che non si sono verificati cedimenti significativi dei supporti e il tunnel non ha subito deformazioni incontrollate. Le prestazioni del sistema di supporto evidenziano l'importanza del rispetto degli standard di controllo qualità (ad esempio, la corretta coppia di serraggio dei bulloni, la verifica dello spessore dello spritzbeton) e dell'adattamento dell'intensità del supporto alla classe di scavo. Adattando il tipo e la capacità del supporto alla classe del terreno (con un supporto pesante solo in terreni di scarsa qualità), il progetto ha garantito sia la sicurezza che l'efficienza. Il caso del tunnel Lowari dimostra l'efficacia del NATM in condizioni geologiche difficili, dove la mappatura e il monitoraggio continui consentono adeguamenti tempestivi del supporto. Lo studio enfatizza l'indagine geologica approfondita e misure proattive come il drenaggio e il rinforzo. Una combinazione ben progettata di spritzbeton, bulloni, rete e travi garantisce la stabilità anche in rocce deboli e tettonicamente attive. La ricerca futura dovrebbe valutare queste strategie in diversi contesti (tipi di roccia e condizioni), metodi di costruzione e i loro impatti economici e a lungo termine.

IJEGE, 1 (2025): 55-66, DOI: 10.4408/IJEGE.2025-01.O-07
E-ISSN 2035-5688 | ISSN 1825-6635 / ©Author(s)

ABSTRACT

This study investigated the stability and sustainability of the Lowari tunnel constructed in the geologically challenging Himalayan terrain by analyzing geological parameters, deformation monitoring, and support system performance. Detailed face mapping across the 8.509 km tunnel revealed highly variable rock types, weathering conditions, and discontinuities -ranging from stable, compact granite to fractured and sheared zones requiring enhanced stabilization. Groundwater ingress in critical sections further exacerbated stability challenges. A network of 269 monitoring stations provided comprehensive deformation data, with 93% of stations recording inward displacements (0.000 m to -0.0364 m), confirming the effectiveness of the new Austrian tunneling method (NATM) in managing excavation-induced stresses. Support systems, including shotcrete (28-day compressive strength: 28.6-30.8 MPa), rock bolts (pull-out load >165 kN), wire mesh, and lattice girders, demonstrated reliable performance, ensuring structural integrity under varying geological conditions. Over-breaks-predominantly in jointed and sheared zones-emphasized the need for refined excavation techniques and real-time monitoring to mitigate avoidable instabilities. The findings underscore the adaptability of NATM, the importance of accurate geological mapping, and the effectiveness of robust support systems in ensuring tunnel stability.

Keywords: *deformation monitoring, support systems, himalayan geology, tunnel stability, sustainable construction*

INTRODUCTION

The intricate interplay of geological hazards, excavation techniques, and support systems is critical in determining the success of tunnel construction. While immediate operational safety remains paramount, the challenge of ensuring long-term sustainability of these vital infrastructure assets is equally significant (Zhang, 2023). Geological factors such as overburden depth, rock mass characteristics, groundwater presence, and even abnormal temperatures or gases profoundly influence tunnelling practices (Zhang, 2023). In particular, detailed mapping of tunnel faces is essential in unpredictable rock formations or areas with substantial water ingress, playing a crucial role in safety and risk mitigation during excavation (Georgiou *et alii*, 2022). A notable example underscoring the importance of precise face mapping is the collapse of the Hanekleiv Tunnel in Norway, which highlights the need for thorough geological evaluation and proactive risk assessment in tunnel projects (Bjørn, 2011). The selection of appropriate tunneling techniques is contingent on the geological strata encountered during excavation. Tunnel deformation (including longitudinal and horizontal displacements) varies significantly with rock conditions (Wang *et alii*, 2022). Controlled blasting techniques, especially at tunnel portals where stress concentrations occur, are vital for preventing roof failures and ensuring structural integrity (Huang *et alii*, 2015; Han *et alii*, 2016). Moreover, the efficacy of tunnel support systems is paramount: continuous monitoring is essential to maintain stability, optimize excavation cycles, and verify the effectiveness of support measures (Chang *et alii*, 2020; Ma *et alii*, 2022). Support systems can be categorized into primary and secondary supports, both integral to preserving stability and managing earth and water pressures (Du *et alii*, 2014). The required excavation support sheet (RESS) is a critical document that provides detailed instructions for support installation following each blast, allowing adaptation of support to the varying geological conditions encountered (Lunardi & Barla, 2014).

In the modern tunneling industry, four primary excavation techniques are prevalent: The classical Austrian tunneling method, the new Austrian tunneling method (NATM), tunnel boring machines (TBM), and the sequential excavation method (Wang et alii, 2019). NATM is particularly notable for its adaptability to diverse ground conditions, emphasizing precise monitoring and deformation control with excavation classes tailored to ground behaviour (Wang *et alii*, 2019). This method strategically addresses weak zones through immediate support installation, as demonstrated in the Lowari tunnel project (Marcher *et alii*, 2020). Following the adoption of NATM globally, shotcrete has emerged as the dominant material for primary tunnel support due to its durability and cost-effectiveness-essential qualities for maintaining stability in challenging ground (Aygar, 2022). Additionally, rock bolts (such as SN bolts and Swellex expandable bolts) play a critical role in controlling deformations and reinforcing the rock mass (Zhang *et alii*, 2022). The use of wire mesh further strengthens tunnel walls, while lattice girders (though shaped to fit the tunnel profile) contribute incrementally to support stiffness (Song *et alii*, 2022). Ensuring that all support elements meet quality standards is crucial; this includes verifying shotcrete compressive strength and the mechanical properties (yield strength, ductility, etc.) of steel supports like bolts, mesh, and girders (Li *et alii*, 2012).

The primary aim of this research is to evaluate the stability of the Lowari Tunnel under challenging geological conditions by examining geological factors, excavation-induced deformations, and the performance of various support systems. The findings will inform the development of robust design and maintenance protocols, thus promoting sustainable tunnel construction practices in similar geotectonic settings.

GEOLOGY OF THE STUDY AREA

The Lowari Tunnel is situated in the Hindu Kush region of northern Pakistan and traverses a complex geological setting comprising five main geological units: a granite unit, a biotite-granite unit, a meta-igneous unit, a meta-sediment unit, and a meta-volcanic unit (Tahirkheli, 1979). The granite unit consists of weathered biotite-rich granite. The biotite-granite unit represents a transition from biotite-rich granite to massive granite and is also deeply weathered. The meta-igneous unit is dominated by ortho-gneiss with some amphibolite; it exhibits textures ranging from massive to jointed and shows distinct gneissic banding (Tahirkheli, 1979; Waseem *et alii*, 2018). This unit is mostly competent rock with only localized sheared segments. The meta-sediment unit includes metamorphosed sedimentary rocks (notably schists and slates) which are less extensive in the tunnel but can introduce weaker zones. The meta-volcanic unit consists of metamorphosed volcanic rocks and banded volcanics interlayered with amphibolite; exposures of metavolcanics are generally minimally weathered, with joints often stained or slightly open due to slope relaxation. Overall, these metamorphic units are predominantly composed of competent rock (Tahirkheli, 1979; Waseem *et alii*, 2018). The Lowari Tunnel's alignment passes through each unit in sequence, reflecting the region's lithological diversity.

Figure 1 presents the aerial view and geological cross-section of the Lowari Tunnel alignment length 8.509 km), illustrating the spatial distribution of major geological units along the tunnel and key geotechnical parameters. The tunnel alignment (horizontal axis in chainage) is color-coded by geological unit (e.g., granite, granodiorite, amphibolite), and overlain with plots of rock mass quality (Rock Mass Rating values), measured tunnel deformation magnitudes, and the locations/volumes of significant over-break events. Notably, sections with lower rock quality (e.g., heavily jointed or sheared zones) coincide with higher deformation and larger over-break volumes, whereas zones of intact, competent rock exhibit minimal deformation and negligible over-break (modified after Khan *et alii*, 2022).

