Stress Induced Deformation in Phyllite Rock in Mahadevtar Tunnel of KTFT Road Project

Binayak Sharma*, Bimala Acharya**, Bharat Lal Shrestha***,
Ramesh Kumar Paikara#, Niroj Gautam##

Abstract

In the NATM Tunneling, the monitoring of the deformation of the surrounding rock is a basic means for confirmation consequently adaptation of the excavation stages as well as rock support measures. In the large tunnels, excavation and support works are carried out in different stages. Once the top heading portion of the tunnel was excavated, deformation leading to various types of problems on the primary support installed in the tunnel occurred at various spots in the approximately 329 m stretch of the right tube and 182 m stretch of the left tube in the Mahadevtar Tunnel of the Kathmandu Terai / Madhesh Fast Track (Expressway) Road Project (KTFT). Some of the problems noticed were longitudinal crack on shotcrete leading to spalling, steel ribs twisting, etc. The monitoring of the deformation was carried out and different correction measures were applied at site to ensure the long-term stability of the tunnel. For the sections yet to be excavated, the support measures were upgraded to cope with the encountered site situation. The numerical modeling was carried out considering original and modified rock support measures. The result showed that the support capacity curve for shotcrete is improved to a slight extent due to modified support measures. For the tunneling in the weak rocks, selection of method and sequence of excavation, time taken between different stages of excavation and support application, adjustment of support measures as per site condition, and monitoring of the deformation are very crucial factors for the successful completion of the project. Monitoring of the deformation plays a crucial role to adjust support measures if required and to verify the adequacy of the installed support. Deformation shall be mostly converged, prior to installation of secondary support.

Keywords: deformation, phyllite, convergence, monitoring, tunneling through weak geological zone, NATM tunneling

Introduction to the Project

Kathmandu-Terai/Madhesh Fast Track (Expressway) Road Project (KTFT) consists of a total road length of 70.977 Km out of which 10 Km is covered by six twin tunnels. Out of the six tunnels, three tunnels at Mahadevtar, Dhedre and Lanedada are under construction. The twin tunnels at Mahadevtar are the longest tunnels in the project with an average length of 3454m. During excavation, it is about 14.5 m wide and 10.5 m high. It incorporates 2 lanes plus a service lane with side drains, cable trenches, inspection galleries etc. During operation it provides a clear width of 11.5 m and clear height of 5 m for the road traffic. Depending upon the geological condition, an inverted arch is provided for weak rock. The twin tunnels are separated by a minimum of 1.5D, where D is the excavation's width. The ground surface elevation in the tunnel region lies between 976m to 1801m. The road surface design level lies between 972m and 978m.Maximum overburden is about 806.5m.

Background

After excavation about 630 m from exit portal various problems were noticed in the primary support in both tunnels. In some sections of the tunnels, longitudinal crack mainly in the crown

* Resident Engineer-Tunnel, KTFT Road Project, Yooshin JV. <binayaksharma@gmail.com.
** Geotechnical Engineer, KTFT Road Project, Nepali Army. <bimalaacharya@gmail.com>
*** Technical Colonel, Head of Planning and Design Division at KTFT Road Project, Nepali Army. <shrestha.bharatlal@gmail.com>
# Technical Colonel, Deputy Head of Planning and Design Division at KTFT Road Project, Nepali Army. <paikara_rk@hotmail.com>
Technical Major, Lendanda Camp Commander (Civil Engineer) at KTFT Road Project, Nepali Army. <nirojgautam2006uet@gmail.com>

part was noticed after excavation of the top heading section. Further, spalling of shotcrete, twisting of some of the installed ribs were noticed along the longitudinal crack.

As soon as the problems were noticed at site, immediate measures such as but not limited to rock support enhancement, improvement in rock mass quality around the tunnel, update on design support with numerical analysis and monitoring etc. were implemented.

Major topics included are geology, problem identification, correction/ rectification measures, numerical analysis, rock support adaptation etc.

Geology of the Area

The project area lies in the Lesser Himalayan rock sequence which is in between the Sub-Himalayas and Higher Himalayas separated by the Main Boundary Thrust (MBT) and the Main Central Thrust (MCT) respectively. The Lesser Himalayas is made up mostly of sedimentary and meta-sedimentary rocks; such as shale, sandstone, conglomerate, slate, phyllite, schist, quartzite, limestone and dolomite.

Geologically the Mahadevtar tunnel lies in the Kathmandu Complex (in North) and Nawakot Complex (in South) of the central Nepal Himalaya separated by the Mahabharat Thrust (MT). The Kathmandu Complex is comprised of metamorphic rocks such as quartzite, schist, marble etc. The Nawakot Complex consists of Metasedimentary to low grade metamorphic rocks like limestone, dolomite, metasandstone, slate, phyllite, quartzite etc. Figure 2 shows the geological profile after the investigation result and Figure 3 shows Magnetotellurics MT survey results.

