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Research article

Effects of different empirical tunnel design approaches on rock mass behaviour during tunnel widening

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ABSTRACT

Empirical based approaches play an important role in tunnel excavation and support system design. These approaches are considered to be very effective in optimising the process of tunnel excavation and particularly tunnel widening. Several reliable empirical approaches have been developed, however the selection or utilisation of an appropriate empirical method for designing the widening of a tunnel is still a challenging task. Therefore, in this work, the analysis of seven different empirical design approaches was carried out to determine the rock mass behaviour during tunnel widening in high in-situ stress state. These approaches include New Austrian Tunnelling Method, Rock Mass Rating, Rock Mass Quality, Rock Mass Index, Rock Structure Rating, Geological Strength Index and Basic Quality Index. On the basis of simulated statistical results obtained from the said empirical approaches, it was found that the application of Rock Mass Quality approach is highly effective in the tunnel widening since it can satisfactorily incorporate the equivalent dimensions and in-situ stress condition of widened tunnel. The method furnishes optimised reinforcement and support design. Additionally, this study also produces reliable data related to the initial excavation of tunnel which can be helpful in defining precise rock mass parameters during tunnel widening.

1. Introduction

During the last half century, the use of underground space technology comprising of tunnels and caverns was nourished tremendously in providing a safe and comfortable mode of connection through highways and railways for the ease of commuters. However, the need of the time is to adjust and accommodate the growing traffic on highways keeping in view the population growth, urbanization and the increasing size of goods transportation. In this regard, the tunnels constructed on such highways are being widened wherever required in different parts of the world. As various empirical methods are being adopted to design the tunnels and caverns, it is mandatory to have the knowledge of rock mass behaviour in widening of tunnel using the existing empirical design approaches and thereupon understanding to apply most optimal design method in hard rock under high stress conditions. It is also worthwhile to have understanding to empirical design methods that are suitable in widening process which cater for the tunnel width, dismantling of already installed support system, rock mass behaviour and thereupon installation of new support in widened tunnel.

The different empirical methods which are widely in use are New Austrian Tunnelling Method (*NATM*) [1], Rock Mass Rating (*RMR*) [2], Rock Mass Quality (*Q*) [3], Rock Mass Index (*RMi*) [4], Rock Structure Rating (*RSR*) [5], Geological Strength Index (*GSI*) [6] and Basic Quality Index (*BQ*) Method [7] etc. These methods were used to evaluate the rock mass behaviour in widening for reinforced rock mass. For this study, the site-specific conditions were opted from Lowari Tunnel located in northern part of Pakistan where the widening of a tunnel was carried out.

Currently, different researchers have studied the possibilities of widening of tunnel with respect to safety in case of tunnel widening being carried out simultaneous with tunnel traffic operations in flow. Babar et al. [8] studied the tunnel deformations for symmetric and asymmetric widening of different tunnel shapes under different stress conditions. Lunardi [9, 10] devised and elaborated the method of tunnel widening during the tunnel operation such that the tunnel could be used for traffic concurrent with the tunnel widening process.

Lee et al. [11, 12] studied the behaviour and stability of enlarged twin parallel tunnels by evaluation of stress and deformation on central pillar and concluded that the tunnel widened on one side (asymmetric)

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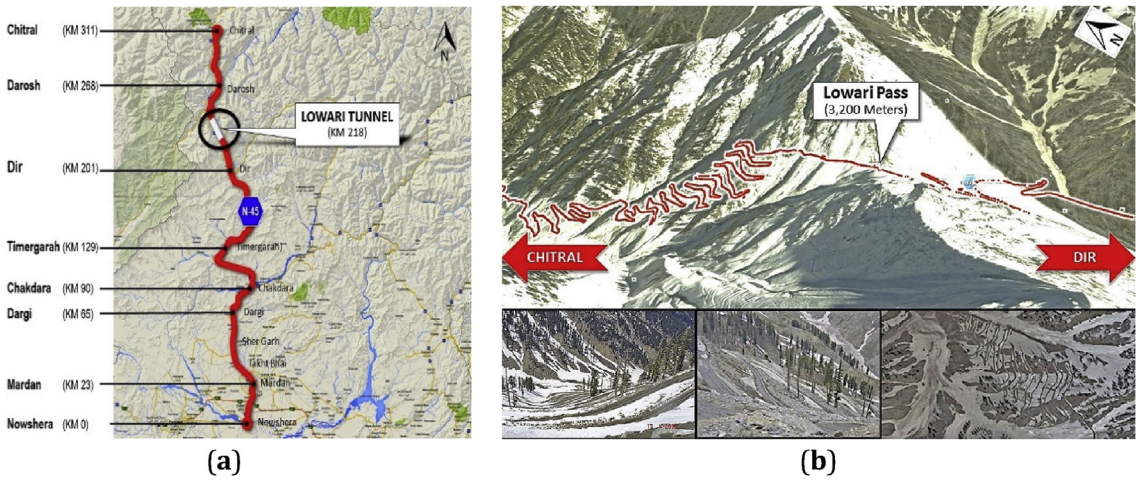


Figure 1. Lowari Tunnel site: (**a**) Location plan; (**b**) Snow and hilly terrain of Lowari Pass.

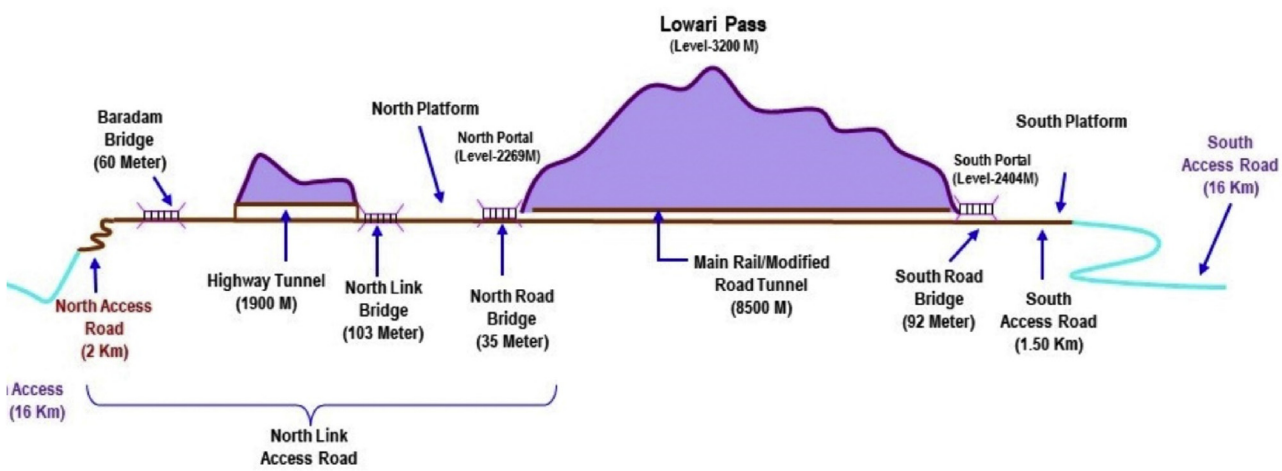


Figure 2. Layout Plan of Lowari Tunnel Project comprising of 8500 M and 1900 M of two tunnels.

produces 5–20% more crown displacement as compared to tunnel widening on both sides (symmetric). Choi et al. [13] devised the methodology for widening of any existing tunnel. It was concluded in the work that the bi-directional tunnel widening has a disadvantage in that the whole tunnel support has to be removed and is comparatively time consuming and expensive.

Hu et al. [14, 15] considered three modes of tunnel expansion including single side expansion, double side expansion and peripheral expansion adopted for Damaoshan Tunnel in China and found single side expansion as the most appropriate solution. They also devised mechanical computational models for single side widened tunnel.

Andrea [16] has elucidated the case study of Castellano tunnel in Italy for widening of brick lined railway tunnel in silty clay/clayey silt with lenses of sand. It was found in the study that the consolidation of strata behind the lining could be achieved by the combined effect of fiberglass elements and cement motor grout injection, which leads to the tunnel stability during the widening process as well as to control the tunnel convergence.

Bertuzzi [17] has considered the case of widening of tunnel during the extension of M2 Motorway in Sydney, Australia in Hawkesbury sandstone and performed back analysis to obtain the values of deformation modulus as well as horizontal to vertical stress ratio which were found to be in order with the range used in Sydney.

The possibility of widening the tunnel in urban areas can also be possible effectively using non-vibrational rock splitting method. The rock splitting method was elaborated by Jafri and Yoo [18] wherein, the rock mass excavation in tunnels was analysed through discrete element analysis.

The studies aforementioned are either related to the stresses and behaviour on the central pillar between twin tunnels during the widening or discussed the symmetric and asymmetric widening of tunnels. However, there is a need to evaluate the best available empirical design method that could be applied for widening of tunnels in high in-situ stress conditions.

In this regard, preliminary excavation and support systems for both the initial tunnel and widened tunnel were analysed separately using all empirical design approaches under the study. Thereafter, the selected excavation and support system were simulated for each empirical design approach with specific in-situ site conditions at Lowari Tunnel in Finite Difference Method (FDM) to obtain the deformation at three critical locations comprising of tunnel crown, spring line and invert. The Lowari Tunnel was opted in the study where the asymmetric widening was carried out in under construction rail tunnel by dismantling the initial tunnel support system along with tunnel surrounding rock mass to achieve the larger profile section for highway. The tunnel deformations were recorded both for initial tunnel profile and widened tunnel profile under New Austrian Tunnelling Method (NATM).

Among the different approaches, the design method which provided the most controlled deformations with respect to other design methods was selected as the most appropriate method for the widening of tunnel.