METHODOLOGY

Figure 2 presents a simplified flowchart of the research methodology employed in this study to systematically address the complex interplay of geological factors, excavation techniques, and support systems. The selection of an appropriate tunneling method was guided by a multitude of factors, including ground

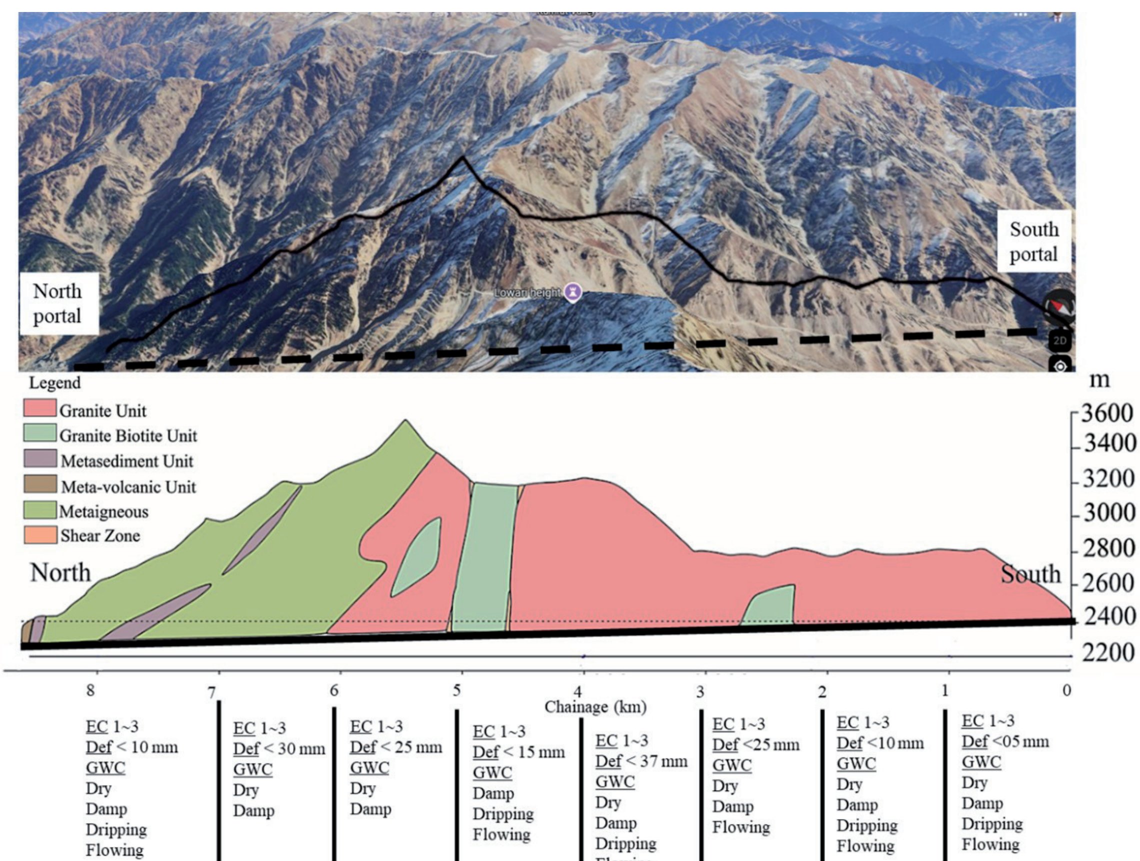

Fig. 1 - Aerial view and geological cross-section of the Lowari Tunnel alignment, showing lithological units, shear zones, excavation classes (EC), deformation (Def.), and groundwater conditions (GWC). The dashed line indicates the tunnel axis profile. (Khan et alii, 2022)

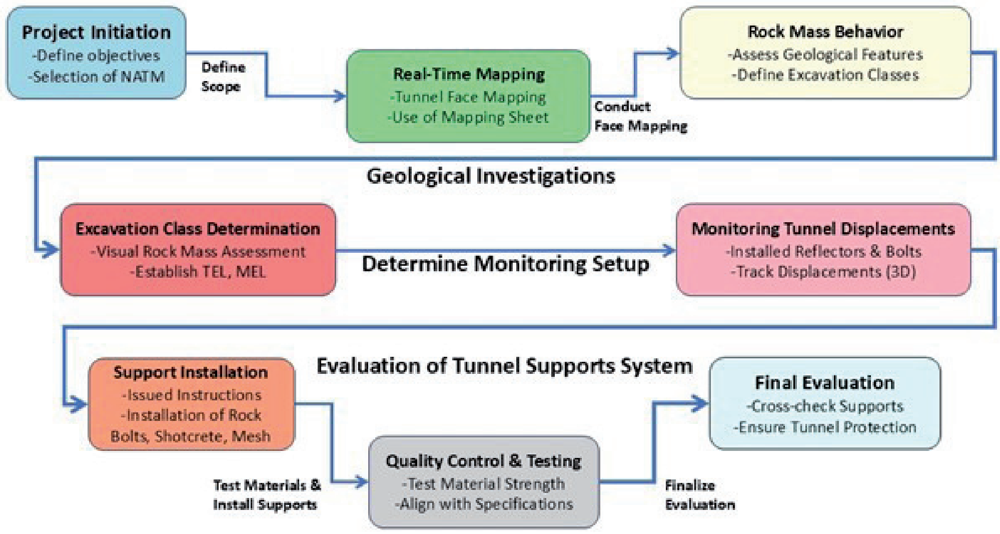

Fig. 2 - Simplified flowchart of the research methodology for tunnel stability analysis

conditions, water inflows, tunnel length and cross-section, depth (overburden), available logistical support, and effective risk management strategies. Given these considerations, the Lowari Tunnel project adopted the NATM approach due to its inherent adaptability to diverse geological conditions and its emphasis on careful monitoring of tunnel deformation with tolerance criteria suited to different excavation classes (ZBIGNIEW *et alii*, 2018).

A comprehensive geological face mapping program was implemented throughout the 8.509 km length of the tunnel. Mapping was carried out in real-time at every excavation advance, immediately following each blasting round and subsequent clearing operations (scaling, mucking, ventilation, etc.). This ensured that all major geological features encountered-such as fault zones, prominent joints, zones of shearing, and water seepage areas-were promptly and accurately documented. By performing mapping after each round of excavation, the likelihood of missing critical geological information that could affect stability was minimized. All observations were recorded on a standardized face mapping sheet with uniform terminology, in line with NATM specifications (GEOCONSULT/TYPSA, 2005). This standardized approach enabled consistent rock classification and facilitated the prediction of potential deformation or hazard zones ahead of the advancing face.

During mapping, key rock properties and conditions were noted. Rock types were identified based on observable characteristics (color, texture, luster), and the degree of weathering was classified (e.g., fresh, moderately weathered, highly weathered) to infer its impact on the excavation. Discontinuities (including the frequency and orientation of joints, presence of cross-joints or faults) and groundwater conditions (dry, damp, dripping, flowing) were also recorded at each face. These factors were synthesized into a qualitative rock mass behavior rating (RMB) for the face, on a scale from 1 to 3, indicating the expected reaction of the rock mass to the excavation (from Type 1=stable/intact to Type 3=highly fractured/sheared). Based on the observed geological conditions and the RMB assessment, an excavation class (also numbered 1 through 3) was assigned to each segment of the tunnel. Excavation Class 1 corresponds to good ground conditions requiring minimal support, while Class 3 denotes poor ground that demands the most robust support measures. The excavation class designation dictated the support installation for that segment as detailed in the Required Excavation Support Sheet (RESS) for the project. Figure 3 provides photographic examples of the different rock mass conditions encountered, which guided these classifications.

