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Numerical evaluation of new Austrian tunneling method excavation sequences: A case study

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ABSTRACT

The main aspects that require attention in tunnel design in terms of safety and economy are the precise estimation of probable ground conditions and ground behavior during construction. The variation in rock mass behavior due to tunnel excavation sequence plays an important role during the construction stage. The purpose of this research is to numerically evaluate the effect of excavation sequence on the ground behavior for the Lowari tunnel project, Pakistan. For the tunnel stability, the ground behavior observed during the actual partial face excavation sequence is compared with the top heading and bench excavation sequence. For this purpose, the intact rock parameters are used along with the characterization of rock mass joints related parameters to provide input for numerical modelling via FLAC 2D. The in-situ stresses for the numerical modelling are obtained using empirical equations. From the comparison of the two excavation sequences, it was observed that the actual excavation sequence used for Lowari tunnel construction utilized more support than the top heading and bench method. However, the actual excavation sequence provided good results in terms of stability.

1. Introduction

Tunnels provide a feasible alternative to cross through the physical barriers or water body. With the rapid development of the world, the utilization of underground space plays an important role not only in urban areas, but also in rural areas and even under water. These underground excavations are not only in excavation friendly in-situ ground environment but also in difficult ground conditions. Keeping in view the complexity of ground conditions, different methods are used for tunnel construction including TBM (tunnel boring machine) tunneling, NATM (new Austrian tunneling method) method of tunneling, cut and cover excavation, immersed method, jacked box tunneling, etc. Out of these, NATM approach utilizing drill and blast excavation technique, offers flexibility in geometry and is widely used in rock tunneling worldwide for almost any size of tunnel.

NATM, which adjusts the excavation sequence mainly in terms of round length, type and timing of support installation, allows for tunneling through a variety of rock mass conditions [1]. Along with the construction sequence in tunneling, there are other features that influence the ground behavior, including the ground composition (rock mass type, in-situ stresses and groundwater) and project-related features (shape and size). Further, the support is applied according to the ground behavior [2,3]. For the excavation method selection, the dominant factors are: properties of surrounding material, shape and size of the tunnel, in-situ and induced stresses, underground hydrology, structural geology and characteristics of weak zone [4]. The tunnel cross section may be divided into multiple drifts, keeping in view the size of excavation and prevailing ground conditions. The selection of an appropriate excavation method for large span tunnel is a crucial factor for the successful completion of the project [5]. The cost and time of tunnels construction are strongly influenced by the excavation method. Selection of appropriate method for excavation is mostly influenced by engineering experiences rather than theoretical calculations. Excavation methods and partial excavation sequencing schemes and their corresponding support application for a tunnel are based on complicated interactions between several factors e.g. safety, cost and schedule considerations [6]. The main aim

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during decision related to the excavation method and sequence is to maintain the structural integrity of the material surrounding a tunnel.

Base on the stand-up time concept, the conventional tunnel excavation is either full face or partial face excavation [7]. Partial face excavation has several types including the top heading and bench, pilot tunnel, side drifts, etc. Initial support is applied early to stabilize and prevent the ground from excessive deformation, depending on ground conditions. Yu & Chern have proposed a diagram for the selection of tunnel excavation methods (either full face, central diaphragm or side wall drift) based on tunnel span and the ratio of tunnel confining ground's uniaxial compressive strength to vertical stress [4]. An excavation sequence using partial excavation is used as a strategic method for rock burst control when tunnels are excavated in a high stress environment [8]. Top heading and bench sequence of partial excavation approach is often selected for excavating tunnels with large span [1].

In this paper, the actual partial excavation sequence of Lowari tunnel project is compared with the top heading and bench excavation sequence in highly stress jointed rock mass environment. This work is the continuation and extension of previous research and the numerical model used in the paper for the comparison of the two excavation sequences has been validated already [9–11]. In Lowari tunnel construction, due to the revised planning, the small tunnel is enlarged and made feasible for the two-way traffic. In this project, to use the already excavated tunnel for vehicle transportation, construction of a new parallel tunnel was a costly alternative. If this two-way traffic road tunnel was planned before the excavation of small tunnel, the possible excavation sequence would have been top heading and bench excavation. Therefore, the actual adopted excavation sequence is compared with top heading and bench excavation and the stability is evaluated numerically in terms of the major principal stress and axial stresses in rock bolts.