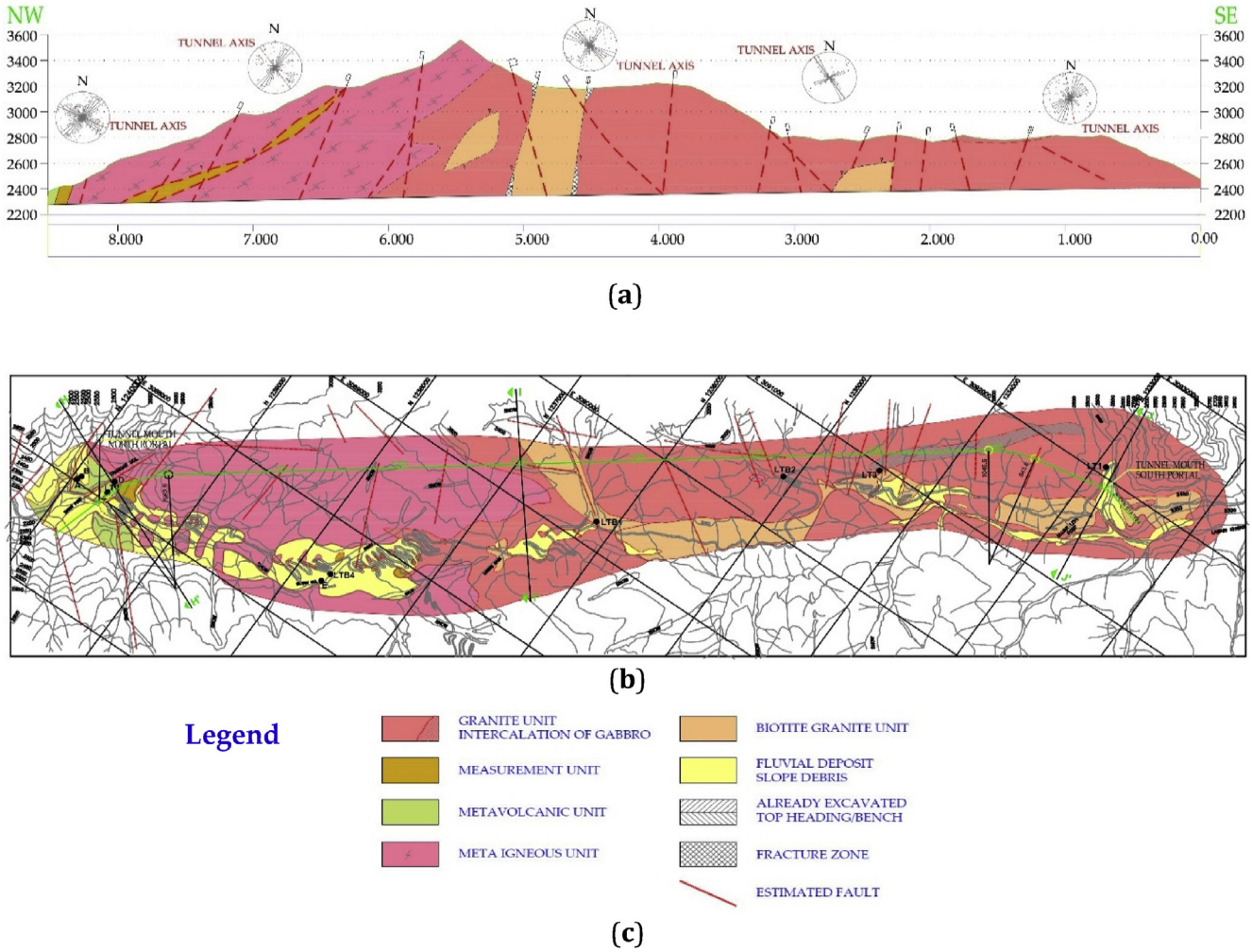


Figure 3. Geological profile along the tunnel axis: (**a**) Geological cross section along the tunnel alignment; (**b**) Plan view of the tunnel alignment, (**c**) Legend.

2. Lowari Tunnel project

2.1. Project brief

Lowari Pass is situated in Pakistan at height of 3,200 Meter above Mean Sea Level (MSL) at the junction of District Chitral and Dir, that covered with snow for the half of year as depicted from Figure 1 that caused termination of access for people of Chitral valley with rest of country. To provide all weather link access to the people of valley, a project of Lowari Tunnel was initiated in the year 2005 with concept of piggy back rail through 8.5 km tunnel with 1.9 km of access road tunnel. The project layout is illustrated in Figure 2.

Using the horseshoe tunnel profile with cross sectional area of 45 m^2, the excavation of 8.5 km tunnel was carried out from 2005 to 2009 incorporating the installation of primary support system including rock bolts, wire mesh and shotcrete.

2.2. Site geology

The project site is situated in Kohistan Complex duly positioned between the Eurasian Continental Plate and Indian Plate. The Kohistan Batholith [19] as part of Trans-Himalayan Batholith and volcanic sedimentary groups are main constituents with slight influence of metamorphism. Faulting and thrusting as regional deformation have occurred owing to the movement of tectonic plates. Due to this reason, the "Building Code of Pakistan – Seismic Provision" [20] has placed the area in "active seismic zone".

The most dominant rock present at site is granite rock mass interbedded with Gneiss, Gabbro and Granodiorite. A detail geological profile along the tunnel alignment was established to demonstrated different geological formation, geological structures and rock mass lithologies. The location and orientation of faults were also depicted in the layout. The geological layout profile is illustrated in Figure 3.

2.3. Conversion from rail tunnel to road tunnel

According to the scheduled timeline, as the secondary concrete lining was about to be installed during year 2009, the regulatory authority made a decision to convert the rail tunnel profile (Concept at Figure 4a) to road tunnel in order to accommodate the anticipated traffic from Central Asian States by including two road lanes in the tunnel. Thus, it was presumed during the time that the tunnel could be easily widened since the secondary lining was not installed in the original tunnel at that time. Thus, asymmetric widening of tunnel was carried out to obtain a total excavated area of 85 m^2 for accommodating two lanes each of 3.5 m width as well as walkways. The widening of tunnel was commenced in year 2012 after dismantling one third (1/3) of the primary support system and excavation was carried out for widen portion in reinforced rock mass. The newly obtained larger tunnel profile after widening is hereafter called "Modified Tunnel" in this study. The modified tunnel pattern

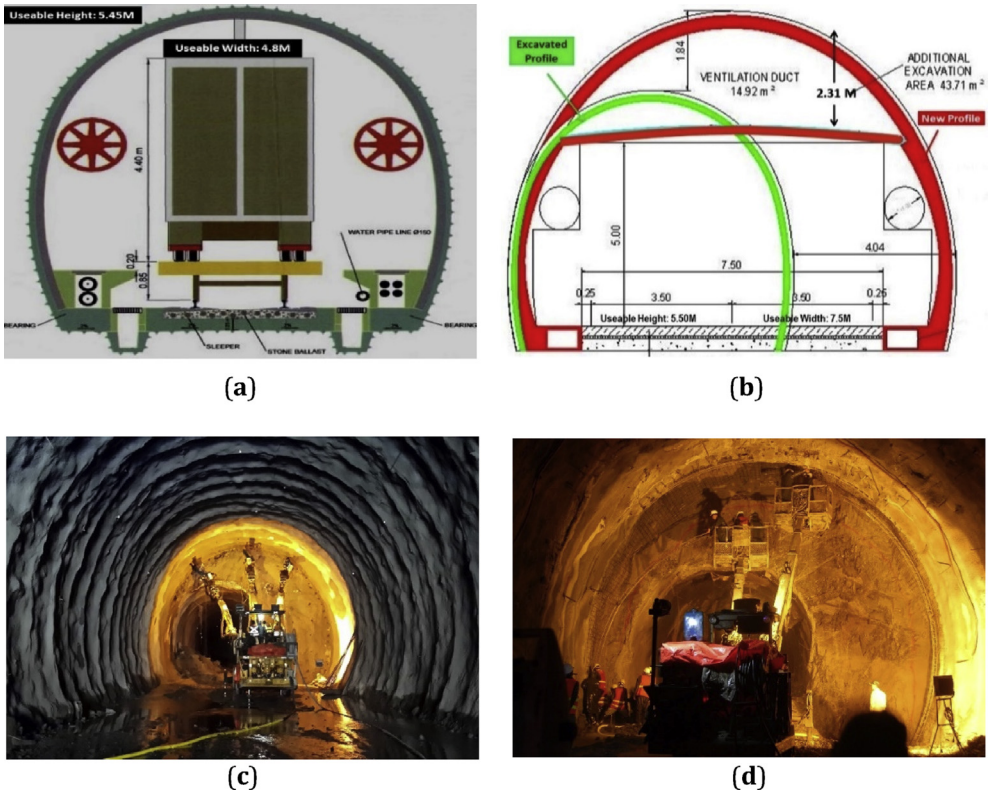


Figure 4. Tunnel profiling: (**a**) Original tunnel concept; (**b**) Modified tunnel concept; the green profile indicates the original tunnel and red profile indicates the modified tunnel; (**c**) Physical widening of tunnel (**d**) Support installation at widened portion.

Table 1. Rock mass type (RMT) estimation at lowari tunnel project [21, 22, 23].

Rock Mass Type	Rock Type	Estimated Uniaxial Compressive Strength (MPa)		Weathering	Spacing of Discontinuities[1]	Properties of Discontinuities[2]
		UCS	Point Load			
RMT-1	Granite, Granodiorite, Gabbro	>100 - 250	25–60	Fresh to slightly weathered	Moderate to wide	Clean, rough with stained
RMT-2		>100 - 200	20–50	Slightly weathered	Close to moderate	Stained, rough
RMT-3	Orthogneiss, intercalations of amphibolite (*metaigneous unit*)	>100 - 250	20–50	Fresh to slightly weathered	Moderate to wide	Clean, rough
RMT-4		>100 - 200	15–45	Slightly weathered	Close to moderate	Stained, rough
RMT-5	Metavolcanics	>100 - 200	15–50	Fresh to slightly weathered	Moderate to wide	Clean, rough
RMT-6		>50 - 200	10–35	Slightly weathered	Moderate to wide	Stained, smooth
RMT-7	Metasediments, mainly quartzite	>100 - 250	15–45	Fresh to slightly weathered	Moderate to wide	Smooth
RMT-8	Metasediments, mainly schists	5 - > 25	0.5–5	Slightly to moderately weathered	Close to very close	Smooth, fillings
RMT-9	Biotite granite with transitions to granodiorite and granite	50–150	5–40	Slightly to moderately weathered	Moderate to wide	-
RMT-10		1–5	0.5	Moderately to highly weathered	Close to moderate	-
RMT-11	Fault and shear zones, crushed rock	1–5	5	Moderately to highly weathered	Sheared and crushed	-

[1] *Moderate: Spacing (*20cm – 60cm*); Wide: Spacing (*60cm – *2m)*.
[2] *Clean: No filling material; Rough: irregular surface; Stained: Marks of irregularities; Smooth: flat surface; Fillings: presence of alternative fill material in joint.*

is shown in Figure 4b and physical widening concept of the tunnel is shown in Figure 4c.

3. Tunnel excavation and support design

3.1. Tunnel empirical design approach

The "New Austrian Tunnelling Method" – (NATM) was used to design the excavation and support system of Lowari Tunnel. The method was applied as per the guidelines of Austrian Society of Geomechanics [1]. The main parameters and procedures adopted in the design of the excavation and support system at Lowari Tunnel are elucidated in following sections.

3.1.1. Rock mass types (RMT)

The rock mass type was designated and defined with respect to the geotechnical features based on the characteristics of intact rock such as lithology, physical property, discontinuities, properties of faulted rock, influence of weathering etc. On the basis of rock mass characterization, the rock mass types (RMT) were classified in eleven categories as summarized in Table 1.

Table 2. Rock Mass Behaviour Type estimation at Lowari Tunnel Project [21, 22, 23].

Rock Mass Behaviour Type (RBT)		Description of Potential Failure Modes/ Mechanism during Excavation of the Unsupported Rock Mass
RBT-1	Stable	Stable rock mass with the potential of small local gravity induced falling or sliding of blocks.
RBT-2	Stable with the potential of discontinuity-controlled block fall	Deep reaching, discontinuity controlled, gravity induced falling and sliding of blocks, occasional local shear failure.
RBT-3	Shallow spalling and shear failure mechanism, raveling material from the crown	Shallow stress induced shear failures in combination with discontinuity and gravity-controlled failure of rock mass.
RBT-4	Deep seated shear failure	Deep seated stress induced shear failure and large deformation.
RBT-5	Rock Burst	Sudden and violent failure of the rock mass, caused by highly stressed brittle rocks and the rapid release of accumulated strain energy.

Table 3. Excavation class distribution opted at lowari tunnel project [21, 22, 23].

Excavation Class (EC)	Related RBT (As per section 3.1.2)	Allowable Deformation (mm)	Excavation Class Extended to Tunnel Length (m)	Section Percentage (%) with respect to Tunnel Length
1	RBT-1	50	1,347	15.85
1a	RBT-5	50	317	3.73
2	RBT-2	50	4,856	57.13
3	RBT-3	100	1,109	13.05
4	RBT-4	150	475	5.58
5	RBT-4	200	396	4.66
Total			8,500	100.00

Table 4. Geological data of the tunnel face at chainage 3 + 680 [21] [23].