To monitor the tunnel behavior during excavation, a deformation monitoring system was installed in parallel with the face mapping and support installation activities. The monitoring network consisted of 269 survey stations placed at approximately 30 m intervals along the tunnel. At each station, boreholes were drilled into the tunnel lining or surrounding rock, and specialized monitoring bolts with reflective targets (bireflex reflectors) were installed. Using a total station surveying instrument with remote distance measurement capability, three-dimensional deformation measurements were taken regularly at these stations. The monitoring captured vertical convergence (settlement/heave), horizontal convergence, and longitudinal displacement of the tunnel profile. By establishing an initial reading soon after support installation and taking periodic readings thereafter, the time-dependent deformation behavior of each tunnel segment was obtained (LUO *et alii*, 2017). Figure 4 illustrates the typical setup of the deformation monitoring system within the tunnel, including the reflector targets and survey instrument arrangement.

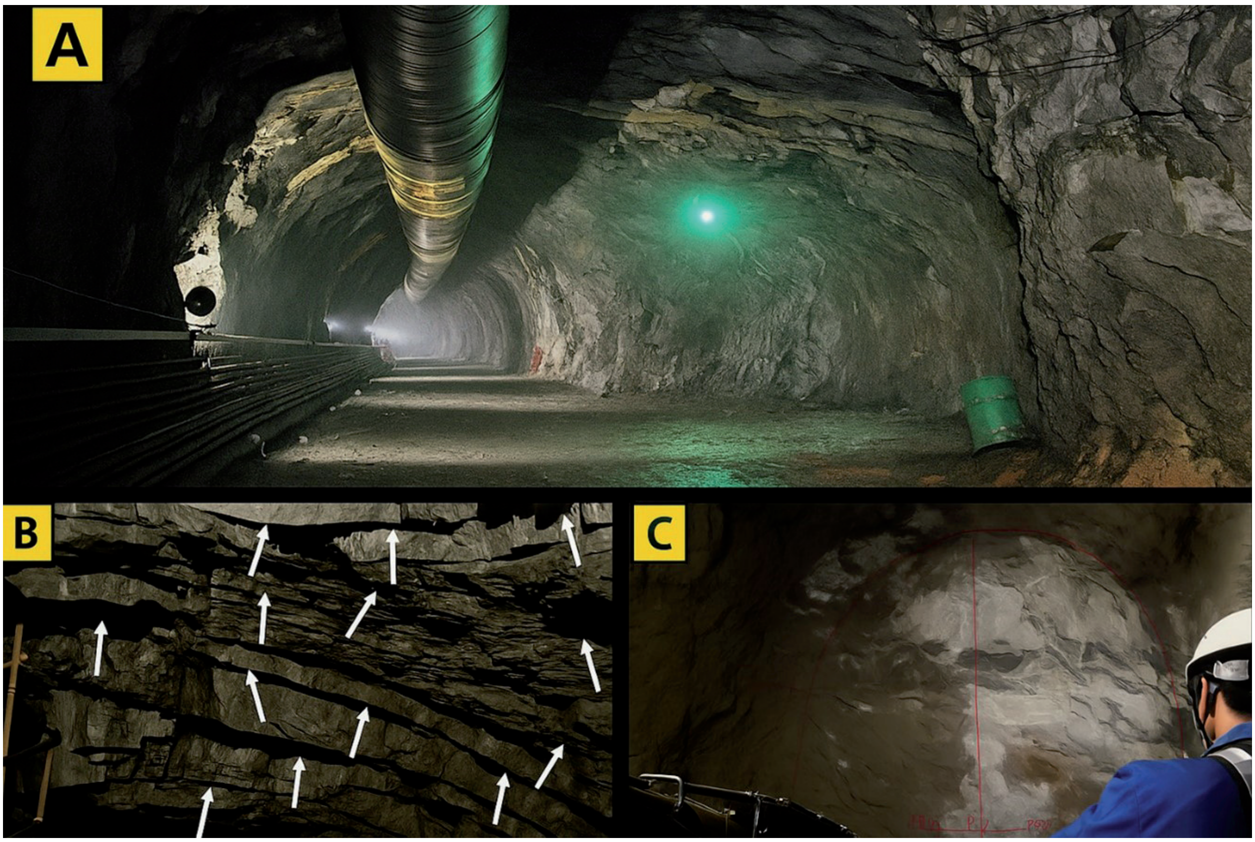

Fig. 3 - Jointing patterns in the Lowari tunnel based on visual assessments of the tunnel face and rock mass behavior at: (a) Inside view of the Lowari tunnel during construction; (b) Chainage1+200 showing moderate jointing; (c) Chainage3+000

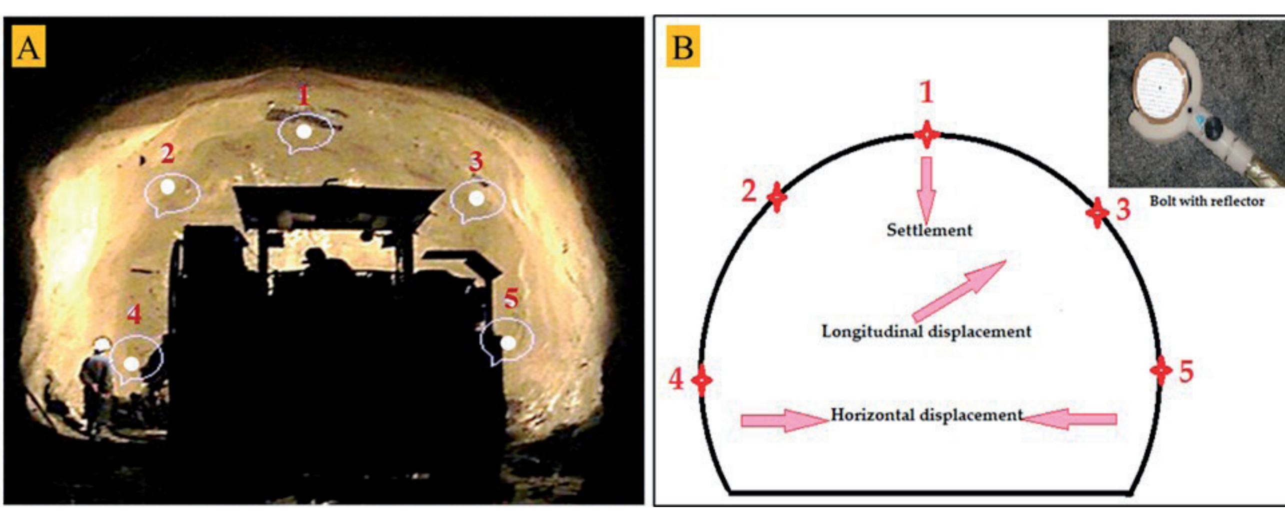

Fig. 4 - Displacement monitoring in the Lowari Tunnel using bireflex reflectors and total station for precise vertical, horizontal, and longitudinal movement tracking

Following the completion of face mapping and initial monitoring for a given segment, the preliminary excavation class assessment could be confirmed or adjusted. The project team then prepared a detailed support installation sheet (an updated RESS entry) for that segment, specifying the type and quantity of supports to install (shotcrete thickness, bolt length and spacing, wire mesh, lattice girder placement if needed, etc.) before the next advance. Installation of rock bolts (including both SN-type and Swellex bolts) was done systematically at the tunnel crown, shoulders, and walls as prescribed. Prior to installation, each batch of rock bolts was subject to quality control tests to ensure they met design specifications (e.g., yield strength, ultimate tensile strength, elongation, bending). Core samples of the shotcrete were taken at regular intervals (every 500 m³ of shotcrete applied) and tested in compression at 3, 7, and 28 days (ASTM C42) to verify that the shotcrete achieved the required strength. Throughout the construction, strict quality control was maintained for all support elements-shotcrete mix and application, rock bolt installation torque and grouting, wire mesh anchorage, and lattice girder placement-to ensure that the actual ground support corresponded to the intended excavation class requirements. The supports installed were carefully inspected and cross-checked against the specified support standards for each class, in light of the prevailing geological conditions and the ongoing deformation measurements. This iterative process of mapping, monitoring, and adjusting support forms the core of the NATM approach used to ensure the long-term structural integrity and safety of the Lowari Tunnel.