Around 60% of the length of the tunnel has been already excavated where the geological condition has been precisely mapped. The central part of the alignment remaining to excavate shall be more governed by the marble according to the findings of the investigation. Distribution of rock mass along the tunnel is presented in Figure 1. It incorporates the rock classification during excavation works. Schicht rock is prevailing at the entrance followed by Marble, bands of phyllite dolomite whereas at the exit side slate and phyllite are present.

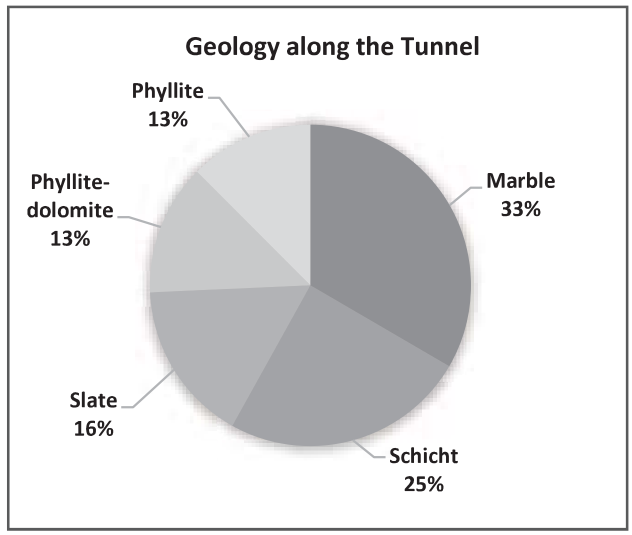

Figure 1:Geology along the Mahadevtar Tunnel

The problematic area (between Chainage 29+211 to 29+540 of the Right Tube and 29+230 to 29+412 of the Left Tube) lies in the phyllite rock section. The section is adjacent to MT thrust at the north side and is also about 800 m close to MBT to the south. Due to the presence of two thrusts at both sides, the rock strata are highly deformed fractured, with prominent small-scale folding and shear zones.

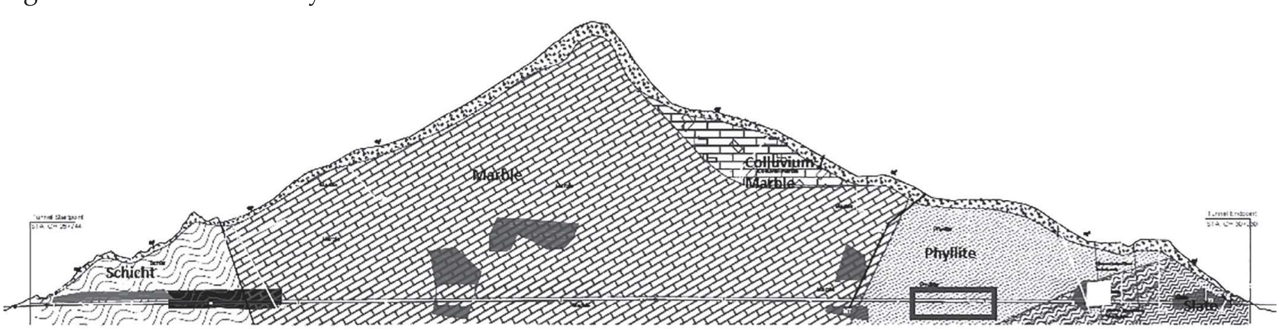

Figure 2: Geological profile along Mahadevtar tunnel, problematic area marked in red, pink patches represent water rich strata (Source: China State, 2021)

The exposed phyllite rock mass was light gray, fine grained, thinly bedded, moderately strong and consists of 3 joint sets. Joint surfaces are undulating and slightly rough. Groundwater condition is dry during construction and partially wet after several weeks.

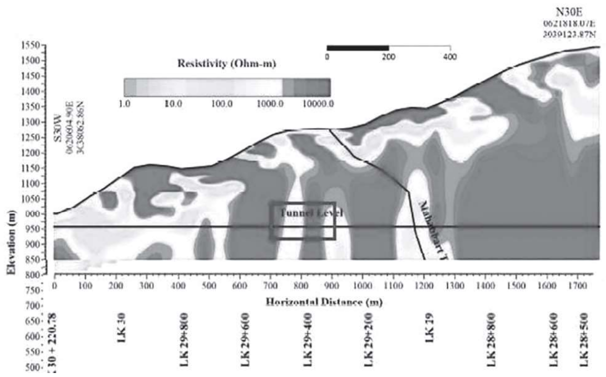

Figure 3: MT survey result- Resistivity diagram -the problematic area is marked in blue (Source: China State, 2022)

The investigation result shows that the bedrock consisting of thinly banded gneiss with quartz veins is present at the depth of 21m from the ground surface. After the depth of 42m from ground surface phyllite rock is present, this was also encountered while tunnel excavation works. In those area the maximum overburden is about 300m and ground water table is present at a depth of 43 m from ground surface. Water rich strata were not identified in the problematic section at the tunnel level during the geophysical investigation. During excavation works the exposure was dry but after several weeks small patches of damp area were noticed locally. Figure 4 shows the Phyllite rock mass exposed after excavation.