2. Project description and geology of the area

The highway N-45 is running from Nowshera District to the town of Chitral via Dir in Khyber Pakhtunkhwa (KP) province, Pakistan. The National Highway Authority (NHA) of Pakistan is the client of this two lane 309 km highway, which includes the Lowari tunnel. The Lowari tunnel has reduced the travelling distance and time, and round the year access to Chitral which was not possible during winter earlier. The location of the tunnel is shown in Fig. 1. Initially, the tunnel was excavated for vehicles transportation through rail and called Lowari Rail Tunnel (LRT). The cross-section of this tunnel was revised by the NHA after the excavation of the LRT, for a road tunnel and was named Modified Road Tunnel (MRT). This MRT is part of the road linking Central Asian countries with Pakistan through Chitral district, traversing Vakhan, Afghanistan. The LRT and MRT were completed in 2009 and 2016, respectively, and were opened for transportation in 2017.

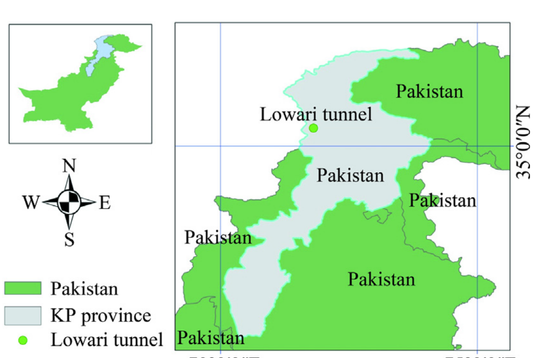


Fig. 1. Location of Lowari tunnel project.

The project area is situated between the active Indian Plate and Eurasian Continental Plate, and belongs to the active seismic zone due to the on-going subduction of Indian plate [12]. The geological investigations conducted for the design and construction of LRT exposed five major geological units along the tunnel route, i.e. the meta sediment unit, the granite unit, the meta igneous unit, the meta volcanic unit, and the biotite granite unit. Further, these units were also matched during the construction of LRT and their details are as below:

(1) Metasediment Unit: Schist and quartzite are mainly available in this geological unit and show different degree of weathering in which quartzite is very slightly weathered as compared to schists. Presence of water in the schist has a negative effect on the strength. Metasediment is the weakest rock mass unit along the tunnel route.
(2) Metavolcanic Unit: Amphibolites, densely jointed, slightly to distinct banding visible is a metavolcanic rock which is mostly exposed on the surface without signs of intensive weathering. Joints are usually only stained and sometimes open due to expansion on slopes. Sound rock is dominant in this rock mass unit.
(3) Metaigneous Unit: In metaigneous unit, gneiss is commonly visible on the surface without signs of rigorous weathering. Like the previous rock mass unit, the joints in this unit are also stained and sometimes open due to expansion on slopes. Sound rock is prevailing, and sheared portions are rare.
(4) Granite Unit: Granite in the project area is mostly exposed on the surface with no signs of intensive weathering and forming steep rock walls. Sound rock is dominant and sheared portions are rare. Local faulting and heavy jointing due to which the rock mass may be weakened are encountered in certain sections of the tunnel.
(5) Biotite Granite Unit: Exposed Biotite Granite is usually intensively weathered and not found fresh on the surface. Effect of weathering is decreasing with increasing overburden. Near water bearing zones, alteration is found at greater depths. This unit is found with frequent transitions into Granite.

3. Intact rock and rock mass properties

The NATM approach has been used during the construction of Lowari tunnel project. In NATM, the geological face mapping, geotechnical monitoring, and observations during tunnel construction help in selection of optimum support system and excavation sequence. The predicted behavior during the tunnel design and observed behavior during construction is compared keeping in view the monitored deformation, support utilization, and overbreak volume. Deviation between the predicted and observed behavior leads to a re-evaluation of the design process, resulting in modifications to the support and excavation methods.