RESULTS AND DISCUSSION

Geological Characterization and Overbreak Analysis

The excavation of the Lowari Tunnel from chainage 0+000 to 8+509 encountered a wide range of geological conditions (Fig. 1). In general, the south portal area and early portion of the tunnel (approximately the first 1-2 km) passed through mainly granitic rocks that were relatively intact and strong, whereas the middle to later portions encountered more metamorphic lithologies (gneiss, amphibolite, metasediments) with varying degrees of jointing and shearing. Rock mass conditions ranged from Type 1 (stable, interlocked) in sections of fresh, unweathered granite and granodiorite, to Type 3 (fractured, sheared) in sections of highly jointed gneiss and amphibolite. For example, the initial 200 m of the tunnel (near the south portal in fresh granite, chainage 0+000 to ~0+200) exhibited compact rock with very few joints, resulting in a stable face and requiring only minimal support (Excavation Class 1). In contrast, a segment around chainage 3+250-3+270 (within the granodiorite unit) was found to be heavily fractured and moderately weathered, corresponding to a sheared zone that demanded additional support and was classified as Excavation Class 3. Towards the north end of the tunnel, particularly in the amphibolite and metavolcanic units between chainage ~8+300 and 8+450, the rock mass was densely jointed and included fault gouge in places; these zones had Rock Mass Behavior Type 3 (unstable) and were likewise assigned to Excavation Class 3. Weathering along the tunnel was predominantly fresh rock, with only localized moderate weathering in zones of faulting or near dikes (e.g., a short section in the 3+250 m range as noted above). Groundwater conditions varied along the alignment: the tunnel was largely dry or damp in intact rock stretches, whereas dripping and occasional flowing water was observed in the more fractured sections. Notably, a highly jointed stretch around chainage 8+330-8+400 experienced flowing groundwater influx, which contributed to instability (rock falls) until drainage and support were improved. Overall, the poorer ground conditions (higher joint density, any faulted intervals, and presence of water) tended to coincide with deeper sections of the tunnel and weaker lithologies, while shallower sections in strong granite/granodiorite remained mostly stable with minimal deformation.

Geological over-break (excess excavation beyond the design profile) was a significant challenge in certain segments of the Lowari Tunnel, and its occurrence was strongly tied to the rock mass quality. Over-breaks were documented and categorized as “unavoidable” when caused by adverse geology (e.g., rock mass defects), or “avoidable” when caused by suboptimal excavation practices (workmanship issues). Figure 5 illustrates the variation in overbreak quantities along the tunnel chainage, highlighting the influence of unfavorable joints and shear zones on excavation stability. In total, approximately 1630 m³ of unavoidable over-break was recorded, attributable to difficult geological conditions. The majority of the unavoidable over-break (about 88% of the volume) resulted from sections of unfavourable jointing (UJ) in the rock mass, while the remaining ~12% was due to shear zones (SZ) intersecting the tunnel. The single largest recorded over-break occurred in a zone of intensely jointed rock between chainages 3+337 and 3+436, which alone contributed 577.4 m³ (over one-third of the total geological over-break volume) due to the rock mass fragmenting along closely spaced joints. Another notably large over-break of 115.4 m³ was associated with a sheared fault zone between chainages 3+118 and 3+145. Aside from these extreme cases, most over-break incidents were much smaller (on the order of a few cubic meters to a few tens of cubic meters) and were concentrated in areas with high joint density or minor fault gouge. By contrast, in the sections of strong, blocky granite with low joint frequency, over-break volumes were negligible (often effectively zero, as excavation stayed within profile). Avoidable over-breaks due to operational issues were comparatively limited in volume and tended to occur in otherwise stable ground (indicating that with optimal blasting and excavation control, they could have been prevented). For instance, an over-break at chainage 1+038 in the left crown occurred in a section classified as Excavation Class 1 (good rock); investigation revealed it was caused by exceeding the recommended blast round length, rather than any inherent geological weakness. Similarly, at chainage 1+853, a minor rockfall and over-break in the crown was observed; the presence of visible drilled blast holes in the fallen rock indicated that the blast pattern may have over-cut, and the rock mass itself was later confirmed to be massive and intact - hence this was considered an avoidable incident. These observations indicate that while the overwhelming proportion of over-break volume was driven by geology (jointed or

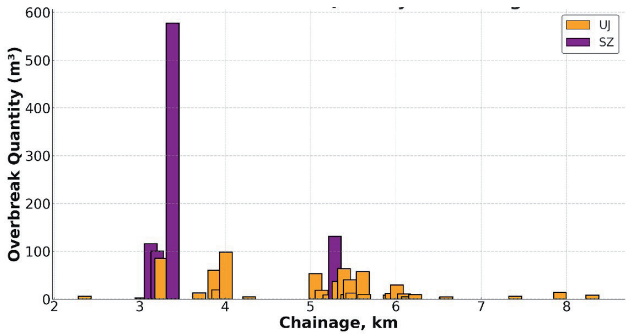

Fig. 5 - Distribution of overbreak quantities along the tunnel alignment as a function of chainage UJ=Unfavorable joints and SZ= Shear zone Peak overbreaks occur between 3.0-3.5 km

sheared rock), careful construction practices are still crucial even in good rock to avoid unnecessary damage.

In summary, the geological mapping and over-break analysis show a clear linkage between rock mass conditions and excavation stability. Sections of the tunnel that exhibited Type 3 rock mass behaviour (highly fractured, jointed, or faulted rock) corresponded to the highest deformation measurements and the largest unavoidable over-break volumes. Conversely, in Type 1 behaviour zones (competent, unfractured rock), the tunnel experienced minimal deformation and virtually no over-break, demonstrating that the initial geological assessment was a reliable predictor of stability performance. This correlation validates the comprehensive face mapping approach and justifies the adjustments made to excavation strategies and support designs in different tunnel segments.

Monitoring and deformations

The Lowari Tunnel's extensive deformation monitoring program provided a detailed record of tunnel response under the varying ground conditions described above. Figure 6 illustrates a representative deformation time-history from one of the monitoring stations (at chainage 8+060) over the period from May 2013 to December 2014. In this example, the plotted displacements (longitudinal convergence, horizontal convergence, and vertical settlement) show that deformations stabilized quickly after the initial excavation of that segment: most movement occurred in the early weeks, and the total magnitudes remained very small (on the order of a few millimeters). The final measurements at this station indicate an overall inward radial displacement of only about 0.3 mm, confirming that the ground-support system effectively contained the excavation with negligible deformation. The largest component of movement at chainage 8+060 was a longitudinal shortening of 4.5 mm (-0.0045 m), which is still minor and within safe limits. This station's data typified the tunnel's behavior in competent rock sections where Class 1 support was applied-essentially stable with only micro-deformations.