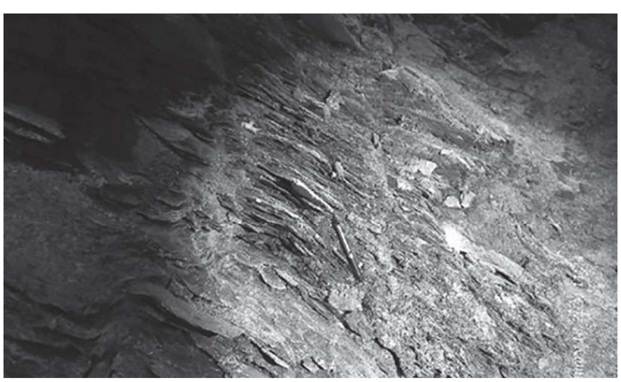

Figure 4: Phyllite Rock mass exposed after excavation

Rock Support Measure

The RMR of the rock mass at the problem area ranges between 21 to 30 which refer to the support class type P-4-2. The original rock support system consists of primary support consisting of 22 mm dia. rock bolts of length 3.5m@ 80 X 100 cm (longitudinal and circumferential) spacing in staggered layout, 8mm dia 25 X 25 cm wire mesh, MB175 steel rib @ 80 cm spacing, 23.5 cm thick M25 shotcrete. As secondary support 40cm thick M30 plain concrete is provided. The invert arch is filled with M15 concrete. The details of the tunnel cross section including support measures is shown in Figure 5.

In the revised rock support system, which is discussed in detail in the following headings, the size of the steel rib was changed from MB175 to MB200, the shotcrete thickness was changed from 23.5 cm to 26 cm and wire mesh opening size was changed from 25 x 25 cm to 20 X 20 cm.

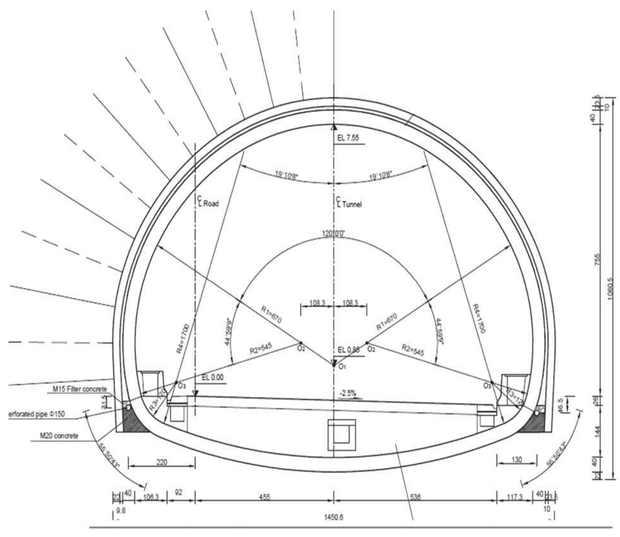

Figure 5:Tunnel cross section with support measures for support class P4-2(Source: China State, 2021)

Problems encountered during construction

After entering about 130 m into the phyllite rock, some hair cracks on the shotcrete ranging from 2-5 m long sections along the crown part of the right tube were noticed on August 2023 after the excavation of top heading. Subsequently, with the further excavation ahead, slight twisting of 5 numbers of neighboring steel ribs (MB175) were noticed along the longitudinal crack at the crown area.

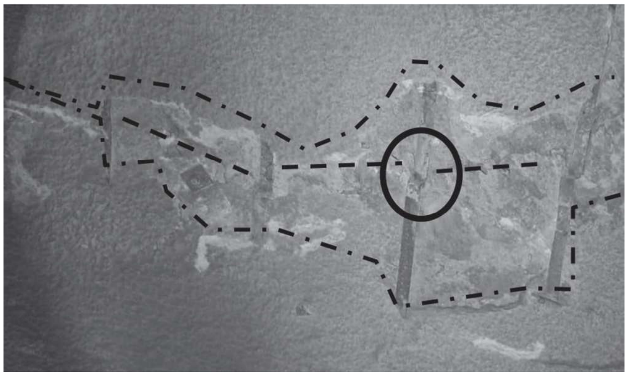

Figure 6: Shotcrete spalling (dotted area in red) and steel rib twisting (red circle) along the longitudinal crack (blue dotted line)

Spalling of shotcrete was observed forming patches at different locations over time due to increased deformation.

While excavating the bench part and invert section, deformation was found to be slightly increased and consequently the problems also increased.

Figure 7 shows punching of the footplate of steel rib of top heading part indicating significant vertical stress.

Figure 7: Foot plate of steel rib heading part found punched while excavation of bench

At the tunnel side walls, windows need to be maintained for the embedded parts as well as openings for various components like cross tunnel. In such an area, steel ribs were installed without shotcrete. Such steel ribs were found buckled in some locations (Figure 8). This phenomenon also indicated significant stress conditions of the steel ribs.