Description	Details
Tunnel Chainage	3 + 680
Rock Type	Granodiorite with intercalation of Gabbro
Overburden	832 m
Uniaxial Compressive Strength (*UCS*)	150 MPa
Density (ρ)	2700 kg/m^3
Geological Strength Index (*GSI*)	60
Poisson's Ratio (υ)	0.25
Modulus Ratio (*MR*)	425
Elastic Modulus of Intact Rock (E_i)	63.75 GPa
Deformation Modulus (E_{rm})	33.15 GPa
Bulk Modulus (*K*)	22.10 GPa
Shear Modulus (*G*)	13.26 GPa
Disturbance Factor (*D*)	0
Hoek-Brown Parameter (m_i)	30
Hoek-Brown Parameter (m_b)	7.19
Hoek-Brown Parameter (s)	0.01174
Hoek-Brown Parameter (a)	0.5028
Rock Mass Type (*RMT*)	2
Rock Mass Strength	50–100 MPa (*opted average i.e.* 75 MPa)
Spacing of Discontinuities	20-60 (*medium*) & 60–200 (*widen*)
Block Size	60-200 (*large*)
Water Condition of Face	Damp to Dry
Jointing Pattern	Two to three sets of joint

Table 5. In-situ stress condition opted for the analysis and deformation (z-axis) at tunnel crown.

Description	Details
Initial Tunnel Diameter along the Spring Line	6.0 m
Modified Tunnel Diameter along the Spring Line	11.0 m
Overburden Height	832 m
Horizontal/Vertical Ratio (*K*)	2
Vertical Stresses (σ_z)	34 MPa
Horizontal Stresses (σ_x)	68 MPa
Longitudinal Stresses (σ_y)	34 MPa

3.1.2. Rock Mass Behaviour Type (RBT)

On the basis of Rock Mass Type (RMT), the Rock Mass Behaviour Type (RBT) was estimated at Lowari Tunnel as per guidelines defined in NATM and defined in Table 2.

3.1.3. Estimated Excavation Classes (EC)

Based on the determination of Rock Mass Types (RMT) and Rock Mass Behaviour Types (RBT) the six different Excavation Classes (EC) were estimated as per guidelines [21]. Each EC has its own characteristics, Rock Behaviour Type (RBT), excavation round length, support system and maximum deformation limit. The different EC categories at the project are stipulated in Table 3.

3.2. Geological detail at tunnel face

The major portion on Excavation Class along the tunnel alignment was EC-2 having a weightage of 57.13% which comprises of RBT-2 as shown in Table 3. Therefore, it was appropriate to have a representative tunnel section which fell under EC-2 for the study. In this regard, multiple tunnel sections based on EC-2 were initially selected, wherein, the tunnel section at chainage 3 + 680, was found most appropriate as all the related data including geological features and deformations were available to simulate the rock mass behaviour with respect to different empirical design approaches. The geological mappings of tunnel face section of initial tunnel (3 + 680) and modified tunnel (3 + 680) are provided in Figure 7 and Figure 8. The geological details duly extracted from said mapping for particular area are elucidated in Table 4.

4. Comparison of tunnel deformation through actual monitoring and numerical simulation

The comparison of deformation at horseshoe shape tunnel as opted in Lowari Tunnel were made between actual tunnel monitoring at project site and tunnel deformation using numerical simulation through Finite Difference Technique (FDM) using FLAC 3D [24]. In this regard, following steps were followed.

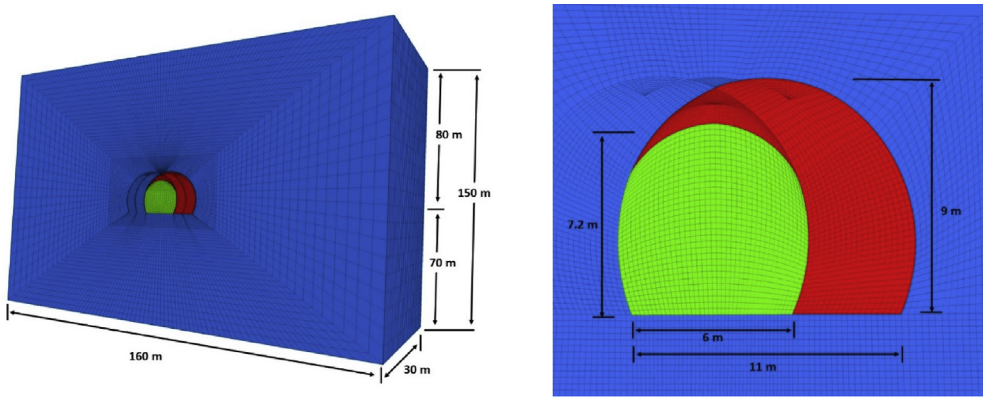
4.1. In-situ stress estimation

The tunnel section at chainage 3 + 680 was selected for the study having the geological parameters as stipulated at Table 4. At this particular section, the overburden was observed as 832 m. The in-situ vertical stress (σ_v) was calculated by the empirical relation defined by Hoek and Brown [25] as stipulated at Eq. (1).

$$\sigma_v = 0.027\, Z\ (MPa) \tag{1}$$

where "*Z*" is the overburden height in meters over the tunnel up to the ground surface. Sheorey [26] has proposed the equation for horizontal to vertical stress ratio "*K*" as given below:

$$K = 0.25 + 7E_h\,[0.001 + 1/Z] \tag{2}$$



(a) (b)

Figure 5. Model developed for the simulation of excavation of initial and modified tunnel; (a) 3D model geometry; (b) Tunnel excavation detail; green color indicates the original rail tunnel profile and red color indicate the widened portion of tunnel.

where "E_h" is the average deformation modulus in GPa. Thereupon, the horizontal stress (σ_h) was calculated with Eq. (3).

$$\sigma_h = K\ \sigma_v \tag{3}$$

As Granodiorite with intercalation of Gabbro rock was encountered at selected section, thereupon, the bulk and shear modulus were calculated from the deformation modulus. The deformation modulus (E_{rm}) was calculated through the empirical equation as established by Hoek and Diederichs [27] after evaluation of series of field data of in-situ deformation modulus test. The proposed equation is stipulated here as:

$$E_{rm} = E_i \left[0.02 + \frac{1 - \frac{D}{2}}{\left(1 + e^{\left(\frac{60+15D-GSI}{11}\right)}\right)} \right] \tag{4}$$

Wherein, the GSI was determined with respect to site rock condition on the principles and guide lines defined by Hoek and Brown [28], different minor discontinuities including the fissured, fracture, joints were also considered in rock mass through the *GSI* as defined by Hoek. "*D*" is the Disturbance Factor [27]. "E_i" is elastic modulus of intact rock and calculated from following equation as defined by Deere [29]:

$$E_i = MR\sigma_{ci} \tag{5}$$

"*MR*" is Modulus Ratio as established by Deere [29], whereas "σ_{ci}" is uniaxial compressive strength of rock taken from [23].

On the basis of aforementioned relations, the in-situ stress state conditions at selected section of Lowari tunnel were calculated and assembled in Table 5.

4.2. Model generation

The model was generated from actual tunnel profile that opted in Lowari Tunnel for initial and modified tunnel. In this regard, the section was simulated in FLAC 3D to get the tunnel profile both for initial tunnel as well as modified tunnel. The model illustration is shown in Figure 5.

The width of initial tunnel at spring line was 6 m and the width of tunnel after widening at the level of spring line was 11 m. Therefore, the large tunnel diameter of 11 m was considered for defining the boundary condition of model which were extended 80 m (7D) in lateral directions, 70 m (6D) bottom model boundary and the model boundary above the tunnel was extended 80 m (7D) from center of the tunnel as per the finding of Su et al. [30]. In longitudinal direction, the model was extended for 30 m in length.

The drive length of 2.5 m was opted in initial tunnel during on site excavation along the chainage 3 + 680, however, the drive length was curtailed to 2.0 m in excavation of modified tunnel along the selected section due to slight change in rock characteristics and increase in span length from initial tunnel profile. Accordingly, the mentioned drive lengths were opted in the numerical simulations with total 12 drives being considered in initial tunnel having 1250-time steps in each excavation drive and 15 drives being considered in the modified tunnel having 1000-time steps in each excavation drive. 15,000 time-steps each were provided for the excavation of both the initial tunnel and modified tunnel, such that all results for stress and deformation were obtained for the number of time-steps up to a total of 30,000.

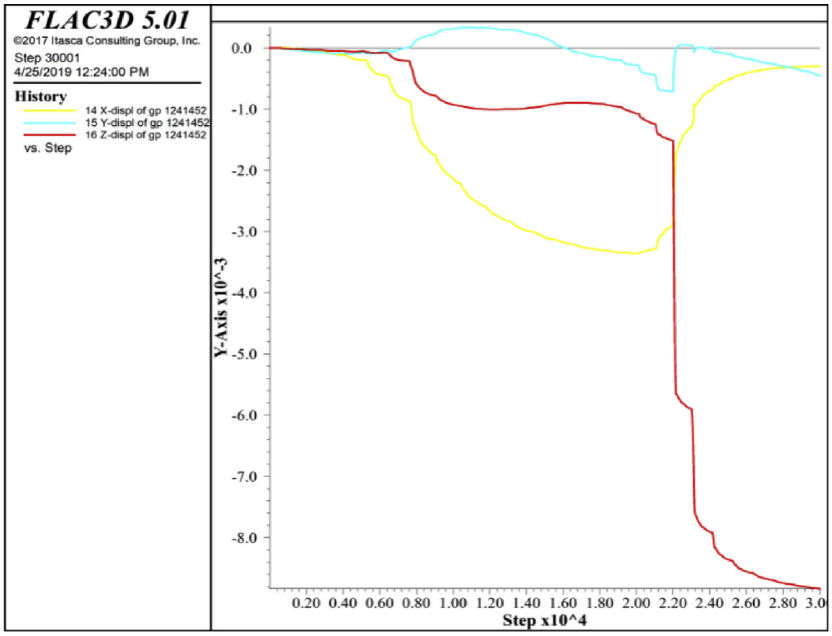
Model boundaries were fixed in all three directions i.e. x-axis, y-axis and z-axis, whereas, the initial in-situ stresses were simulated according to those provided in Table 5.

4.3. Comparision of deformation results

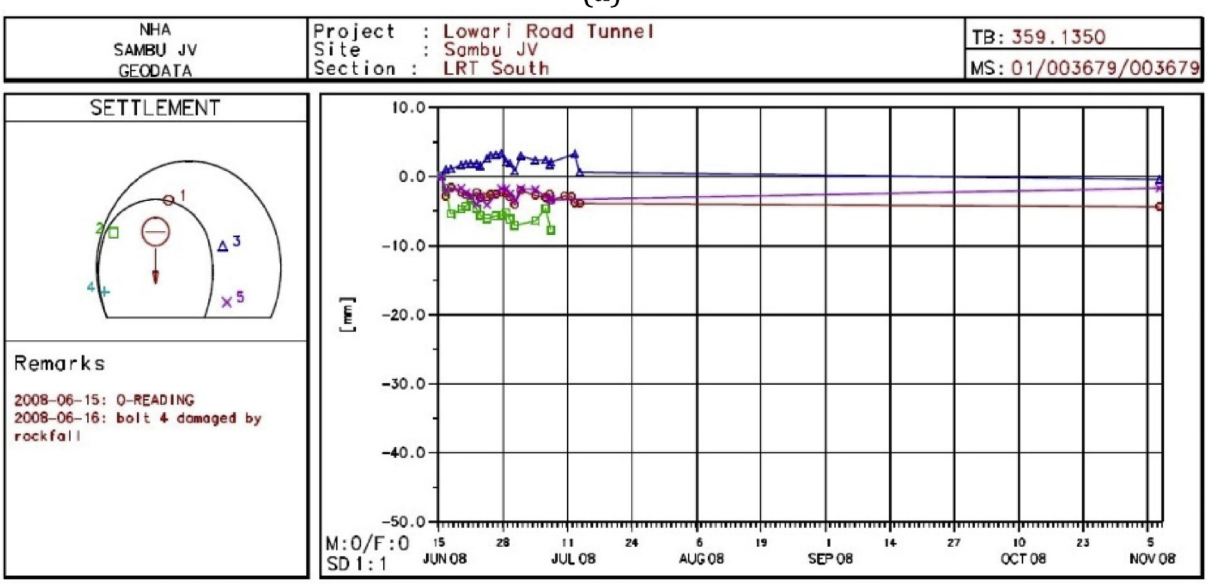
The deformations in vertical direction (*Z-axis*) were recorded at the tunnel crown. The simulation results showed that the tunnel crown deformation in initial tunnel was 8.8 mm at completion of 30,000 time-steps. The actual monitoring was available at tunnel chainage 3 + 679, just 1 m behind the tunnel section under study. According to the actual monitoring, the deformation value of 5.2 mm was observed. The deformation graph of both the simulated deformation and actual recorded deformation are illustrated in Figure 6.