During the excavation of LRT and MRT, informations were recorded for both intact rock and discontinuities, as documentation of in-situ data is a compulsory part in NATM tunnelling approach [13]. The intact rock properties for the selected chainage where the rock units are granodiorite and gneiss, are shown in Table 1.

The rock mass rating (RMR) and tunneling quality index (Q) systems are used specifically for tunnel design in jointed rocks [14–

Table 1
Physical and mechanical properties of the intact rock.

Chainage	Overburden (avg.) (m)	Rock type	Intact rock strength (MPa)	E_i (GPa)	m_i	γ (kN/m^3)	v
3 + 900–4 + 000	915.6–971.1 (944.82)	Granodiorite	75	31.875	29	27.20	0.24
6 + 300–6 + 460	874.1–925.5 (903.55)	Gneiss	100	52.500	23	26.98	0.21

Notes: E_i is the intact rock modulus; m_i the material constant for intact rock; γ the unit weight; and v the Poison's ratio.

16]. Using the documented informations, rock masses are characterized using RMR and Q systems for joints and their charactristics. These values are used for the geological strength index (GSI) value determination using rock mass fabric approach [17]. In this approach, the common parameters of four classification systems (RMR, Q, GSI and rock mass index (RMi)), which characterize solely the rock mass, are used for rating the rock structure and the joint surface conditions. The ratings are grouped together in a common Fabric Index chart, which is further used for their correlation. In RMR system, for calculation of rock quality designation (RQD) rating (R_2) and joint spacing (x) rating (R_3), the contineous rating equations (Eqs. (1) and (2)) were used [18].

$$R_2 = 0.22RQD - 0.0002RQD^2 \tag{1}$$

$$\begin{cases} R_3 = 2.281\ln(x) - 3.41, & x = 5 \text{ - } 200 \text{ mm} \\ R_3 = 4.175\ln(x) - 13.51, & x = 200 \text{ - } 900 \text{ mm} \\ R_3 = 6.250\ln(x) - 27.55, & x = 900 \text{ - } 2000 \text{ mm} \end{cases} \tag{2}$$

Based on the characterization, the detailed joint informations and calculated GSI values are shown in Table 2.

The intact rock properties were extrapolated to the rock mass as shown in Table 3 with the help of RocLab software which is based on the generalized Hoek-Brown criteria [19].

4. Tunnel excavation and support

Stand-up time classification for an unsupported span was presented by Lauffer in 1958, since been modified by a number of authors and now forms part of the general tunneling approach known as NATM, which is also known as Austrian tunneling practice [20]. In Austrian tunneling practice, a ground class is assigned to ground conditions based on field observation. Each ground class is assigned a support system. This qualitative ground description is linked with excavation techniques, together with principles and timing of standard support requirements. In Lowari tunnel project, the 2001 guideline for conventional underground excavation were used for LRT and MRT construction. In these guidelines, the procedures from design to monitoring of underground excavation are summarized along with the suggested basic procedures of excavation and support design for underground structures [13].

4.1. Actual tunnel excavation

Road tunnel cross sectional area depends upon the way how it will accommodate the vehicle. Although the financial viability of a tunnel depends on its life cycle cost, however, the cross sectional area is directly related to the construction cost of the tunnel [1]. Originally conceived as a railway tunnel in 1975 for vehicle transportation, a small tunnel with 7.12 m of span was planned and named LRT [10]. Due to funds unavailability, the construction work on the 8.51 km long LRT started in 2005 and completed in 2009 [9]. This horseshoe shape tunnel was constructed using drill and blast method of excavation in rock followed by the support installation. The primary purpose of the LRT construction was to connect the Chitral district with the remaining country through an all-weather asphalt road. During this large span of time (1975 to 2009), the geopolitical and geostrategic condition of the region (Afghanistan and Central Asia) changed and therefore, the same route was decided for trade with Central Asia. In this condition, there were two alternatives: either to construct a new tunnel parallel to LRT or to enlarge the existing LRT for two-way road traffic. Finally, it was decided to enlarge the existing LRT (named MRT) and was completed in 2016. The drill and blast excavation approach followed by support installation is used during enlargement of LRT to MRT. This excavation includes the removal of shotcrete sprayed and rock bolts installed during LRT. Compared to the widely used excavation sequence in rock tunneling, the top heading and bench excavation, the amount of support installed during LRT construction is an additional cost as this was removed during the construction of MRT.