Figure 8: Buckling of steel ribs without shotcrete

In about 20m long section, significant deformation (10-12cm) is noticed along the right crown part of the tunnel as presented in Figure 9.

Figure 9: deformation at right crown part

Cause analysis

As per Chen et al (2013) big deformation in tunneling can be classified into four categories as: squeezing deformation, swelling deformation, losing deformation and soft rock deformation under high in-situ stress.

Phyllite is normally known as weak and soft rock mass. Deformation of the rock mass is thought to occur because of the presence of soft rock under significant overburden pressure. Further the next potential cause of the problems is due to the presence of weak rock in the stress condition in the region between two thrust zones. Increased deformation is further contributed by the tunnel's large size. During excavation tunnel face is dry, after several weeks, some damp patches are visible randomly. Therefore, water pressure is not considered as the root cause.

While excavation of the tunnel ahead, excavation of bench and invert section as well as excavation of next tunnel tube running parallel, some impact will be noticed which will increase deformation. The construction of large twin tunnels in the weak rock section is a challenging job.

Correction measures

After the problems were noticed, implementation of correction measures was started immediately. Type and amount of measures were based on the seriousness of the issues. The correction measures are subdivided mainly in following two categories:

1. Additional stabilization measures for already excavated section and
2. Revision of support measure and adoption of excavation, support sequence for the section to excavate ahead. These measures are discussed in detail in the following.

Measures implemented in the excavated sections

When the problem was noticed, only the top heading part of the tunnel was excavated and the tunnel face was about 30 to 50m ahead. Bench and invert sections were still to be excavated. Various support measures were added for the enhancement of the already installed rock support system.

In the crack area, and sections with increased deformation cement grouting with W/C ratio 0.8 to 1.0was carried out at both sides of the crack area for the improvement of rock mass quality (Figure 10). Grouting is performed via perforated steel pipe dia. 42mm and 4.0m long at longitudinal spacing of about 0.8 to 1 m. The grouting pipe will subsequently act as a rock bolt while it remains at site.

The principle behind it is to provide bonding in the rock mass in the plastic zone and fill the gap due to detachment between the rock layers if any. It will ultimately increase the shear strength of rock mass and enhance the arching effect of the tunnel.

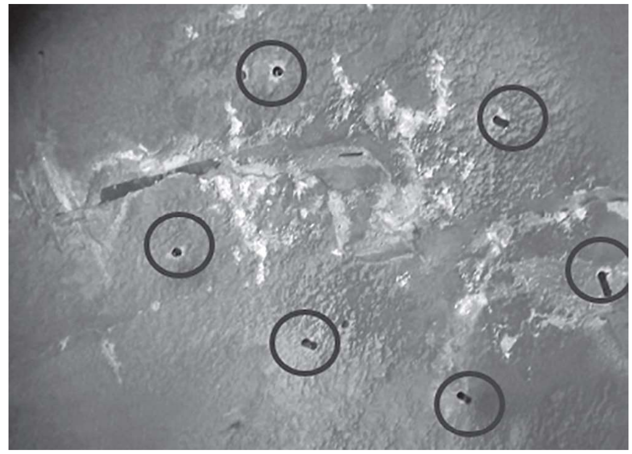

Figure 10: Grouting with hollow perforated pipe along the crack

For the section with increased deformation, grouted foot locks (perforated hollow steel pipes dia. 42mm and length 4 m) were installed at both sides of the steel ribs and connected to the steel ribs as shown in Figure 12.

Figure 11: Foot locks for the steel ribs strengthening

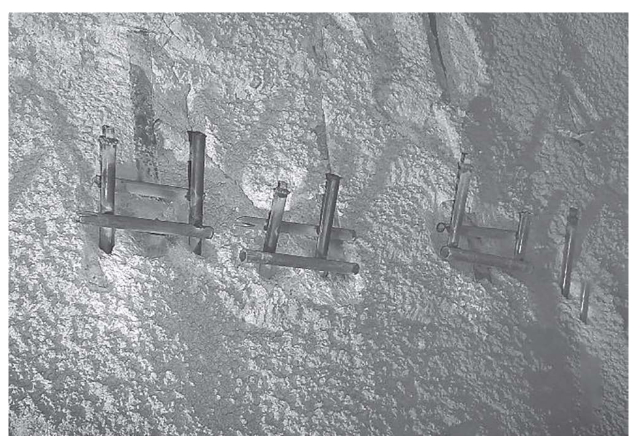

Figure 12: Foot locks for the steel ribs strengthening- a close view

For the section with twisted steel ribs, grouting is also performed along the crack area.

Additionally, to ensure the stability of the tunnel, additional steel ribs were installed at the inner profile of the existing twisted steel ribs.

Figure 13: Additional steel ribs installed inside the twisted steel ribs

Once the deformation is converged, the deformed tunnel profile (as shown in Figure 9) was corrected. For this shotcrete as well as steel ribs of the bulged section were removed and replaced locally.