According to Carranza-Torres and Fairhurst [31], the tunnel face deformation is at least one-third of the total deformation. Accordingly, around 10–15% of the total deformation at tunnel periphery was allowed to develop between tunnel face and support approaching the face which was 1.0 m behind the face. The monitoring targets were installed after applying the primary support. Thus, the deformation at any given point at face had already been achieved to be around 35–40% of the total deformation before the installation of monitoring targets.

In this case, the actual deformation was recorded as 5.28 mm. Once the face deformation enhanced up to 40%, the preliminary deformation as aforesaid was observed to be 8.88 mm. However, the simulated deformation was 8.84 mm with a difference of only 0.54% compared to the actual deformation. As both of the results were quite similar, therefore, the FLAC 3D tool and the tunnel widened model could be used effectively for the desired study.



(a)



(b)

Figure 6. Tunnel deformation detail, (**a**). Tunnel deformation by numerical simulation at tunnel crown in red line (*z-vertical direction*), (**b**). Actual monitoring of tunnel deformation.

5. Evaluation of different empirical design approaches for tunnel widening

After the validation of FLAC 3D model to be used in this study, seven empirical design approaches for excavation and support system were analysed for tunnel widening. As the model was extended for 30 m along the tunnel, 15,000 time-steps were assigned for excavation and support installation for the initial tunnel and similar time steps were assigned for the excavation of modified tunnel. The rock mass geological properties observed at the tunnel face at Chainage 3 + 680 as provided in Table 4 was used for the study, whereas, the details of tunnel face characteristics devised from the face mapping of initial tunnel and modified tunnel at Figures 7 and 8 respectively were taken in each case. The tunnel deformation results along with longitudinal displacement profile (LDP) at crown, spring line and invert of tunnel in each empirical design method are combinedly plotted in section 6.

The numerical simulation using each of the seven empirical design approaches is elucidated in subsequent sections.

5.1. Numerical simulation using New Austrian Tunnelling Method (NATM)

During the construction of Lowari Tunnel both initial and modified excavations were based on the NATM principles [32]. Therefore, the first trial for numerical evaluation in this study was conducted with NATM method of tunnel construction, wherein, the Rock Mass Type (*RMT*) and Rock Mass Behaviour Type (*RBT*) were based on the guidelines of Austrian Society for Rock Mechanics [21]. The rock mass geological

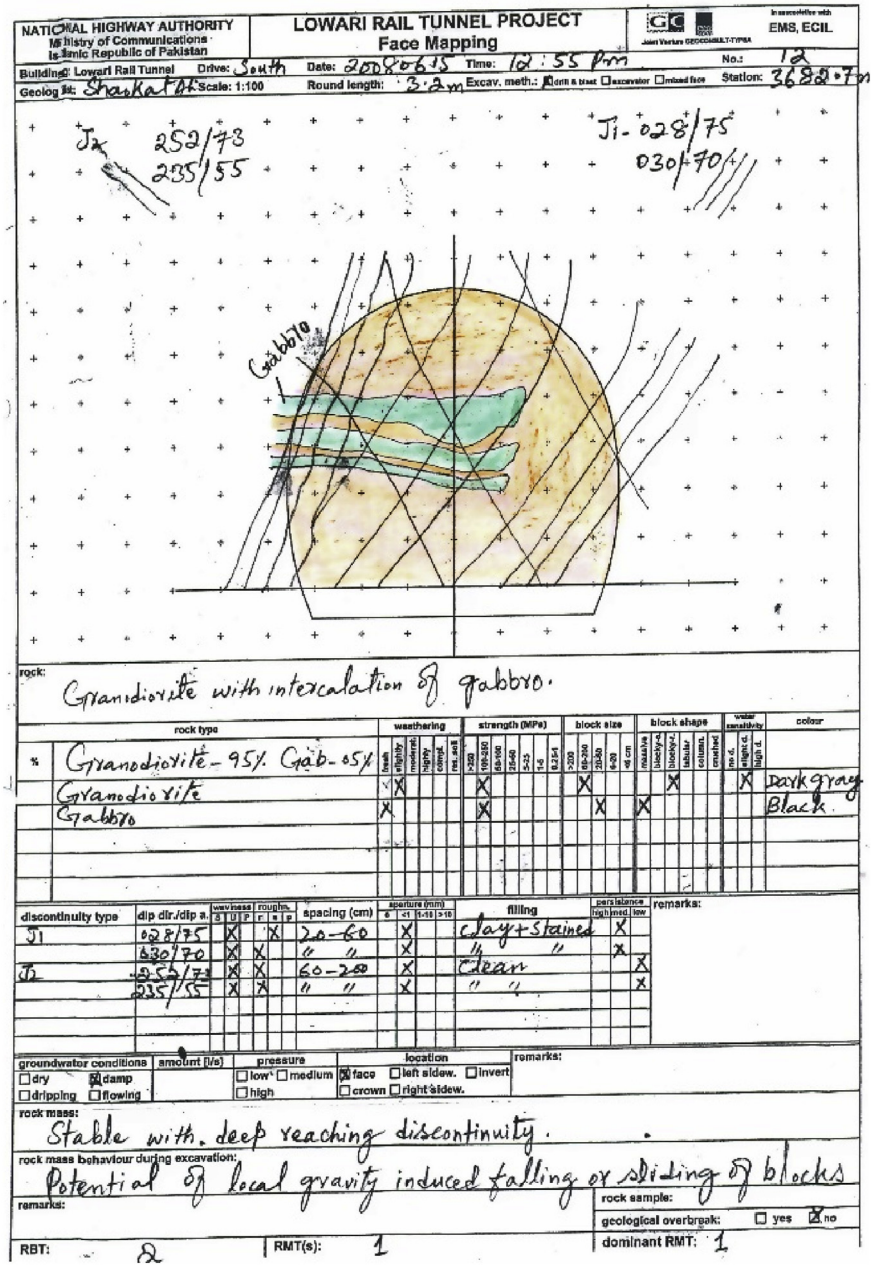


Figure 7. Geological Mapping of initial excavated tunnel at chainage 3 + 682.

properties observed at the tunnel face at Chainage 3 + 680 and support system devised are given in Table 6.

Lowari tunnel was executed for 2.5 m and 2.0 m excavation drive for the initial tunnel and modified tunnel, respectively. Therefore, the model was prepared such that the excavation steps for initial tunnel were controlled through Fish code in FLAC 3D for 2.5 m advance excavation drive, wherein, the support system comprising of shotcrete and rock bolts was installed in each excavation drive. Furthermore, each of the excavation drive with support system was evaluated with 1,250 time-steps such that 15,000 steps were achieved in initial tunnel excavation. Accordingly, the excavation drive length for modified tunnel was adjusted for 2.0 m, wherein, the removal/dismantling of partial to one-third of the primary support system and installation of new support system for widened profile of tunnel were modeled with 1,000 time-steps to achieve 15,000 steps for excavation of completed section of the modified tunnel. The 3D simulation after the installation of support system at the end of the excavation of modified tunnel section is shown in Figure 9.

5.2. Numerical simulation using rock mass rating (RMR_{14})

The Rock Mass Rating (*RMR*) was first presented by Bieniawski [2] and thereafter it was optimised as RMR_{89} [33]. Over the time, the method was further improved on the basis of data collection from different tunnels all over the world, with the latest work being done by Celada et al. [34] which is acknowledged as "RMR_{14}". The RMR_{14} as defined by Celada et al. [34] is expressed in Eq. (6).

$$RMR_{14} = (RMR_b + F_0)\, F_e\, F_s \tag{6}$$

where,

RMR_b = basic rock mass rating without adjustment factors.
F_0 = adjustment factor for tunnel excavation orientation.
F_e = adjustment factor to incorporate the tunnel excavation method.

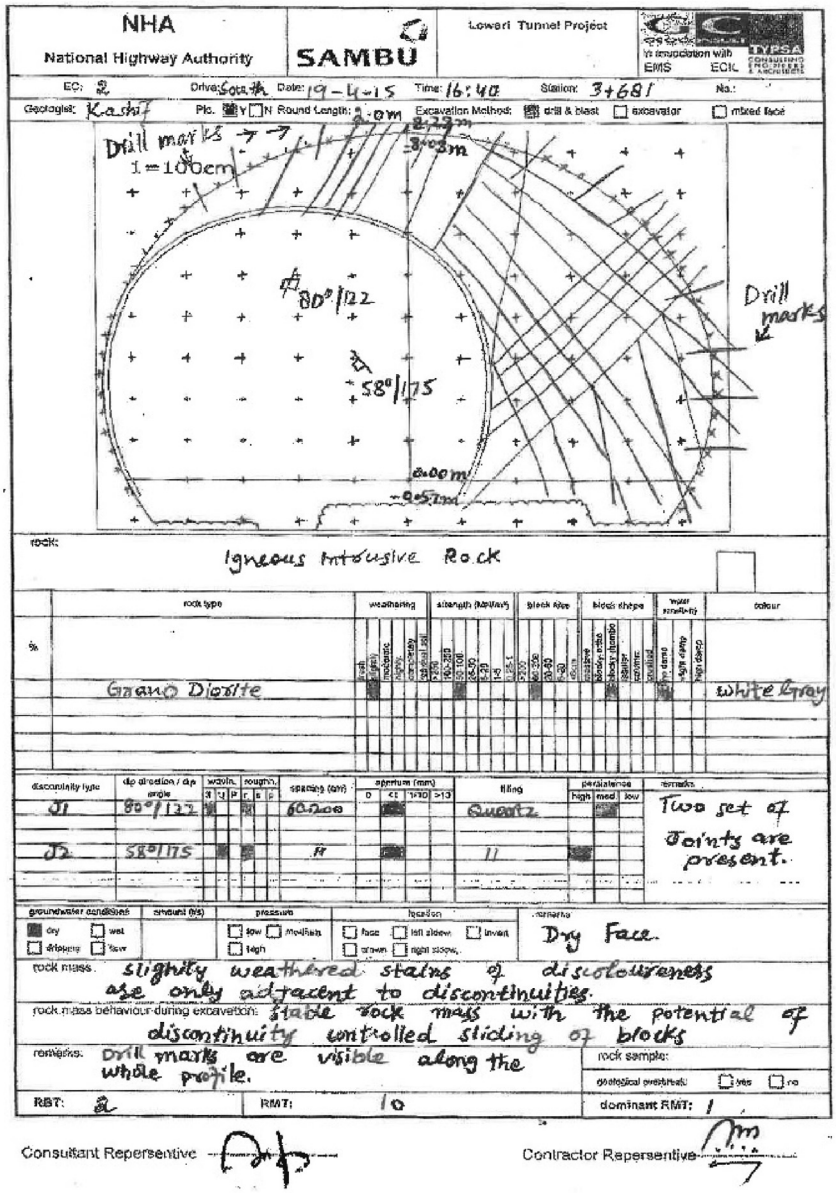


Figure 8. Geological Mapping of modified excavated tunnel at chainage 3 + 681.