4.2. Top heading and bench excavation

The current empirical rock mass classification system has the application of predicting stand-up time [21]. This stand-up time predicts whether to use full face excavation or partial excavation in sequential excavation approach. The most widely used partial sequential excavation approach in rock tunnelling is top heading and bench excavation method. In this excavation approach, top heading excavation and support installation is completed first, followed by bench excavation. In this study, it is assumed that if MRT would have been constructed by top head and bench method, then what would be the rock behaviour and support requirements. Numerical modelling is adopted to accomplish this task.

5. In-situ stresses

The role of in-situ stresses is significant for the design and construction of tunnel. Knowledge of the in-situ stresses is mandatory to design the underground structure in the rock mass. There are numerous approaches presented for the determination of in-situ stresses and always needed to assess it in the best possible way [22]. In-situ stress measurement is a costly venture therefore alternative approaches e.g. experience from nearby underground project or empirical approaches are used for its estimation. Numerous empirical equations have been recommended by the

Table 2
Rock structure and joint surface condition characterization and corresponding GSI values.

Rock type	RQD/J_n	J_r/J_a	$R_2 + R_3$	R_4	GSI
Granodiorite	5.33	3	30.70	22.50	65
Gneiss	8.70	3	19.33	22.50	64

Notes: J_n is the rating for the joint sets; J_r the rating for the joint roughness; J_a the rating for the joint alteration; and R_4 the rating for the joint condition.

Table 3
Rock mass properties for different rock units.

Rock type	c (MPa)	Φ (°)	m_b	s	a	E_{rm} (GPa)	Sigc (GPa)	Sigt (GPa)
Granodiorite	4.704	47.730	8.309	0.0205	0.502	20.136	10.648	−0.185
Gneiss	4.745	48.130	6.358	0.0183	0.502	32.020	13.420	−0.288

Notes: c is the rock mass cohesion; Φ the rock mass frictional angle; m_b the reduced value of material constant m_i; s and a the constants for the rock mass; E_{rm} the rock mass deformation modulus; Sigc the rock mass uniaxial compressive strength; and Sigt the tensile strength of rock mass.

research community for the estimation of in-situ stresses. In this section, some of the commonly used equations are discussed.

The believed perception about the vertical stress (σ_v) shown in Eq. (3) reveals that σ_v increases with depth (H).

$$\sigma_v = \gamma \times H \tag{3}$$

where γ is the rock unit weight.

The horizontal stresses (σ_h) calculation in the in-situ environment are considerably more challenging than σ_v. The ratio of σ_h to σ_v is denoted by K_0. An elasto-static thermal stress model was formulated to take into account the action of tectonic forces by [23] and suggested the following equation for K_0 [20].

$$K_0 = 0.25 + 7E_h(0.001 + 1/z) \tag{4}$$

where z in Eq. (4) is the depth below the surface in m; and E_h the average deformation modulus in GPa.

Sheorey et al. suggested Eq. (5) for computing σ_h [24].

$$\sigma_h = (\nu/(1-\nu))\sigma_v + (\beta E_{rm} G/(1-\nu))(H + 1000) \tag{5}$$

where G = 0.024 ℃/m is the geothermal gradient; $\beta = 8 \times 10^{-6}$/℃ the linear thermal expansion coefficient; and ν the Poisson's ratio; and E_{rm} the rock mass deformation modulus.

The in-situ stresses calculated using the above equations are presented in Table 4. In the numerical modelling, the average values are used.

6. Numerical modelling

FLAC version 7.0 is used for the analysis. FLAC is an explicit 2D finite difference program that is suited for sequential excavation modelling. The actual tunnel sequence of excavation and support were followed i.e. excavation and support of the LRT followed by excavation and support of MRT. For comparison, the MRT was excavated using top heading and bench excavation approach. The two excavation stages for the actual as well as top heading and bench excavation are shown in Fig. 2. Due to the asymmetry of the excavation sequences in the actual excavation and top heading and bench approach, the entire domain was considered in the model.