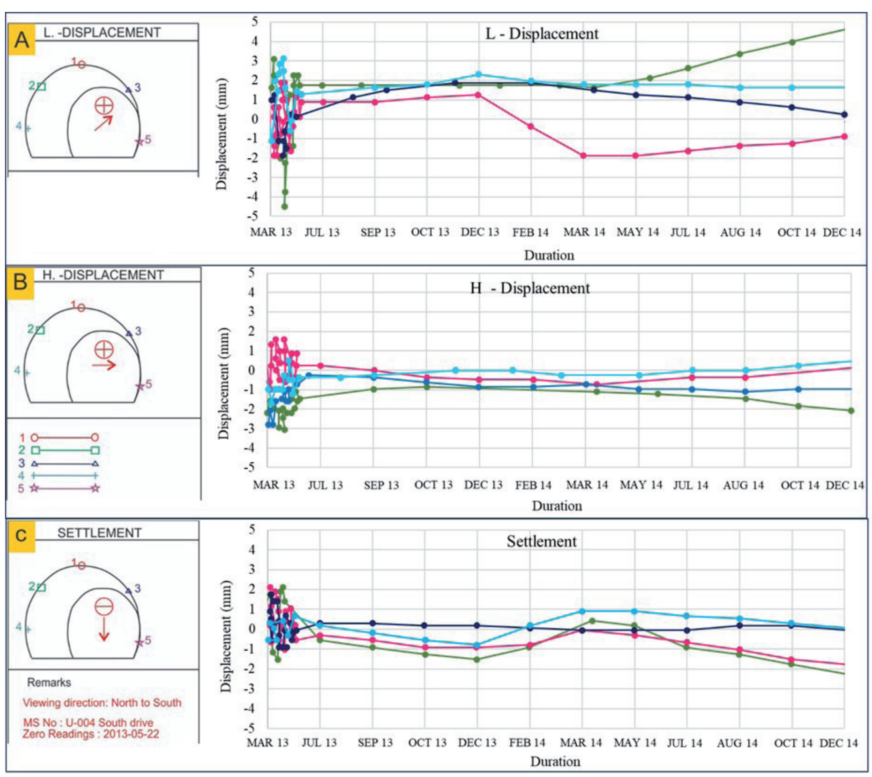

Fig. 6 - Results of typical deformations observed in Lowari tunnel at chainage 8+060 showing longitudinal, horizontal, and vertical displacement over time after excavation (May 2013-Dec 2014)

Displacement analysis

When considering the entire set of 269 monitoring stations, the overall deformation trends are consistent with the geological conditions along the tunnel. Figure 6 presents the distribution of maximum tunnel displacement at each station versus chainage. The measured convergence or divergence ranged from essentially 0 mm (no movement) up to about 36.4 mm at the most extreme location. The vast majority of points recorded inward movement (convergence) of the tunnel walls: 93% of stations showed negative displacement (i.e., the tunnel cross-section tightening under compressive load), while only 18 stations (~7%) registered slight positive displacement (expansion). This prevalence of inward deformation indicates that the surrounding rock mass, in most areas, responded in a stable manner under the NATM support, with the ground loading the support system rather than loosening. The largest convergence recorded was -36.4 mm at chainage 3+391, which occurred in a section of highly weathered granodiorite under one of the deepest cover sections of the tunnel. Despite this relatively larger inward movement, no structural instability was noted there; the deformation stabilized over time and was kept within the designed allowable range by the Class 3 support applied. Another area with notable deformation was around chainage 1+038 (approximately 1 km from the portal), where an outward displacement was observed. This coincides with the location of an over-break incident (as mentioned earlier), suggesting that the local loosening of the rock due to the over-break resulted in a minor dilation of the tunnel before the support was able to contain it. Such cases of outward movement were isolated and localized around disturbance events.

Excavation Classes and Support System Performance

Based on the geological assessments and monitoring results, the Lowari Tunnel excavation was divided into Classes 1, 2, or 3, which guided the installation of appropriate support systems (Supp. Table 1). The distribution of excavation classes along the tunnel roughly corresponded to the geological conditions encountered: Class 1 (good ground) was the most common, applied in the long stretches of competent granite, granodiorite, and other strong rock where spans were stable; Class 2 (moderate ground) was assigned to sections of intermediate quality rock or moderate jointing; and Class 3 (poor ground) was used in relatively short segments where the rock was highly fractured, faulted, or otherwise very weak. Each class had a predefined support configuration, which was adjusted slightly as needed based on field observations. In general, the support measures were as follows for each class:

- Class 1 (Stable/Good Rock)

Thin layer of shotcrete (≈5 cm) as diate lining, rock bolts (pattern bolting, typically 4-5 m long, spaced to stabilize key blocks), and wire mesh reinforcement. No lattice girders were required in Class 1 sections.

- Class 2 (Fair/Moderate Rock)

Medium support with shotcrete totaling ~10 cm (usually

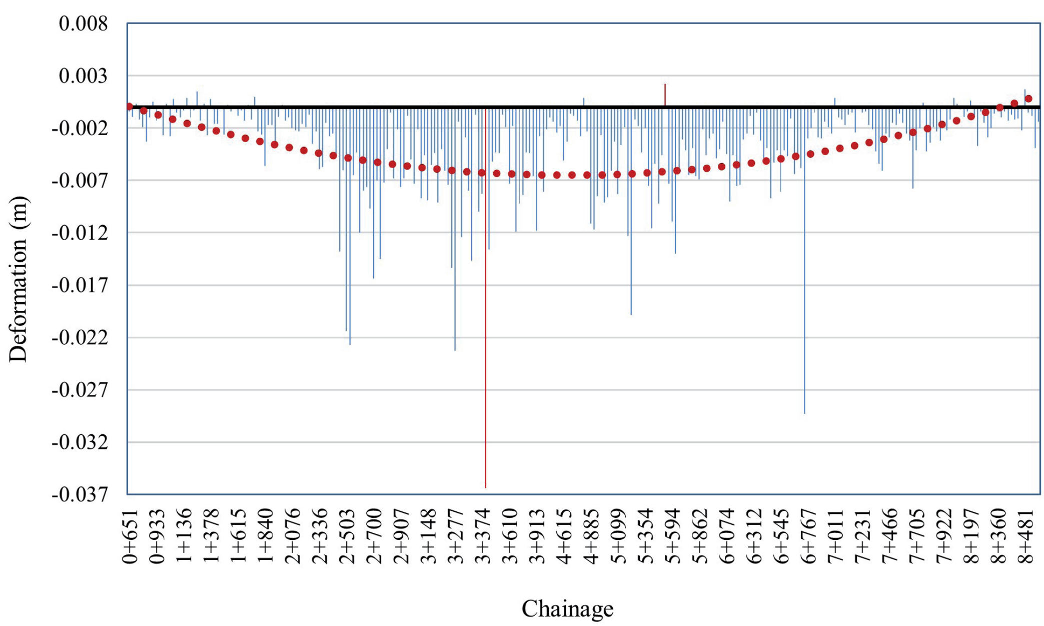

Fig. 7 - Lowari tunnel overall deformations observed at different chainages

applied in two layers of 5 cm each), systematic rock bolting (closer spacing than Class 1, to secure blocky ground), and wire mesh. Lattice girders were generally not used in Class 2 unless a specific need arose at a localized weak spot.

- Class 3 (Poor/Very Weak Rock)

Heavy support regime including a total of ~15 cm of shotcrete (applied in three layers for gradual build-up), dense pattern rock bolting (and the use of additional face bolts or forepoling if required in faulted ground), wire mesh, and steel lattice girders installed to provide a skeletal support frame. Class 3 was only designated for the most challenging sections and thus lattice girders (LG) were present only in these Class 3 excavations.

This support classification ensured that the support installation was commensurate with the rock conditions: for example, lattice girders and extra layers of shotcrete were only employed where absolutely necessary (poor ground), thereby optimizing the construction time and cost in better ground by avoiding over-supporting. Figure 1 reflects that the majority of the tunnel length was excavation Class 1, with Class 2 and 3 comprising progressively smaller proportions, aligning with the fact that truly poor ground was limited to a few discrete zones. The systematic use of shotcrete (SC), rock bolts (RB), and wire mesh (WM) across all classes indicates that these primary supports were the backbone of the NATM support system throughout the tunnel, while lattice girders were an auxiliary measure for the worst conditions.