Table 6. Estimation of Tunnel Excavation and Support System on the basis of NATM.

Description	Initial Tunnel	Modified Tunnel
Tunnel Face Characteristics		
Rock Mass Type (*RMT*) in accordance with Table 1	1	2
Rock Behaviour Type (*RBT*) in accordance with Table 2	2	2
Excavation Class (*EC*) in accordance with Table 3	2	2
Round Length of Excavation	2.5 m	2.0 m
Type of Excavation Profile	Full Face	Full Face
Tunnel Support Characteristics		
Shotcrete	50 mm	150 mm
Wire Mesh	Grade-60 (*150 × 150 × 6 mm*) at full perimeter	Grade-60 (*150 × 150 × 6 mm*) at full perimeter
Lattice Girders	-	-
Rock Bolts	6 nos. in circumference/length = 4.0 m (*2.5 m long spacing, 2.5 m circumferential spacing*)	10 nos. in circumference/length = 4.0 m (*2.0 m long spacing, 2.0 m circumferential spacing*)

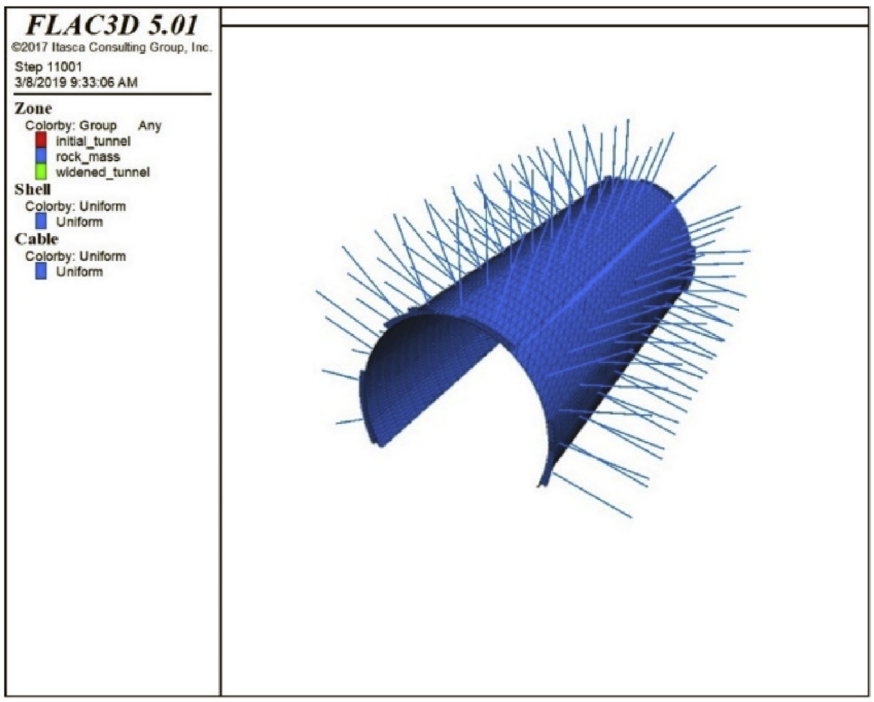


Figure 9. 3D Simulation model at the end of excavation with scale of 1–440.

Table 7. Ratings evaluated as per RMR_{14} and valued as per RMR_{89}.

RMR_{14} - Parameters	Description	Values	Rating as per RMR_{14} [34]
Initial Tunnel			
R_1	Intact Rock Strength (*UCS*)	150 MPa	11
R_2	Rock Quality Designation (*RQD*) and Spacing of Discontinuities	2 discontinuities per meter	29
R_3	Condition of Discontinuities	Persistence 3–10 m, rough, soft filling <5 mm, moderately weathered	10
R_4	Ground Water Rating	Damp	10
R_5	Intact Rock Alterability Id_2 (%)	Id_2 if greater than 85 as the rock is less prone to slake	10
RMR-basic for Initial Tunnel			70
F_0 (*dipping at average of 80° with drive*)			0
F_e (*excavation through blasting*)			1.0
F_s ($ICE = 28.7$)			1.25
RMR_{14} – Initial Tunnel (As per Eq 6)			87.50
RMR_{89} (As per Eq 8)			77.73
Modified Tunnel			
R_1	Intact Rock Strength (*UCS*)	140 MPa	10
R_2	Rock Quality Designation (*RQD*) and Spacing of Discontinuities	3 discontinuities per meter	28
R_3	Condition of Discontinuities	Persistence 3–10 m, rough, soft filling <5 mm, moderately weathered	10
R_4	Ground Water Rating	Dry to damp	10
R_5	Intact Rock Alterability Id_2 (%)	Id_2 is around 60–85 as the rock was moderately prone to slake	8
RMR-basic for Modified Tunnel			66
F_0 (*dipping at average of 80° with drive*)			0
F_e (*excavation through blasting*)			1.0
F_s ($ICE = 22.67$)			1.27
RMR_{14} – Modified Tunnel (As per Eq 6)			83.82
RMR_{89} (As per Eq 8)			74.38

Table-8. The Rock Mass Class and Support System as defined by Bieniawski [33].

Rock Mass Class (RMC)	Excavation	Rock Bolts (20mm *diameter, fully grouted*)	Shotcrete	Steel Set
RMC-II – Good Rock RMR: 61-80	Full Face, 1–1.5 m advance, complete support from 20-m face	Locally bolts in crown 3 m long, spaced 2.5 m with occasional wire mesh	50 mm in crown where required	None

Table 9. Estimation of rock mass quality – (*Q-System*) and tunnel rock support.

Parameter Description	Symbol	Parameters for Initial Tunnel	Parameters for Modified Tunnel
Estimation of Rock Mass Quality (Q)			
Rock Quality Designation	*RQD*	70	65
Joint Set Number	J_n	6 (*two joint set plus random*)	6 (*two joint set plus random*)
Joint Roughness Number	J_r	3 (*rough and irregular, undulating*)	2 (*smooth undulating*)
Joint Alteration Number	J_a	2 (*slightly altered joint walls*)	3 (*small quartz fraction*)
Joint Water Reduction Factor	J_w	0.66 (*medium inflow*)	0.66 (*medium inflow*)
Uniaxial Compressive Strength	σ_c	150	150
Maximum Principle Stress	σ_1	68	68
UCS and Principle Stress Ratio	σ_c/σ_1	2.2	2.2
Strength Reduction Factor	*SRF*	12	12
Rock Quality Index (As per Eq 9)	*Q*	0.96	0.397
Span Length (*at spring line*)	*B*	6.0 m	11.0 m
Equivalent Support Ratio	*ESR*	1.0	1.0
Equivalent Dimension	D_e	6.0	11.0
Tunnel Support Estimate through Q – System			
Reinforcement Categories		5	6
Shotcrete		-	-
Fiber Reinforce Shotcrete		7 cm	12 cm
Steel Arches		-	-
Rock Bolts Length		2.5 m	3.0 m
Rock Bolts Spacing		@ 1.7 m–1.5 m c/c	@ 1.5 m c/c
Length of Excavation Drive		1.5 m	1.5 m
Optimised Length of Rock Bolt [37] "$L = 2 + \frac{0.15B}{ESR}$"		2.9 m–3.0 m	3.65 m–4.0 m

Table 10. Estimation of rock mass index – (*RMi*) and tunnel rock support.

Parameter Description	Symbol	Parameters for Initial Tunnel	Parameters for Modified Tunnel
Estimating Rock Mass Index (RMi)			
Uniaxial Compressive Strength of Intact Rock Mass	σ_c	150 MPa	150 MPa
Block Volume	V_b	0.35 m^3 (*discontinuity spacing at 0.7 m*)	0.15 m^3 (*discontinuity spacing at 0.5~0.6 m*)
Joint Size Factor	*jL*	0.75 (*long jointed; 1–10 m*)	1.0 (*long jointed; 10–30 m*)
Joint Roughness	*jR*	2 (*slight undulated & rough*)	2 (*slight undulated & rough*)
Joint Alteration	*jA*	1 (*staining*)	1 (*staining*)
Joint Condition Factor	*jC* ($jC = jL.jR/jA$)	1.5	2.0
Factor in Jointing Parameter Equation	*D* ($D = 0.37\ jC\text{-}0.2$)	0.341	0.322
Jointing Parameter	*JP* ($JP = 0.2\sqrt{jC.\ VbD}$)	0.171	0.154
Interlocking Adjustment Factor	*IL*	1.3 (*very tight structure*)	1.0 (*tight structure*)
Rock Mass Index ($RMi = \sigma_c.\ JP.\ IL$)	*RMi*	33.34 (*Very High*)	23.10 (*Very High*)
Tunnel Support Estimate through RMi			
Joint Pattern/Set Number	*Nj*	1.5 (*2 joint sets*)	1.2 (*2 joint sets + random joints*)
Orientation of Main Joint Set	C_o	1 (*very favorable*)	1 (*very favorable*)
Gravity Adjustment Factor for Support	*C*	1 (*roof*)	1 (*roof*)
Joint Smoothness	j_s	1.5 (*rough or irregular*)	1.25 (*rough*)
Joint Undulations	j_w	2 (*moderately undulating*)	2 (*moderately undulating*)
Joint Alteration	j_a	2 (*slightly weathered joint walls - stained*)	2 (*slightly weathered joint walls - stained*)
Ground Water Condition	*GW*	1 (*dry or damp*)	1 (*dry or damp*)
Rock Stresses Around the Tunnel	S_L	1.5 (*high stress level*)	1.5 (*high stress level*)
Tunnel Diameter	D_t	6.0 m	11.0 m

(*continued on next page*)

Table 10 (*continued*)

Parameter Description	Symbol	Parameters for Initial Tunnel	Parameters for Modified Tunnel
Equivalent Block Diameter	D_b ($D_b = 3\sqrt{V_b}$)	0.70 m	0.53 m
Size Ratio	Sr $Sr=(D_t/D_b).(C_o/N_j)$	5.71	17.30
Ground Condition Factor	G_c $G_c=RMi\ (S_L.C.G_w)$	50.01	34.65
Tunnel Support			
Shotcrete		-	50 mm
Rock Bolts Spacing		3.0 m	2.5 m
Rock Bolts Length (*As the length is not specified in the support chart, therefore, the support is assumed to be 3.0 m*)		3.0 m	3.0 m

Table 11. Estimation of rock structure rating – (*RSR*) and tunnel rock support.

Parameter Category	Description	Parameters for Initial Tunnel		Parameters for Modified Tunnel	
		Characteristics	Rating	Characteristics	Rating
Estimating of Rock Structure Rating (RSR)					
Parameter-A	General area geology	Igneous, medium rock (*type-2*), slightly folded or faulted	20	Igneous, medium rock (*type-2*), moderately folded or faulted	13
Parameter-B	Joint pattern, direction of drive	Moderately to blocky, 1–2 ft, perpendicular to tunnel axis & with dip	32	Moderately to blocky, 1–2 ft, perpendicular to tunnel axis & with dip	32
$A + B$	-	-	52	-	45
Parameter-C	Ground water, joint condition	Slight <500 gpm	19	Slight <500 gpm	19
$A + B + C$	-	-	71	-	64
Tunnel Reinforcement and Support System					
Excavation Drive		3.0 m		2.0 m	
Shotcrete		1.5 inch–40 mm		2.0 inch–50 mm	
Rock Bolts Spacing		-		6.0 ft–1.8 m ~ 2.0 m	
Rock Bolts Length (*As the length is not specified in the support chart, therefore, the support is assumed to be 3.0 m*)		-		3.0 m	

Table 12. Estimation of geological strength index (GSI) and tunnel rock support.