The two excavation stages for each approach and three construction steps (excavation, applying soft shotcrete and rock bolt, and hard shotcrete) in each stage are simulated for analysis. To eliminate the boundary effect, enough distance from the tunnel to the sides of the model was used. The model dimensions are 80 m × 60 m for both approaches. The modified Hoek-Brown model was used for the analysis. This model is based on the nonlinear relation between major and minor principal stresses, σ_1 and σ_3, and the criterion is used for plastic yielding when σ_3 is compressive. Around the tunnel boundary, the fine mesh was simulated for the better results. The model was fixed at bottom and sides, and σ_{yy} (vertical stresses) were applied at the top of the model. The in-situ stress environment was created using gravity, σ_{yy}, and FISH function. During the three construction steps for each excavation stage, 40%, 30% and 30% relaxation is used.

7. Results and discussions

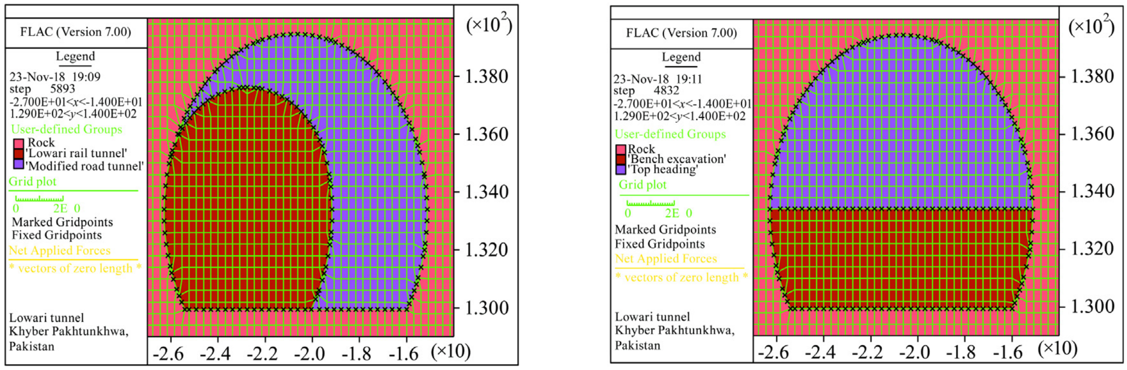
The numerical model is solved statically for both granodiorite and gneiss rock mass, each for the actual excavation sequence and top heading and bench excavation approach and their results in term of major principal stress and axial forces on rock bolts are compared. The numbering of rock bolts are different in two excavation cases, but this is due to the order in which the structural elements are created.

As can be seen in Fig. 3, rock bolts installed in top heading experience high axial stresses when excavation is carried out through top heading and bench sequence as compared to the actual excavation. However, the trend is different for rock bolts installed at bench level. Comparing the axial stresses in the two different rock units, the rock bolts installed in granodiorite rock unit are comparatively heavily loaded than gneiss rock unit. This is due to better properties of gneiss rock mass and low overburden height as compared to granodiorite rock mass unit. The maximum axial stress on rock bolt is for rockbolt number 2 of granodiorite rock unit and tunnel excavated with top heading and bench excavation sequence which is 1.374E05.

During the excavation of the tunnel, the stresses in the vicinity of the excavation are changed and new stresses are induced due to the redistribution of virgin stress field. The induced principal stresses are mutually perpendicular, but they are inclined to the direction of applied in-situ stress field. The minimum principal stress is negligible at excavation periphery, which gradually increased with the distance from the excavation periphery due to the increase in confinement. On the other hand, the maximum principal stress was at its peak near the excavation periphery, which gradually decreased with the distance from the excavation periphery. The ratio of the major principal stress and rock mass strength is used as a simple index for the stability [25]. Therefore, the major principal stress contours are plotted for the excavation sequence effect as shown in Fig. 4. The results showed that top heading and bench excavation sequence has higher contour for major principal stress as compared to the actual excavation sequence for both granodiorite and gneiss units.