For the section where tunnel profile is acceptable and only steel ribs are deformed as shown in Figure 6, only the deformed section of the steel ribs and shotcrete were replaced.

For the section where double steel ribs are installed, the full profile of the steel ribs is replaced with larger steel ribs i.e. 2xMB175 is replaced with single MB200 steel ribs. Taking out the installed support is a very challenging and risky job and needs fine judgment and supervision of highly experienced engineers. Even a minor mistake may lead to the collapse of the tunnel.

The reason behind the replacement is that, twisted inner rib is theoretically not taking any load and the inner added ribs lie in the profile of the secondary lining i.e. the thickness of secondary lining shall be otherwise compromised.

Measures taken in the section to excavate

Based on the experience gained from the problems encountered in the already excavated section of the tunnels, various adjustments were made not only in the method of excavation but also in the support measures as follows:

Revision of rock support and adaptation of excavation and support sequence:

Round length of excavation is reduced from 2.4 m to 0.8 ~ 1.6 m as per site condition. For blasting specific charge is reduced to 0.4 kg/m3 in the top heading section and around 0.17 kg/m3 in the benching section. For the further reduction of blasting impact, explosive per delay detonator is reduced. Whenever possible, blasting is avoided and mechanical excavation is done. Significant distance (both in time as well as length) is maintained between the excavation of the top heading and benching so that the deformation of the top heading part gets almost converged while excavation of the bench part begins.

Revised Rock Support:

The rock support was increased in comparison to the rock support specified in the design drawing. As mentioned under heading 'rock support measures', the revision was made on the size of the steel rib, the thickness of shotcrete and the opening size of the wire mesh.

Continuous monitoring and add Support

In the NATM tunneling, the measured deformation result play vital role to confirm the adequacy of the installed support as well as to adapt the support measure as per encountered site condition. The tunnel can only be stable, once the deformation is mostly converged prior to construction of secondary lining.

As a general principle, allowing deformation from the rock mass results in a lesser amount of rock support. Jinyan Fan et.al (2020) states that an appropriate amount of deformation shall be allowed to release deformation energy from the surrounding rock.

Figure 14: Optical Targets for Convergence Monitoring of the Tunnel

In the problem area of both the tunnels, sufficient delay period is considered for the observation and implementation of correction measures between excavation of top-heading to benching and invert excavation. Further, optical targets are installed in close intervals, normally 5 m for convergence monitoring with 5 to 7 points per section (Figure 7). Not only after the excavation, but the monitoring is continued to monitor the effect after the rectification of steel ribs or shotcrete. The monitoring points and results for Ch 29+299 are shown in Figure 15 and Figure 16 respectively. The total displacement of 2.5 cm has almost converged after about 10 days. Field observation has shown deformation up to 10-12 cm locally in some areas.

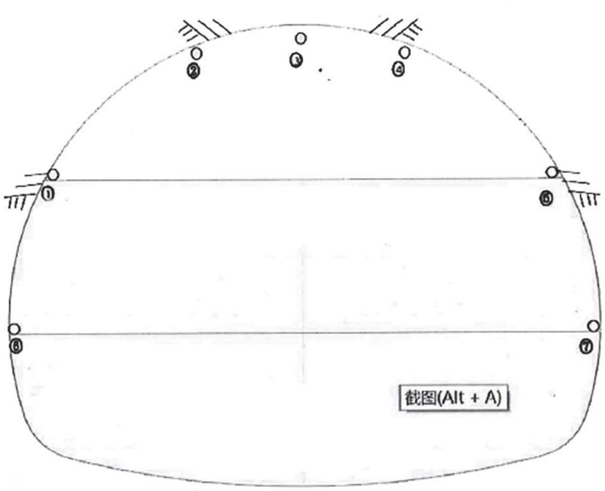

Figure 15: Monitoring points in Ch. 29+299

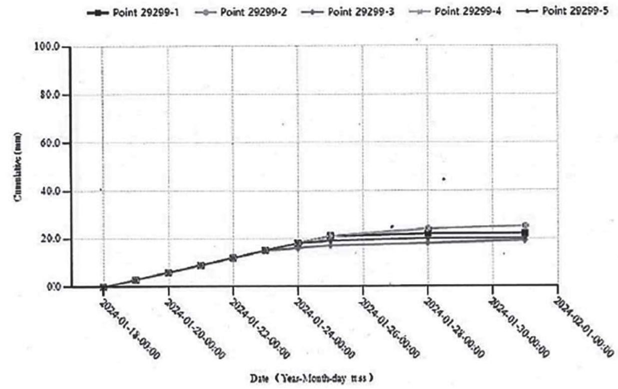

Figure 16: Monitoring data of the Tunnel at Ch. 29+299

Consideration for Secondary Lining

The design of the tunnel considers, in-situ concrete lining as secondary support for the tunnel. While the lining may generally remain unreinforced, structural design considerations and project design criteria will dictate the need for and amount of reinforcement (FHWA, 2009). The thickness of M30 concrete lining varies from 30 cm to 60 cm for the Mahadevtar tunnel based on type of rock class. The problem area is under lining type P-4-2 which refers to 40 cm thick M30 plain concrete lining with inverted arch construction.