Parameter Description	Symbol	Parameters for Initial Tunnel	Parameters for Modified Tunnel
Estimating Geological Strength Index (GSI)			
Structure	-	Blocky, well interlocked undisturbed rock mass consist of cubical blocks formed by two to three intersecting discontinuity sets	Blocky, well interlocked undisturbed rock mass consist of cubical blocks formed by three intersecting discontinuity sets
Surface Conditions	-	Rough, slightly weathered, iron stained surface	Smooth to rough, moderately weathered, iron stained surface
Geological Strength Index from the chart defined by Hoek	GSI	65	60
Tunnel Support Estimate using RMR			
Rock Mass Rating (RMR_{89}) ($RMR_{89} = GSI + 5$)		70	65
Rock Mass Class		II	II
Excavation Drive		1.25 m	1.25 m
Shotcrete		50 mm	50 mm
Rock Bolts Spacing		2.5 m	2.5 m
Rock Bolts Length		3.0 m	3.0 m
Tunnel Support Estimation using Q-System			
Rock Quality Index (Q) $Q = e^{[(RMR-44)/9]}$		17.97–18	10.31–10
Category		3	4
Excavation Drive		2.0 m	2.0 m
Shotcrete		-	50 mm
Rock Bolts Spacing		2.0 m (*circumferential*)	2.3 m (*circumferential*)
Tunnel Width (B)		6.0 m	11.0 m
ESR (*Excavation Support Ratio*)		1	1
Rock Bolts Length ($L = 2 + \frac{0.15B}{ESR}$)		3.0 m	4.0 m

Table 13. Estimation of basic quality index – (*BQ*) and tunnel rock support.

Parameters Description	Symbols	Parameter for Initial Tunnel	Parameter for Modified Tunnel
Unconfined Compressive Strength	σc	150 MPa	140 MPa
Volumetric joint count as per [4]	Jv	3	4
Integrity coefficient of the rock mass $K_v = 0.943e(-0.025J_v)$	Kv	0.81	0.76
Basic Quality Index ($BQ=90+3\sigma_c+250K_v$)	BQ	742.5	700
Correction factor for ground water	K_1	0 (*dripping state*)	0 (*dripping state*)
Correction factor for controlled key joint attitude	K_2	0.5 (*joint dipping angle between 50~75°*)	0.5 (*joint dipping angle between 60~75°*)
Correction factor for the initial stress state	K_3	0.5 (*region of high stress*)	0.5 (*region of high stress*)
Corrected BQ $[BQ]=BQ-100(K1+K2+K3)$	$[BQ]$	642.5	600
Grade (*from classification reference table of the BQ method*)	-	I	I
Tunnel Support Estimation using Q-System			
RMR ($BQ=80.786 + 6.0943$RMR)		92.17	85.20
Rock Mass Class as per RMR		I (*very good rock, RMR: 81–100*)	I (*very good rock, RMR: 81–100*)
Excavation Drive		3.0 m (*full face*)	3.0 m (*full face*)
Shotcrete		-	-
Rock Bolts		-	-
Lattice Girders		-	-

F_s = adjustment factor to incorporate the stress-strain behaviour of rock mass at tunnel face.

As per Celada et al. [34] the relation between RMR_{14} and RMR_{89} is correlated in Eq. (7).

$$RMR_{14} = 1.1\, RMR_{89} + 2 \tag{7}$$

After rearranging Eq. (7), the RMR_{89} becomes:

$$RMR_{89} = \frac{(RMR_{14} - 2)}{1.1} \tag{8}$$

The RMR_{14} and RMR_{89} are evaluated as given in Table 7.

The RMR_{89} for initial tunnel as well as modified tunnel excavation fell in the range of 61–80, which according to Bieniawski [33] is categorized under Rock Mass Class "II", wherein, the support was defined for the said range is summarized in Table 8.

The model geometry was generated as defined in section 4.2 and illustrated in Figure 9. The support was applied as per Table 8. The excavation steps were controlled for the excavation drive of 1.5 m, however, as defined the bolts were installed at every 1.5 m. The time step at each drive was kept 750 so that, at excavation of both initial and modified tunnel got evaluated for 15,000-time steps, thus the results for deformation were obtained for the number of time-steps up to a total of 30,000.

5.3. Numerical simulation using rock mass quality (Q-system)

The rock mass quality mostly famous with *Q-system* was first introduced by Barton et al. [3] in 1974 at the Norwegian Geotechnical Institute (NGI) and upgraded through characterization and classifications in 1993 by Grimstad [35]. This was further optimised by Barton [36]. *Q-system* was based on six parameters, each having specialized characteristic and rating. The *Q* values are based on semi-log and values range between 0.001 (*being an exceptionally poor-quality rock*) to 1000 (*being an exceptionally good quality rock*). The equation as defined by Barton [3] is given below:

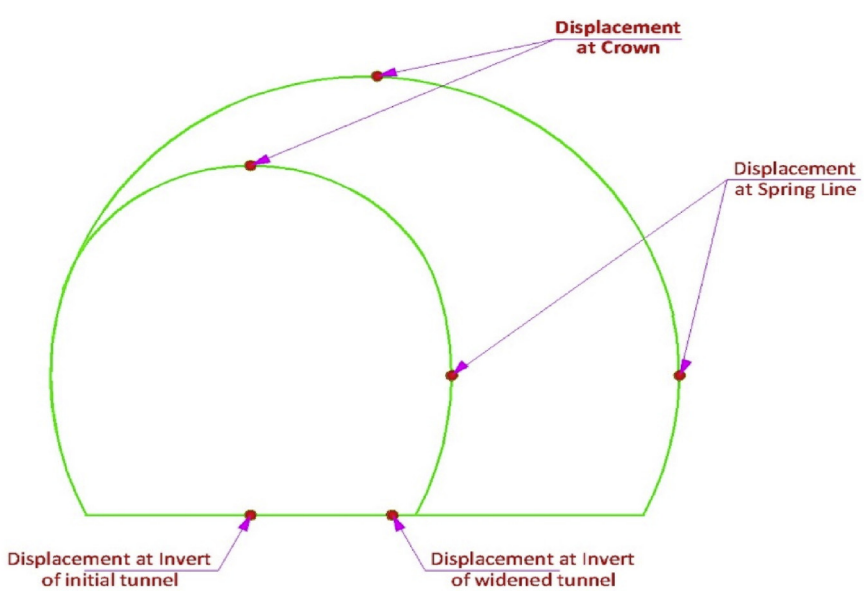


Figure 10. Tunnel deformation monitoring locations.

$$Q = \frac{RQD}{J_n} \quad \frac{J_r}{J_a} \quad \frac{J_w}{SRF} \tag{9}$$

Whereas:

RQD = Rock quality designation.
J_n = Joint set number.
J_r = Joint roughness number.
J_a = Joint alteration number.
J_w = Joint water reduction factor.
SRF = Stress reduction factor.

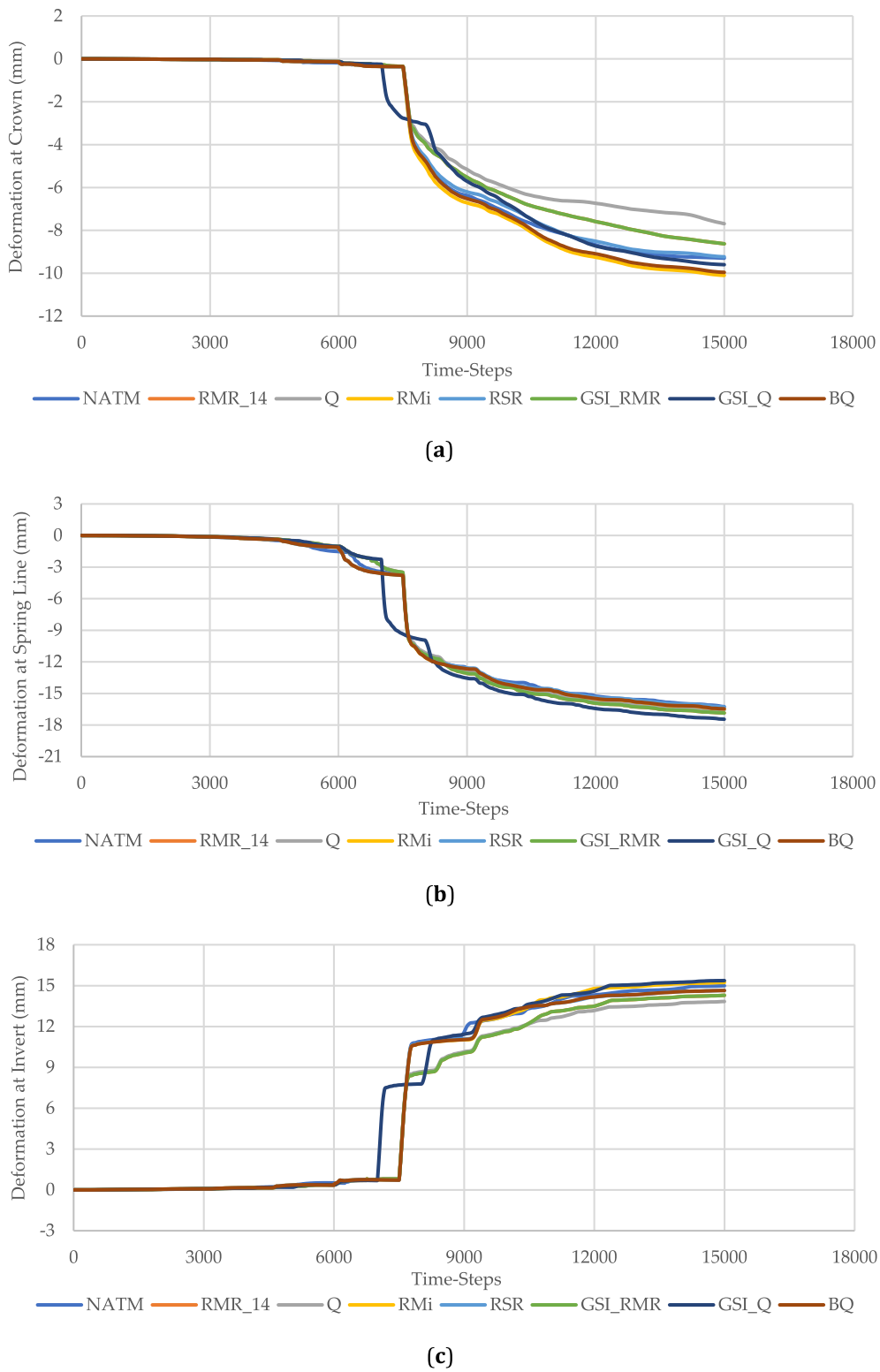


Figure 11. First part of the first phase - rock mass behaviour of different empirical design approach at excavation of initial tunnel with 15,000 time-steps: (**a**) At tunnel crown; (**b**) At tunnel spring line; (**c**) At tunnel Invert.