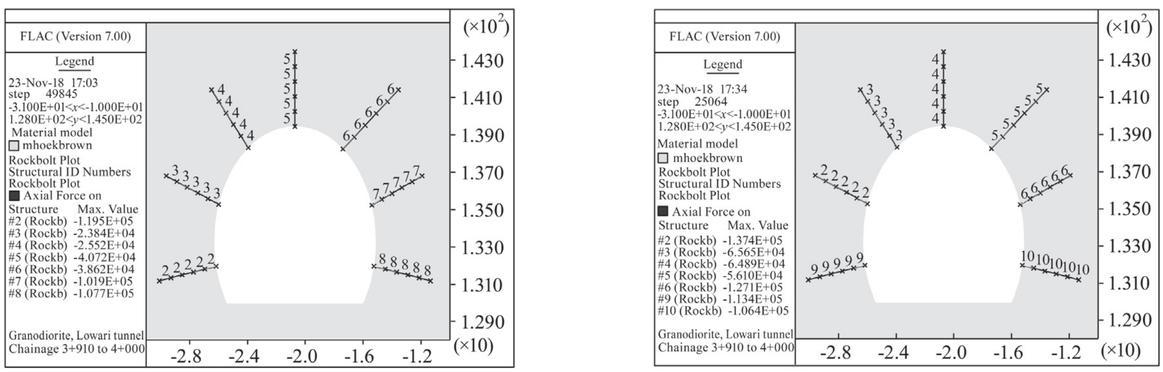
Table 4
Estimated in-situ stresses using empirical equation.

Rock type	Vertical stress (MPa)	Eq. (4)		Eq. (5)	
		Horizontal stress	K_0	Horizontal stress	K_0
Granodiorite	25.704	13.88	0.54	13.433	0.52
Gneiss	24.39	17.61	0.722	14.297	0.586

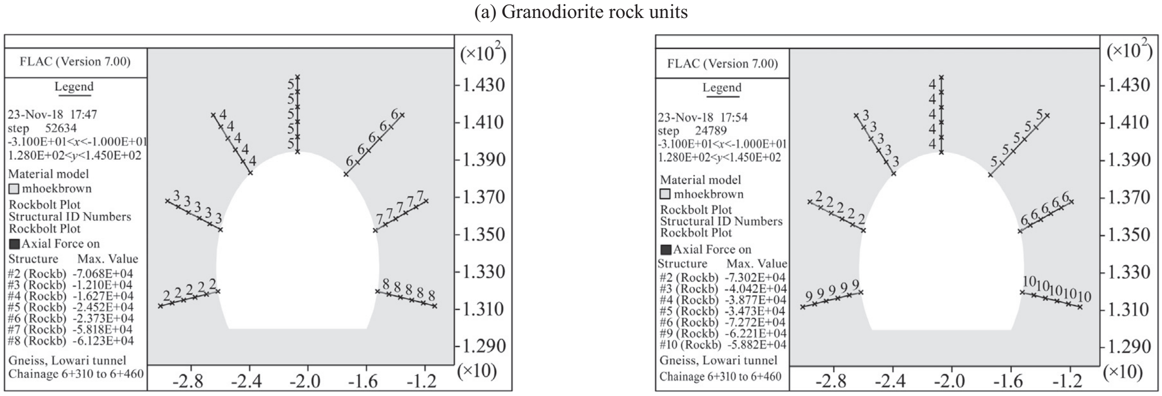


(a) Actual tunnel excavation (b) Top heading and bench method

Fig. 2. Actual tunnel excavation, top heading and bench method for Lowari tunnel.