In terms of support performance, extensive field testing and quality assurance measures were conducted to verify that the installed supports met design specifications. Shotcrete performance was monitored through compressive strength tests of core samples. Average compressive strength values for shotcrete at various chainages are summarized in Supp. Table 2. The early-strength tests (at 3 and 7 days) indicated that the shotcrete attained 11.5-12.9 MPa by 3 days and 17.2-18.6 MPa by 7 days, which demonstrated rapid strength gain crucial for initial ground support. By 28 days, the compressive strength stabilized in the range of 28.6-30.8 MPa, well above the typical minimum requirement (usually ~25 MPa for such applications), confirming that the shotcrete quality and curing were satisfactory. The strength results were fairly uniform along the tunnel with only minor variations; any slight differences (e.g., a section yielding 28.9 MPa vs another 30.8 MPa) could be attributed to local temperature/humidity conditions or slight mix adjustments, but all values exceeded the structural needs of the design. Rock bolt efficacy was confirmed through both quality control tests and pull-out tests. A comprehensive record of rock bolt installations and quality test results are presented in Supp. Tables 3 and 4, respectively. In laboratory tensile tests, all tested bolts (nominal 25 mm diameter steel) exceeded the minimum yield strength specified for the project and showed adequate elongation and bend resistance, indicating no material defects. On-site pull-out tests demonstrated that the anchorage of the bolts in the drilled holes was sound-every tested bolt achieved more than 80% of its yield load in pull-out capacity, satisfying the safety factor criteria (Supp. Table 5). For example, the commonly used SN rock bolts exhibited pull-out loads well above 165 kN, which provided confidence that the bolts would hold under the maximum anticipated rock pressure. The wire mesh installed was also tested for tensile strength and flexibility; results showed a wire yield strength on the order of 77000-78000 psi (approximately 530-540 MPa), surpassing the minimum requirement, and adequate ductility (elongation >12%) with no failures in bend tests (Supp. Tables 6 and 7). Lastly, the lattice girders were inspected visually and through occasional load tests after installation-all girders were found to meet the specified steel grades and curvature, and they fit well against the shotcrete lining, contributing to the overall support structure without any signs of distress during their service in the tunnel (Supp. Table 8).

Collectively, the monitoring and testing confirm that the support systems performed as intended. Importantly, the zones classified as excavation Class 3 - which had the most onerous combination of weak rock and high stress - were effectively stabilized by the heavy support installed, as evidenced by their deformation readings falling within allowable limits. Meanwhile, Class 1 and 2 zones were efficiently and safely supported with lighter support, which is reflected in their minimal deformations and lack of any significant instability. The clear correspondence between rock mass classification, deformation magnitude, and support requirements in the Lowari Tunnel provides a valuable case study: it shows that thorough geological characterization and a flexible NATM design allow for an optimal allocation of support resources, ensuring safety without unnecessary construction effort in good ground. It also highlights the success of the chosen support system - shotcrete, bolts, mesh, girders - in delivering a stable tunnel excavation across a spectrum of challenging ground conditions.

CONCLUSIONS

This study examined the critical factors influencing the stability and sustainability of a tunnel in a challenging Himalayan geologic environment, with a specific focus on the Lowari Tunnel project. Through comprehensive geological mapping, robust monitoring, and thorough support system evaluation, several key insights were gained regarding the interplay of ground conditions and tunnel performance:

- Detailed tunnel face mapping identified a wide range of rock types (from granite to metavolcanics), weathering states (predominantly fresh with some localized moderate weathering), and groundwater conditions (mostly dry with some dripping to flowing zones). The mapping effort successfully pinpointed potentially problematic areas - for instance, audible cracking sounds and minor spalling in the left crown at chainages ~0+925-0+936 alerted the team to rocks under high stress, and the presence of a slickensided fault with breccia at chainage ~1+002-1+008 was correlated with small fragment releases. These incidents underscore the importance of precise geological mapping and face observations in foreseeing stability issues before they escalate. By continuously evaluating the face conditions and adjusting excavation approaches, the project avoided major collapses and managed risks associated with complex geology.

- The extensive network of 269 deformation monitoring stations enabled a comprehensive assessment of tunnel stability in real time. The monitoring revealed that inward (convergent) deformation was the dominant response - about 93% of measured displacements were inward, indicating that the surrounding rock mass generally compressed onto the tunnel support rather than loosening outward. This is a favorable outcome, confirming the efficacy of the NATM strategy and support installation in keeping the tunnel in a state of compressive equilibrium. The range of convergence (0 to ~36 mm) observed across the tunnel aligned well with geological conditions: larger convergences occurred under higher overburden and in weaker, jointed rock (e.g., a -36.4 mm convergence in a highly weathered granodiorite zone), whereas near-surface and strong-rock sections saw only negligible movements. The rare instances of outward movement were traced to specific events like over-breaks, reinforcing that they were not systemic issues. Overall, the monitoring data indicate that the tunnel's deformation behavior remained within safe limits throughout construction, demonstrating the success of the design in accommodating the ground loads.
- The careful installation and rigorous testing of the tunnel support systems confirmed their effectiveness in maintaining stability under varied geological conditions. Shotcrete linings achieved their required strength and provided a continuous confinement to the rock surface. Rock bolts, wire mesh, and lattice girders all met or exceeded their design criteria in quality tests, and importantly, they functioned together as a system to prevent any collapse even in the most challenging sections. The result was that no significant support failures occurred, and the tunnel experienced no uncontrolled deformations. The performance of the support system highlights the value of adhering to quality control standards (e.g., proper bolt installation torque, shotcrete thickness verification) and adapting support intensity to excavation class. By matching support type and capacity to the ground class (with heavy support only in poor ground), the project ensured both safety and efficiency.

The Lowari Tunnel case demonstrates the effectiveness of NATM in challenging geological conditions, where continuous mapping and monitoring allow timely support adjustments. The study emphasizes thorough geological investigation and proactive measures like drainage and reinforcement. A well-designed combination of shotcrete, rock bolts, mesh, and girders ensure stability even in weak, tectonically active rock. Future research should assess these strategies in varied settings (rock types and conditions), construction methods, and their long-term and economic impacts.

ACKNOWLEDGMENTS

We are thankful to National Centre of Excellence in Geology, University of Peshawar, Pakistan for facilitation during carrying out this research.

SUPPLEMENTARY TABLES

Chainage(m)	Excavation Class	Installed Supports			
		Shotcrete	Rock Bolts	Wire Mesh	Lattice Girders
0 + 000 ~ 1 + 000	1 ~ 3	√	√	√	√
1 + 000 ~ 2 + 000	1 ~ 3	√	√	√	√
2 + 000 ~ 3 + 000	1 ~ 3	√	√	√	√
3 + 000 ~ 4 + 000	1 ~ 3	√	√	√	√
4 + 000 ~ 5 + 000	1 ~ 3	√	√	√	√
5 + 000 ~ 6 + 000	2	√	√	√	-
6 + 000 ~ 7 + 000	1 ~ 3	√	√	√	√
7 + 000 ~ 8 + 509	1 ~ 3	√	√	√	√

Supp. Tab. 1 - Installed supports system Lowari tunnel chainages

Chainage(m)	Strength (MPa)					
	3 Days		7 Days		28 Days	
	Maximum	Minimum	Maximum	Minimum	Maximum	Minimum
0 + 000 ~ 1 + 000	12.3	12.0	18.1	18.4	30.2	29.9
1 + 000 ~ 2 + 000	12.8	11.7	18.5	17.2	30.7	29.5
2 + 000 ~ 3 + 000	12.9	11.5	18.6	17.2	30.8	29.4
3 + 000 ~ 4 + 000	12.3	11.5	18.3	17.6	30.2	29.5
4 + 000 ~ 5 + 000	12.9	11.7	18.4	17.4	30.8	29.6
5 + 000 ~ 6 + 000	12.0	11.5	17.8	17.3	30.6	29.4
6 + 000 ~ 7 + 000	12.5	11.5	18.3	17.2	30.6	29.3
7 + 000 ~ 8 + 509	12.8	11.6	18.6	17.2	30.4	28.6