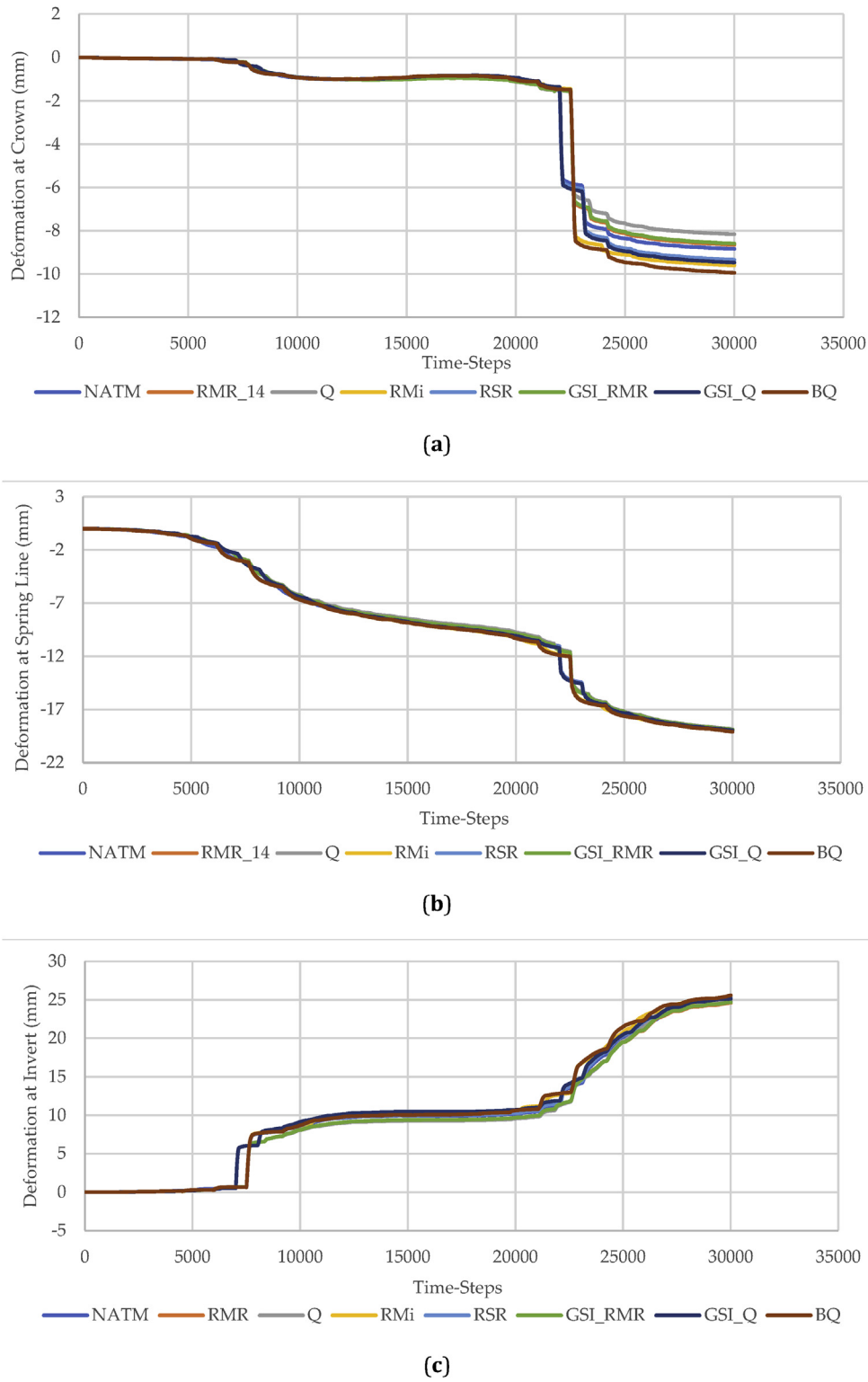


Figure 12. Second part of the first phase - rock mass behaviour of different empirical design approach at excavation of initial and modified tunnel at completion of 30,000 time-steps: (**a**) At tunnel crown; (**b**) At tunnel spring line; (**c**) At tunnel invert.

The estimation and support under *Q-system* is given in Table 9.

The tunnel model was prepared as elucidated in section 4.2. The excavation drive was fixed for 1.5 m both for initial and modified tunnel, which were controlled through Fish code in FLAC 3D, wherein, the support system, comprising of shotcrete and rock bolts were installed in each excavation drive with 750 time-steps. As the model was extended for 30 m along the tunnel, 15,000 time-steps were assigned for excavation and support installation for the initial tunnel and similar time steps were assigned for the excavation of modified tunnel such as to get the final deformation values after the total number of time-steps of 30,000.

Table 14. Comparison of tunnel deformation between initial and modified tunnel.

S. No.	Empirical Design Approach	(First Part) Initial Tunnel Deformations (mm) (15,000 steps)			(Second Part) Modified Tunnel Deformations (mm) (30,000 steps)			Percentage Difference (%)		
		Crown	Spring	Invert	Crown	Spring	Invert	Crown	Spring	Invert
a.	*NATM*	9.29	16.25	14.99	8.84	18.92	24.91	-4.8%	16.4%	66.2%
b.	RMR_{14}	8.63	16.85	14.29	8.65	18.91	24.62	0.2%	12.2%	72.3%
c.	*Q-System*	7.69	16.05	13.54	8.16	18.85	24.59	6.1%	17.4%	81.6%
d.	RM_i	10.10	16.47	15.23	9.60	18.97	25.34	-5.0%	15.2%	66.4%
e.	*RSR*	9.23	16.28	14.65	9.34	18.98	25.31	1.2%	16.6%	72.8%
f.	*GSI_RMR*[1]	8.63	16.85	14.29	8.60	18.88	24.68	-0.3%	12.0%	72.7%
g.	*GSI_Q*[2]	9.60	17.45	15.37	9.46	18.97	25.13	-1.5%	8.7%	63.5%
h.	*BQ*	9.96	16.46	14.64	9.94	19.10	25.60	-0.2%	16.0%	74.9%

[1] *Support system with RMR.*
[2] Support system with Q.

5.4. Numerical simulation using Rock Mass Index (RMi)

The Rock Mass Index was introduced in 1995 by Arild Palmstrom and finally published through [4] by considering the three dimensional block volume.

The estimation and support under *RMi*-System are stipulated in Table 10.

The excavation drive was fixed for 3.0 m and 2.5 m for initial tunnel and modified tunnel, respectively, which was controlled through Fish code in FLAC 3D, wherein, the support system comprising of shotcrete and rock bolts were installed in each excavation drive. 1,500 time-steps were distributed in initial tunnel excavation drive, whereas, 1,250 time-steps were distributed for modified tunnel such that 15,000 time-steps achieved after the excavation of 30 m initial tunnel and same quantity of time-steps achieved at excavation of 30 m for modified tunnel. Thus, the final deformations were obtained at execution of total time-steps of 30,000.

5.5. Numerical simulation using Rock Structure Rating (RSR)

This quantitative method was defined by Wickham [5] to establish the classification of Rock Structure Rating (RSR). The estimation and support under *RSR*-System is stipulated in Table 11.

The excavation drive was fixed for 3.0 m and 2.0 m for initial tunnel and modified tunnel, respectively, which was controlled through Fish code in FLAC 3D, wherein, the support system comprising of shotcrete and rock bolts were installed in each excavation drive. 1,500 time-steps were distributed in initial tunnel excavation drive, whereas, 1,000 time-steps were distributed for modified tunnel so that 15,000 time-steps achieved after the excavation of 30 m initial tunnel and same quantity of time-steps achieved at excavation of 30 m for modified tunnel. The face deformation results were collected at execution of accumulated time-steps of 30,000.

5.6. Numerical simuation using Geological Strength Index (GSI)

The Geological Strength Index (*GSI*) was introduced by Hoek [6] as a descriptive approach which provides a numerical value for the rock mass physical appearance. The estimation of *GSI* was opted from the chart defined by Hoek et at [38]. However, the *GSI* system does not define the tunnel reinforcement and support system. To estimate the support, a correlation was extracted from *GSI* to *RMR* and *Q-system* to define the tunnel support system.

The values of *GSI* using the chart defined by Hoek et at [38]. brought the values of 65 for initial tunnel and 60 for small tunnel. Hoek and Brown [28] has defined the correlation between *RMR* and *GSI* as given in Eq. (10).

$$RMR_{89} = GSI + 5 \tag{10}$$

The results obtained for RMR_{89} was 70 for initial tunnel and 65 for the modified tunnel. Accordingly, the support system was calculated using RMR_{89} as elucidated in Table 12. Simultaneously, the values of *GSI* were converted to *Q-System* using RMR_{89} from Eq. (10). Bieniawski [33] has established the correlation between *RMR* and *Q-system* as given in Eq. (11).

$$RMR = 9 \ln Q + 44 \tag{11}$$

Similarly, Barton [39] proposed the relationship between RMR and Q-system as given in Eq. (12).

$$RMR = 15 \log Q + 50 \tag{12}$$

The values of *Q* were obtained both from Eqs. (11) and (12) which are stipulated in Table 12, however, the values obtained from Eq. (11) provides conservative values, therefore, the same was used in estimation of support system. The reinforcement and support system defined from *Q-system* is elucidated in Table 12. The excavation drive was fixed for 2 m for both initial and modified tunnel as per respective design guide lines, which was controlled through Fish code in FLAC 3D wherein, the support system comprising of shotcrete and rock bolts, that were installed in each excavation drive. 1,000 time-steps were distributed both for initial and modified tunnel excavation drive so that 15,000 time-steps were achieved after the excavation of 30 m initial tunnel and same quantity of time-steps were achieved at excavation of 30 m for modified tunnel. Thus, the final deformation values were obtained at execution of total time-steps of 30,000.

5.7. Numerical simulation using Basic Quality Index (BQ)

The Basic Quality (*BQ*) Method was proposed by "*The National Standards Compilation Group of People's Republic of China*" and adopted as Chinese basic national standard for engineering classification for rock mass GB 50218-94 [7]. The BQ system for widening of tunnel was evaluated as per the guidelines defined by Feng and Hudson [40].