(a) Granodiorite rock units



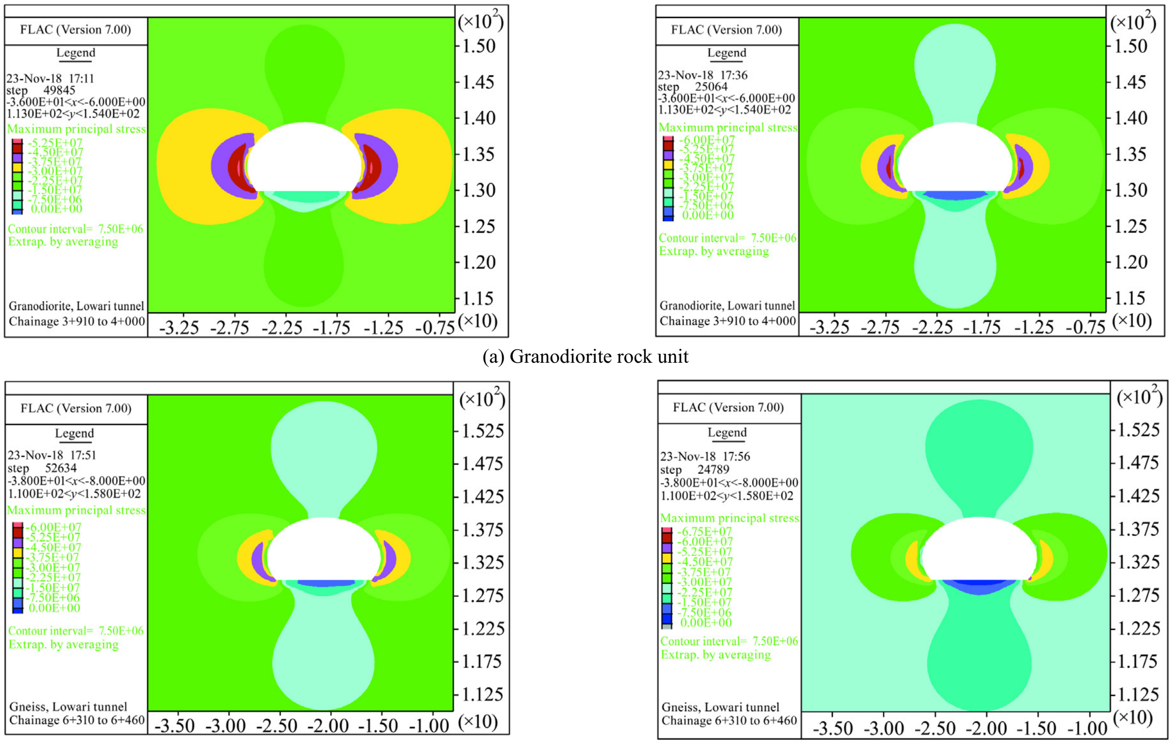
(b) Gneiss rock units

Fig. 3. Axial stresses on rock bolts due to the different excavation sequences (actual tunnel excavation (left) and top heading & bench excavation sequence (right)).

8. Conclusion

This study compares the actual construction sequence of Lowari tunnel project with the widely used excavation sequence in the rock tunnelling, the top heading and bench excavation sequence. Due to the revised planning, the top heading and bench excavation sequence is not adopted during the construction of MRT which results in more support installation, in terms of shotcrete and rock bolts, during the actual excavation. In the MRT construction, most of the installed rock bolts and sprayed shotcrete of LRT were removed.

It is observed during numerical analysis that by following the actual excavation sequence, the principal stress contours are lower as compared to the top heading and bench excavation sequence. The major principal stress contours are 5.25E7 and 6.0E7 for granodiorite and Gneiss rock units, respectively, using the actual excavation sequence. However, in case of top heading and bench excavation sequence, these values are 6.0E7 and 6.75E7, respectively. The numerical analysis results in terms of axial forces in rock bolts also showed that most of the rock bolts installed duirng the actual excavation sequence experienced less amount of stress as compared to the assumed top heading and bench approach. With respect to stability of the tunnel, the excavation sequence adopted in Lowari tunnel construction was more credible as compared to the assumed excavation sequence. This stability trend in the case of actual excavation sequence is due to twice releasing of stresses as compared to the top heading and bench excavation sequence. After LRT excavation and support, a new stress environ-



(a) Granodiorite rock unit

(b) Gneiss rock unit

Fig. 4. Major principal stress scenario due to different excavation sequences (actual tunnel excavation (left) and top heading & bench excavation sequence (right)).

ment created around LRT once stresses are released. This stress environment around the LRT is further lowered during the construction of MRT.

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