Although the secondary lining can bear significant load from the surrounding rock, the primary support is designed as the main load bearing structure to bear the pressure from the surrounding rock for this tunnel. So that the secondary lining shall not bear the excess stress from the surrounding rock, it shall not be constructed too early. Time shall be given so that the deformations are converged first. Otherwise, stress cracks and heaving of invert etc. can occur. Repair of the secondary lining will be a tedious job. Convergence of deformation shall be confirmed prior to casting of the secondary lining. Guo et al. (2024) suggests secondary lining shall be applied after reaching a deformation rate corresponding to 95% of the deformation of the first stable stage.

Convergence of deformation needs several weeks to months in the rock mass like weak phyllite. Sequential excavation i.e. top heading followed by benching and inverted arch helps to maintain time as well as distance between the different sequences of tunnel excavation. Secondary reinforcement is going to be installed in the problematic areas after time elapses of up to 6 months after the start of excavation in the respective section.

For the Concrete lining of the affected section of the tunnel following approaches are being implemented:

1. Inverted arch construction is meaningful for better transfer of the load to the underlying rock strata. If constructed too early, heaving of the inverted arch might occur. In the Project, an inverted arch is constructed after about 1.5 to 2 months of upper section excavation and no any upheaving symptoms are noticed.
2. Reinforced Concrete Secondary Lining: The severely impacted stretches with multiple twisted steel ribs are upgraded to reinforced concrete lining for the additional safety. In Right tunnel about 140 m and in left tunnel about 80 m long section will be upgraded to reinforced concrete.
3. Plain concrete Secondary Lining: For the section with minor problems such as longitudinal crack but without twisting of the steel ribs, the secondary lining remained plain concrete as specified in the initial design.
4. In order to avoid water pressure on the concrete lining, a layer of geotextile along with circumferential perforated pipe as well as waterproof membrane will be laid between the primary and secondary lining. A photograph of the section with secondary lining is shown in Figure 17.

Figure 17: Secondary Lining

Numerical Modelling:

The numerical modeling was carried out firstly with the original support measure and then with the revised support measures. For this two-dimensional finite element program Phase 2 version 8 is used. The generalized Hoek Brown material model is considered for rock mass. The rock mass parameter is derived from laboratory test results and as pre encountered site geological conditions as shown in the following table.

Table 1: Rock mass parameters for Phyllite

Unit weight KNm3	Intact UCS (MPa)	GSI	mi	Youngs modulus intact rock (Ei) GPa	Pois-son's ratio
26	12	37	7	20	0.2

Disturbance factor of 0 is selected for the analysis with the assumption of good quality-controlled blasting as such there will be minimal disturbance to the rock mass surrounding the tunnel.

For the problematic section the maximum overburden is 300m which is taken into account for numerical modeling. The total gravity stress ratio (H/V) of 0.43 is considered. The Locked horizontal stress in plane 1.96 MPa and out of plane 2.9 MPa is considered.

Parameters for shotcrete and rock bolts for numerical modeling are as follows.

Table 2: Parameters for shotcrete, and rock bolt for finite element analysis

Shotcrete		Rock bolt	
	M25		
Young's modulus (MPa)	25000	Bolt Modulus (MPa)	200000
Poisson's Ratio	0.15	Tensile capacity (MN)	0.15
Compressive Strength (MPa)	25	Residual tensile capacity (MN)	0.1
Tensile strength (MPa)	3.5		
Unit weight (MN/m3)	0.024		

In the first model, construction stage with excavation and primary support in the top heading part is considered. Firstly, the analysis is carried out with original support parameters and then it is repeated with revised parameters. Considering the original support parameters, it was found that the total displacement at invert and sides reached 11 cm after application of support (Figure 18) which reflects the site scenario. The support capacity curve for shotcrete is shown in the Figure 19.

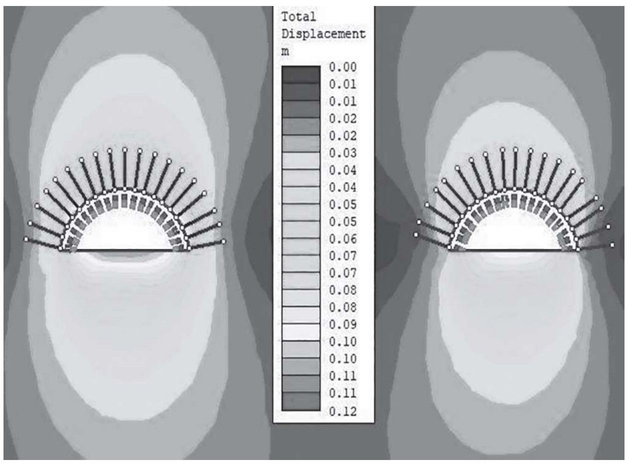

Figure 18: Total displacement contour after installation of support (stage: excavation and application of original support in the top heading part)