Supp. Tab. 2 - Average compressive strength of the shotcrete in Lowari tunnel

CHAINAGE	EC	Type	No.	(m)	CHAINAGE	EC	Type	No.	Length
0+000 to 0+665	1	SN	5×4	4	2+397 to 2+428.5	1	SN	5×4	4
0+665 to 0+732	2	SN	5×4	4	2+428.5 to 2+473.5	2	SN	11×10	4
0+732 to 0+816	1	SN	5×4	4	2+473.5 to 2+515.1	3	SD	13×12	4
0+816 to 0+839	2	SN	5×4	4	2+515.1 to 2+689	2	SN	11×10	4
0+839 to 1+056.5	1	SN	5×4	4	2+689 to 2+699	1	SN	4×5	4
1+056.5 to 1+076	2	SN	8×7	4	2+699 to 2+751.4	3	SD	10×11	4
1+076 to 1+164	1	SN	5×4	4	2+751.4 to 2+939	2	SN	11×10	4
1+164 to 1+176	2	SN	5×4	4	2+939 to 2+952	3	SD	11×10	6
1+176 to 1+241	1	SN	5×4	4	2+952 to 3+040	2	SN	11×10	4
1+241 to 1+248	2	SN	6×5	4	3+040 to 3+103	3	SD	11×10	6
1+248 to 1+303	1	SN	4×3	4	3+103 to 3+121.5	2	SN	11×10	4
1+303 to 1+316.3	2	SN	5×4	4	3+121.5 to 3+156.5	3	SD	11×10	6
1+316.3 to 1+320.2	3	SD	8×7	4	3+156.5 to 3+217.5	2	SN	11×10	4
1+320.2 to 1+326	2	SN	6×5	4	3+217.5 to 3+332	3	SD	10×9	6
1+326 to 1+407.5	1	SN	4×3	4	3+332 to 3+483.5	2	SN	9×8	4
1+407.5 to 1+425	2	SN	5×4	4	3+483.5 to 3+489.5	3	SD	11×10	6
1+425 to 1+437	1	SN	5×4	4	3+489.5 to 4+459	2	SN	10×11	4
1+437 to 1+461	2	SN	6×5	4	4+459 to 4+581.5	1	SN	5×4	4
1+461 to 1+494	1	SN	5×4	4	4+581.5 to 4+935.5	2	SN	10×9	4
1+494 to 1+504	2	SN	5×6	4	4+935.5 to 4+952.4	3	SD	10×9	6
1+504 to 1+593	1	SN	5×4	4	4+952.4 to 6+065.5	2	SN	10×9	4
1+593 to 1+600.5	2	SN	7×6	4	6+065.5 to 6+139	3	SD	11×10	6
1+601 to 1+620	1	SN	5×4	4	6+139 to 6+172	2	SN	10×9	4
1+620 to 1+626.5	2	SN	7×8	4	6+172 to 6+369	3	SD	11×10	6
1+626.5 to 1+717.3	1	SN	5×4	4	6+369 to 6+488	2	SN	11×10	4
1+717.3 to 1+722.1	3	SD	10×11	4	6+488 to 6+601.6	3	SD	10×9	4
1+722.1 to 1+742	2	SN	10×11	4	6+601.6 to 6+616	2	SN	6×7	4
1+742 to 1+748	1	SN	6×5	4	6+616 to 6+624	3	SD	9×8	4
1+748 to 1+762.5	2	SN	8×7	4	6+624 to 6+774.5	2	SN	11×10	4
1+762.5 to 1+801	1	SN	6×5	4	6+774.5 to 6+868.9	3	SD	9×8	4
1+801 to 1+824.5	2	SN/Swellex	9×8	4	6+868.9 to 6+873	2	SN	11×10	4
1+824.5 to 1+837.5	1	SN/Swellex	5×4	4	6+873 to 7+ 048	3	SD	9×8	4
1+837.5 to 1+918.5	2	SN/Swellex	8×7	4	7+048 to 7+059.5	2	SN	8×7	4
1+918.5 to 1+961.5	1	SN/Swellex	6×5	4	7+059.5 to 7+089.5	1	SN	6×5	4
1+961.5 to 1+983	2	SN/Swellex	10×11	4	7+089.5 to 7+129	2	SN	5×6	4
1+983 to 2+023	1	SN/Swellex	5×4	4	7+129 to 7+223.5	1	SN	5×6	4
2+023 to 2+039	2	SN/Swellex	5×4	4	7+223.5 to 7+257.5	2	SN	10×9	4
2+039 to 2+047.5	1	SN/Swellex	8×7	4	7+257.5 to 7+399	1	SN	6×5	4
2+047.5 to 2+064	2	SN/Swellex	6×5	4	7+399 to 7+405	3	SD	12×11	4
2+064 to 2+068	3	SN/Swellex	12×11	4	7+405 to 7+418	2	SN	10×11	4
2+068 to 2+097	2	SN/Swellex	10×9	4	7+418 to 7+740	1	SN	6×5	4
2+097 to 2+126	1	SN/Swellex	5×6	4	7+740 to 7+807.5	2	SN	6×5	4
2+126 to 2+140	2	SN/Swellex	10×11	4	7+807.5 to 7+811	3	SD	8×7	4
2+140 to 2+163	1	SN/Swellex	5×6	4	7+811 to 7+814.8	2	SN	8×7	4
2+163 to 2+168	2	SN/Swellex	8×7	4	7+814.8 to 7+858.5	3	SD	8×7	4
2+168 to 2+249	1	SN/Swellex	6×5	4	7+858.5 to 7+947	1	SN	6×7	4
2+249 to 2+267	2	SN	8×7	4	7+947 to 7+972	2	SN	5×4	4
2+267 to 2+281	1	SN	6×5	4	7+972 to 7+994	1	SN	6×7	4
2+281 to 2+326	2	SN	8×7	4	7+994 to 8+491	2	SN	5×4	4
2+326 to 2+358.5	1	SN	4×5	4	8+491 to 8+509	3	SD	9×8	4
2+358.5 to 2+397	2	SN	9×8	4					

SN = Store-Norfors SD = Self Drilling

Supp. Tab. 3 - Details of the rock bolts, installed in Lowari tunnel

Nominal diameter (mm)	Yield Strength (psi)		Tensile Strength (psi)		% Elongation		Effective Diameter (mm)	Weight (lb/ft)	Bend test
	Achieved	Min Required	Achieved	Min Required	Achieved	Min Required			
25 mm	79046		101367		16.63		25.240	2.64	OK
25 mm	74958	60000	98724	9000	18.11	12	25.234	2.64	OK
25 mm	79061		101277		16.14		25.283	2.65	OK

Supp. Tab. 4 - Installed rock bolt quality tests

Rock bolt type:		SN	Yield Load:	203 kN
Testing Method:		Hydraulic Jack	Specified Load:	162.4 kN
Specification:		ISRM Doc. 2, Part 1		
S.NO	Testing Date	Chainage(m)	Applied Load (kN)	Remarks
1	28/3/2013	0+511.5	165	Applied load is 165 kN.(R/S Wall)
2	28/3/2013	0+511.5	165	Applied load is 165 kN.(R/S Wall)
3	28/3/2013	0+514	165	Applied load is 170 kN.(R/S Wall)
4	28/3/2013	0+514	165	Applied load is 165 kN.(R/S Wall)
5	28/3/2013	0+516.5	165	Applied load is 170 kN.(R/S Wall)

Supp. Tab. 5 - Installed rock bolt pull-out test

Yield Strength (psi)		Tensile Strength (psi)		Elongation %		Bend test
Achieved	Min Required	Achieved	Min Required	Achieved	Min Required	
77595		110925		15		OK
78300	60000	111940	90000	15	12	OK
77947		111432		15		OK