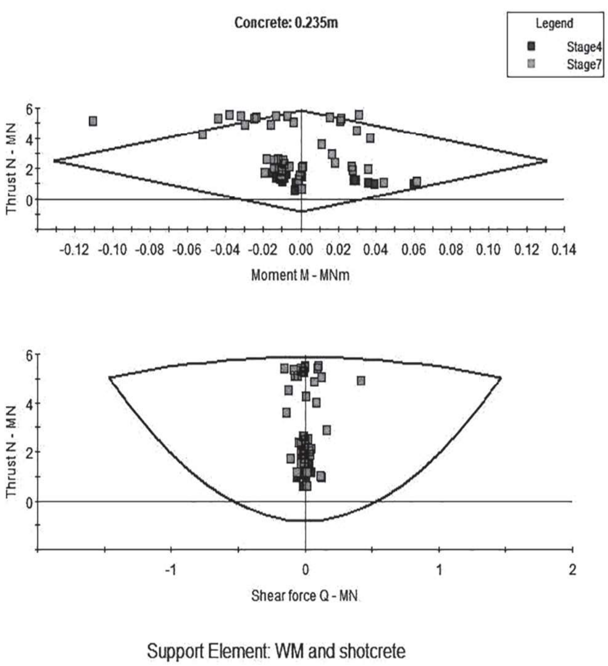

Figure 19: Support capacity plots for shotcrete (stage: excavation and application of original support in the top heading part)

The support capacity curve shows a graph between thrust and moment and between thrust and shear force with an envelope representing required factor of safety. The support capacity curves for shotcrete show the factor of safety for several points are marginal to the border and few points are even beyond the envelope for required factor of safety. The scenario can be correlated to the longitudinal crack on shotcrete at site. The support capacity curves for steel ribs and wire mesh are well within the factor of safety which does not exactly match site situations where twisting of steel ribs is noticed.

With the revised support parameter, the displacement behavior was not significantly different, but the support capacity curve for shotcrete is improved to a slight extent as shown in Figure 20.

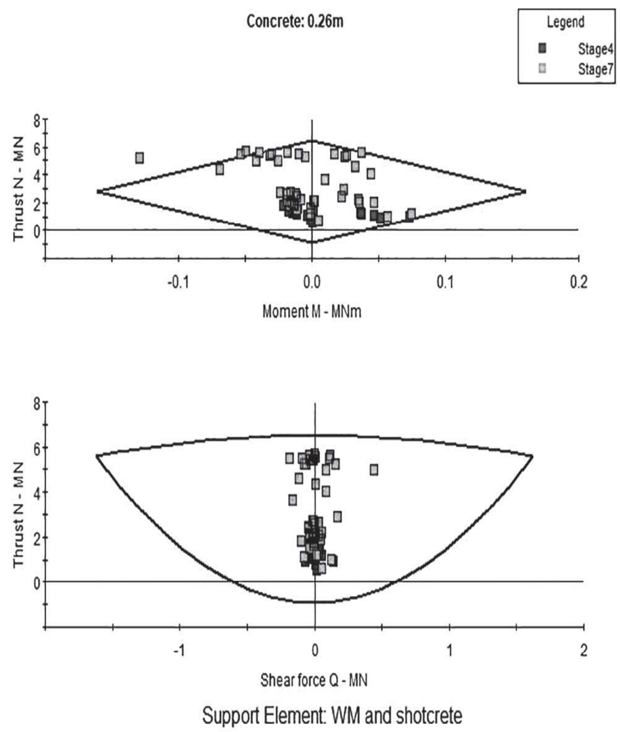

Figure 20: Support capacity plots for shotcrete (stage: excavation and application of revised support in the top heading part)

In the second model, full face excavation and rock support measures as per original support parameter and revised parameters are considered. The secondary lining is not considered for the top heading and benching part because it serves as an additional safety measure.

Firstly, with the original support parameters the maximum displacement is 5cm in the invert level and 4 cm in the crown as shown in Figure 21 which is comparatively reduced than in the first model.

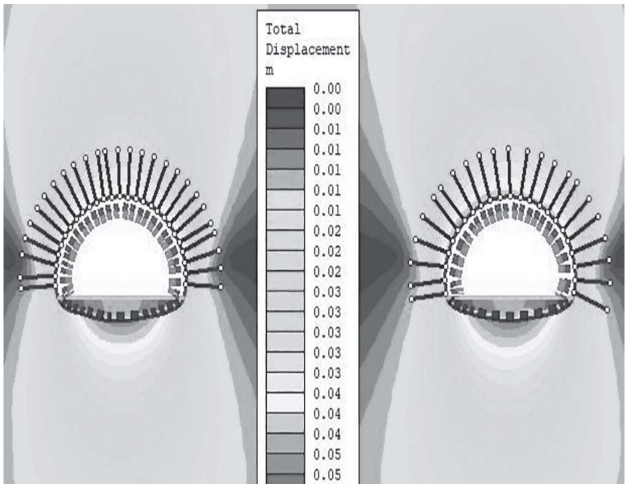

Figure 21: Total displacement contour after installation of support (stage: full face excavation and application of original support)

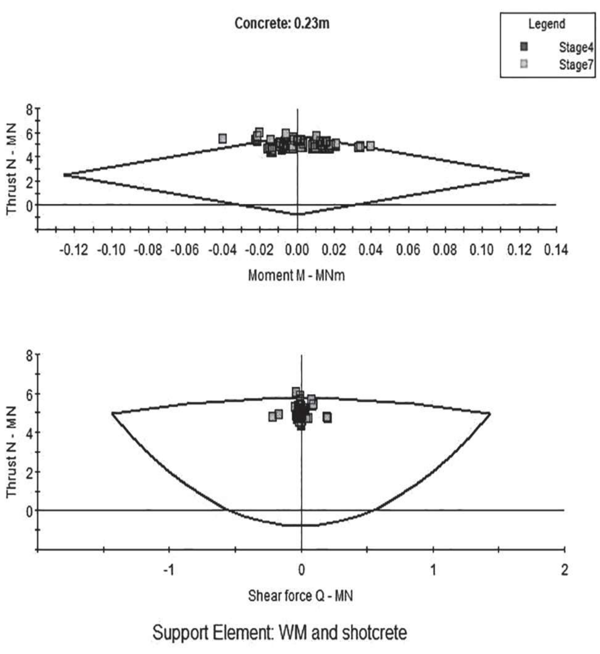

Figure 22: Support capacity plots for shotcrete (stage: Full face excavation and application of original support)

With the revised support parameter, the displacement behavior is not significantly different, but the support capacity curve for shotcrete is improved to a slight extent as shown in Figure 23.

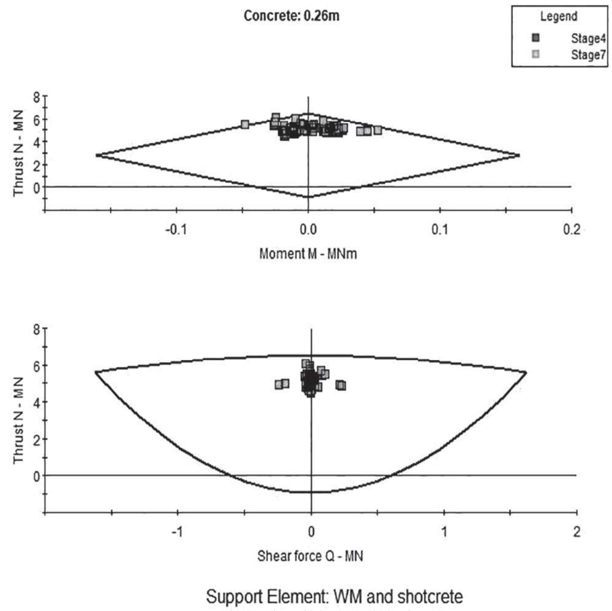

Figure 23: Support capacity plots for shotcrete (stage: Full face excavation and application of revised support)

Conclusion

The analysis of the deformation encountered during the excavation of the Mahadevtar tunnel has revealed the significance of proper support measures and monitoring in ensuring the stability and safety of the structure. The observed deformation, including shotcrete spalling, steel ribs twisting, and crack formation, necessitated the implementation of various correction measures to improve the rock mass strength and add additional rock supports.

The numerical modeling conducted for the top heading part of the tunnel showed that the maximum displacement in the invert and side wall was approximately 11 cm, and the support capacity curve for the shotcrete was marginally on the borderline, with a few points slightly beyond the line. This reflected the site scenario and the need for further improvements.

When the numerical modeling was repeated for the complete tunnel profile, including the invert and primary support, the results showed better performance compared to the previous case. This demonstrates the significance of incorporating an inverted arch in the tunnel design, as it enhances the overall stability and performance of the structure.

The revised support measures, as determined by the numerical modeling, showed a considerable improvement in the support capacity curve for the shotcrete, indicating the effectiveness of the corrective actions taken.

Recommendations

Based on the findings and analysis, it is recommended that:

1. For tunneling in weak rocks, the method of excavation, the time taken between different stages of excavation and support application, the adjustment of support measures based on site conditions, and the monitoring of deformation are crucial factors for the successful completion of the project.
2. In NATM tunneling, monitoring plays a vital role not only in confirming the adequacy of the installed support but also in making timely decisions regarding the need for correction measures and additional support measures to ensure stability.
3. Deformation monitoring should be continued for several weeks; as underground structures are subject to long-running deformation phenomena. The secondary lining should be installed only when an acceptable limit of convergence is achieved.
4. Monitoring of the tunnel deformation should continue during the tunnel operation period to ensure the long-term stability and safety of the structure.
5. The incorporation of an inverted arch in the tunnel design is recommended for weak rock formation where significant deformation is expected, as it enhances the overall stability and performance of the structure, as demonstrated by the numerical modeling results.

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