Supp. Tab. 6 - Quality test of the installed wire mesh in Lowari tunnel

Chainage(m)	Excavation Class	Layers/Grade/Dimension
0 + 000 ~ 1 + 000	1 ~ 3	1/60/2.1×1.85 and 2/60/2.1×1.85
1 + 000 ~ 2 + 000	1 ~ 3	1/60/2.1×1.85 and 2/60/2.1×1.85
2 + 000 ~ 3 + 000	1 ~ 3	1/60/2.1×1.85 and 2/60/2.1×1.85
3 + 000 ~ 4 + 000	1 ~ 3	1/60/2.1×1.85 and 2/60/2.1×1.85
4 + 000 ~ 5 + 000	1 ~ 3	1/60/2.1×1.85 and 2/60/2.1×1.85
5 + 000 ~ 6 + 000	2	1/60/2.1×1.85
6 + 000 ~ 7 + 000	1 ~ 3	1/60/2.1×1.85 and 2/60/2.1×1.85
7 + 000 ~ 8 + 509	1 ~ 3	1/60/2.1×1.85 and 2/60/2.1×1.85

Supp. Tab. 7 - Wire mesh details in Lowari tunnel

CHAINAGE	QUALITY CONTROL				CHAINAGE	QUALITY CONTROL			
	YS	TS	EL	Bend		YS	TS	EL	Bend
1+316.3 to 1+320.2	78250	97813	15	OK	6+065 to 6+139	77980	97475	15	Ok
1+717.3 to 1+722.1	77523	96904	16	OK	6+172 to 6+369	78150	97688	17	Ok
2+064 to 2+068	77710	97138	14	OK	6+488 to 6+601.6	77710	97138	15	Ok
2+473.5 to 2+515.1	78320	97900	15	OK	6+616 to 6+624	78345	97931	17	Ok
2+699 to 2+751.4	78425	98031	15	OK	6+774.5 to 6+868.9	78000	97500	15	Ok
2+939 to 2+952	77980	97475	15	OK	6+873 to 7+ 048	78240	97800	15	Ok
3+040 to 3+103	77980	97475	16	OK	7+399 to 7+405	77980	97475	15	Ok
3+121.5 to 3+156.5	78945	98681	15	OK	7+807.5 to 7+811	78320	97900	16	Ok
3+217.5 to 3+332	78165	97706	15	OK	7+814.8 to 7+858	79830	99788	15	Ok
3+483.5 to 3+489.5	78400	98000	16	OK	8+491 to 8+509	78900	98625	15	Ok
4+935.5 to 4+952.4	77960	97450	15	OK					

YS = Yield Strength TS = Tensile Strength EL = Elongation %

Supp. Tab. 8 - Lattice girders quality control details in Lowari tunnel

REFERENCES

Aygar E.B. (2022) - *Evaluation of the relationship between the collapsed mechanism and excavation method in tunnels excavated in schists.* Scientific Reports, **12**: 21274. https://doi.org/10.1038/s41598-022-25767-1.

Bjørn N. (2011) - *Cases of instability caused by weakness zones in Norwegian tunnels.* Bulletin of Engineering Geology and the Environment, **70**: 7-13.

Chang X., Chai J., Liu Z., Qin Y. & Xu Z. (2020) - *Comparison of ventilation methods used during tunnel construction.* Engineering Applications of Computational Fluid Mechanics, **14**: 107-121.

Georgiou D., Kalos A. & Kavvadas M. (2022) - *3D numerical investigation of face stability in tunnels with unsupported face.* Geotechnical and Geological Engineering, **40**: 355-366. https://doi.org/10.1007/s10706-021-01900-1.

GEOCONSULT/TYPSA (2005) - *Bid documents*. II-A, 109-121.

Han K.H., Zhang C.P. & Zhang D.L. (2016) - *Upper-bound solutions for the face stability of a shield tunnel in multilayered cohesive-frictional soils.* Computers and Geotechnics, **79**: 1-9. https://doi.org/10.1016/j.compgeo.2016.05.018.

Huang L., Bohne R.A., Bruland A., Jakobsen P.D. & Lohne J. (2015) - *Environmental impact of drill and blast tunnelling: life cycle assessment.* Journal of Cleaner Production, **86**: 110-117.

Huang F., Ran B. & Pan X. (2014) - *Safety evaluation of illuminance transition at highway tunnel portals on basis of visual load.* Transportation Research Record, **2458**(1): 1-7. https://doi.org/10.3141/2458-01.

Khan J., Ahmed W., Yasir M., Islam I., Janjuhah H.T. & Kontakiotis G. (2022) - *Pollutants concentration during the construction and operation stages of a long tunnel: a case study of Lowari tunnel, (Dir–Chitral), Khyber Pakhtunkhwa, Pakistan.* Applied Sciences, **12**(12): 6170. https://doi.org/10.3390/app12126170.

Li S. J., Feng X.T., Li Z.H., Chen B.R., Zhang C.Q. & Zhou H. (2012) - *In situ monitoring of rockburst nucleation and evolution in the deeply buried tunnels of Jinping II hydropower station.* Engineering Geology, **137-138**(7): 85-96.

Luo Y., Chen J., Gao S., Deng X. & Diao P. (2017) - Stability analysis of super-large-section tunnel in loess ground considering water infiltration caused by irrigation. Environmental Earth Sciences, **76**: 763. https://doi.org/10.1007/s12665-017-7106-7.

Lunardi P. & Barla G. (2014) - *Full face excavation in difficult ground.* Geomechanics and Tunnelling, **7**: 461-468.

Ma E., Lai J., Xu S., Shi X., Zhang J. & Zhong Y. (2022) - *Failure analysis and treatments of a loess tunnel being constructed in ground fissure area.* Engineering Failure Analysis, **134**: 106034. https://doi.org/10.1016/j.engfailanal.2022.106034.

Marcher T., Erharter G.H. & Winkler M. (2020) - *Machine learning in tunnelling - capabilities and challenges.* Geomechanics and Tunnelling, **13**(2): 191-198.

Pan Q. J. & Dias D. (2016) - *The effect of pore water pressure on tunnel face stability.* International Journal for numerical and analytical methods in Geomechanics, **40**: 2123-2136. https://doi.org/10.1002/nag.2528.

Song W., Zhu Z., Pu S., Wan Y., Huo W. & Peng Y. (2022) - *Preparation and engineering properties of alkali-activated filling grouts for shield tunnel.* Construction Building Materials, **314**. https://doi.org/10.1016/j.conbuildmat.2021.125620.

Tahirkheli R.K. (1979) - *Geology of Kohistan, Karakoram, Himalaya, Northern.* Geological Bulletin of University of Peshawar, **11**:1-30.

Wang Y., Guo P., Lin H., Li X., Zhao Y., Yuan B., Liu Y. & Cao P. (2019) - *Numerical analysis of fiber-reinforced soils based on the equivalent additional stress concept.* International Journal of Geomechanics, **19**. https://doi.org/10.1061/(ASCE)GM.1943-5622.0001504.

Wang J., Zhou P., Song Z., Li S. & Zhang Q. (2022) - *A new calculation method for tunneling-caused stratum settlement.* KSCE Journal of Civil Engineering, **26**: 2624-2640. https://doi.org/10.1007/s12205-022-1258-z.

Waseem M., Lai C.G. & Spacone E. (2018) - *Seismic hazard assessment of northern Pakistan.* Natural Hazards, **90**(2): 563-600.

Zbigniew N., Małkowski P. & Majcherczyk T. (2018) - *Application of the NATM method in the road tunneling works in difficult geological conditions:the Carpathian flysch.* Tunnelling and Underground Space Technology, **74**: 41-59.

Zhang W., Li H., Han L., Chen L. & Wang. L. (2022) - *Slope stability prediction using ensemble learning techniques: a case study in Yunyang County, Chongqing, China.* Journal of Rock Mechanics and Geotechnical Engineering, **14**(4): 1089-1099. https://doi.org/10.1016/j.jrmge.2021.12.011.

Zhang C., Chen R., Wu H., Gao B. & Yang S. (2023) - *Research on the effect of double sealing gaskets arrangement on the mechanical behavior of the segment longitudinal joints with large diameter shield tunnel.* Tunnelling and Underground Space Technology, **141**: 105341. https://doi.org/10.1016/j.tust.2023.105341.

Received April 2025 - Accepted June 2025