Wenki Feng at al. [41] analysed more than 200 sets of *BQ* and *RMR* values via regression analysis based on different sites and found the relation as given in Eq. (13).

$$BQ = 80.786 + 6.0943RMR \tag{13}$$

The values of *RMR* were obtained from Eq. (13) which are stipulated in Table 13. As per *RMR*, both of the tunnel excavations i.e. initial

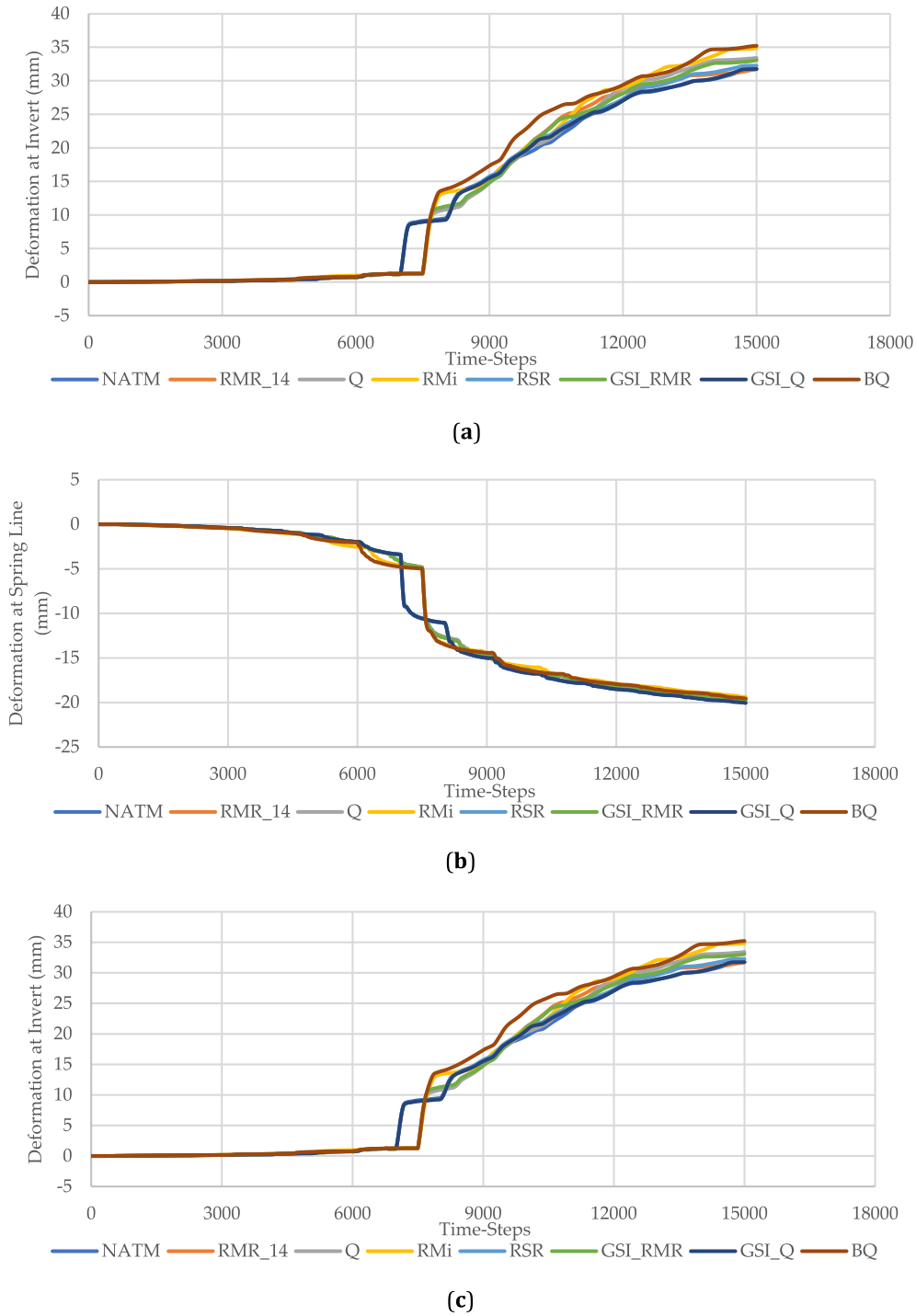


Figure 13. Rock mass behaviour after excavation of single enlarge tunnel: (**a**) At tunnel crown; (**b**) At tunnel spring line; (**c**) At tunnel invert.

tunnel and modified tunnel fell under Rock Mass Class-I, wherein, no support was required as per respective design guide lines. The excavation drive of 3.0 m was fixed for both initial tunnel and modified tunnel which were controlled through Fish code in FLAC 3D. 1,500 time-steps were distributed both for initial tunnel and modified tunnel excavation drive so that 15,000 time-steps were achieved after the excavation of 30 m initial tunnel and same quantity of time-steps achieved for excavation of 30 m of modified tunnel. Thus, final tunnel deformations were obtained after the execution of accumulative 30,000 time-steps.

6. Results & discussion

The rock mass deformations were obtained through numerical simulation by opting Finite Difference Method (FDM) using FLAC 3D after evaluating the excavation and support design from different empirical approaches commonly in practice as elucidated in Section 5. In this regard, two separate activities were carried out in two associated phases, wherein, the first phase was carried out to compare the tunnel deformation between initial and modified tunnels, when they were excavated one after another. The second phase was carried out to

Table 15. Comparison of Tunnel Deformation between tunnel profile achieved through consecutive excavation of initial and modified tunnel and excavation of single enlarge tunnel.

S. No.	Empirical Design Approach	(First Part) Modified Tunnel Deformations (mm) (30,000 steps)			(Second Part) Large Tunnel Excavation Deformations (mm) (15,000 steps)			Percentage Difference (%)		
		Crown	Spring	Invert	Crown	Spring	Invert	Crown	Spring	Invert
a.	*NATM*	8.84	18.92	24.91	9.72	20.03	31.72	10.0%	5.9%	27.3%
b.	RMR_{14}	8.65	18.91	24.62	9.70	19.70	31.77	12.1%	4.2%	29.0%
c.	*Q-System*	8.16	18.85	24.59	9.32	19.98	34.42	14.2%	6.0%	40.0%
d.	RM_i	9.60	18.97	25.34	10.38	19.38	34.85	8.1%	2.2%	37.5%
e.	*RSR*	9.34	18.98	25.31	10.45	20.01	32.26	11.9%	5.4%	27.5%
f.	*GSI_RMR*[1]	8.60	18.88	24.68	9.70	19.72	33.08	12.8%	4.4%	34.0%
g.	*GSI_Q*[2]	9.46	18.97	25.13	10.23	20.06	31.76	8.1%	5.7%	26.4%
h.	*BQ*	9.94	19.10	25.60	10.82	19.56	35.05	8.9%	2.4%	36.9%

[1] Support system with RMR.
[2] Support system with Q.

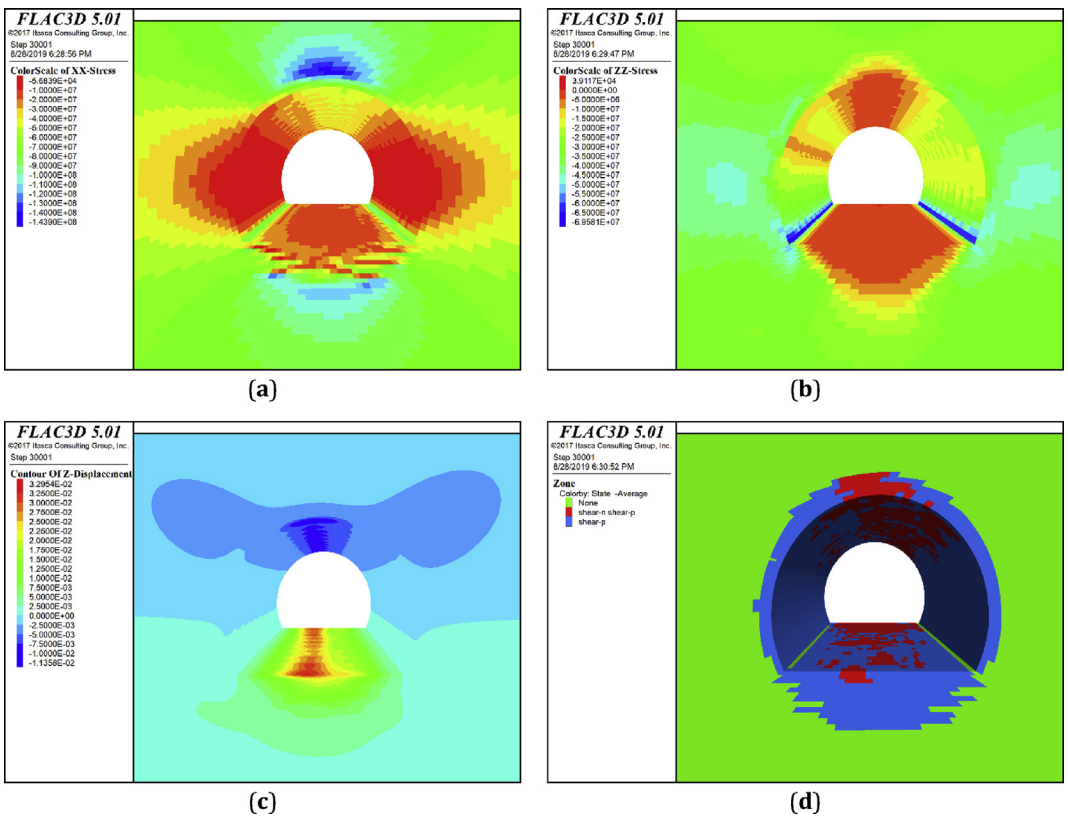


Figure 14. Rock mass behaviour around the tunnel after simulation through Rock Mass Quality (Q) Method; a) Horizontal stress distribution; b) Vertical stress distribution; c) Vertical displacing around the tunnel; d) Plastic zone created around the tunnel.

compare the deformation of modified tunnel with the deformation obtained from the single stage excavation of enlarge tunnel. The details of both analyses are elucidated in subsequent sections.

In each phase, the deformation was monitored at crown, spring line, invert at middle of model length (*i.e. 15 m from face*), as illustrated in Figure 10.

6.1. First phase - comparison of initial and modified tunnel deformation

This phase was further split in two parts. The first part comprises the monitoring of tunnel simulated deformation at selected locations after excavation and support installation of 30 m length of initial tunnel model (45 m^2) based on 15,000 time-steps. The longitudinal deformation profile (LDP) after simulation using different empirical approaches at three selected locations (as shown in Figure 10) are illustrated in Figure 11.

In second part of this phase, the tunnel was simulated consecutively; firstly, for initial tunnel excavation profile (45 m^2) with 15,000 time-steps followed by one-third removal of support system and widening of tunnel to get modified tunnel profile (85 m^2) with additional 15,000 time-steps. The monitoring was recorded at selected locations of modified tunnel after the execution of total time-steps of 30,000. The longitudinal deformation profile (LDP) at selected three locations for different empirical design approaches are illustrated in Figure 12. The deformation results of both parts of this phase are summarized in Table 14.

In first part of this phase, the tunnel deformation at selected monitoring points of initial tunnel as given in Table 14 depicted that the Rock Quality Index (*Q system*) Method [3] provides controlled and lowest deformation among the other design approaches. In this part, the excavation of initial tunnel provides the deformation of 7.69 mm, 16.05 mm and 13.54 mm at crown, spring line and invert, respectively, under specific Lowari tunnel site conditions.

In second part, the *Q-system* also provides lowest tunnel deformations among all other options of empirical design as considered in the study. At the modified tunnel profile, the deformations obtained at crown, spring line and invert are 8.16 mm, 18.85 mm and 24.59 mm respectively.

While comparing the deformation of initial tunnel and modified tunnel (*first and second part*), it was found that, the percent difference of deformation variation at crown, spring line and invert were 6.1%, 17.4% and 81.6%, respectively, which were highest among all other empirical design methods. Thus, it exhibited from the percent variation that the tunnel deformation occurred as a result of widening of tunnel can be better controlled through *Q-system* design approach with respect to other empirical design methods.

6.2. Second phase - comparison of two consecutive stage excavation with single stage excavation

In this phase, a comparison was drawn to calculate the percent variation of rock mass deformation between two parts, wherein, the first part comprising of consecutive stage excavation of initial and modified tunnel after evaluation of different empirical approach methods (*final deformation from first phase in section 6.1*). The second part comprising of direct excavation of full large tunnel profile of 85 m^2 with a total of 15,000 time-steps. The rock reinforcement and support systems defined for large profile tunnel are similar to that as defined in Section 5 for modified tunnel in respective tunnel design methods.

The results obtained through the simulation for different design approach are shown in the graph at Figure 13. The table is extracted from the graph at Figure 12 and Figure 13 and summarized in Table 15. The results revealed that, the deformation recorded in first part at the crown, spring line and invert was lowest in *Q-system* design method among the seven different empirical methods as already revealed in first phase in section 6.1; the deformation at crown, spring line and invert was recorded as 8.16 mm, 18.85 mm and 24.59 mm, respectively. In the second part, the results from the single stage enlarge excavation of tunnel recoded as 9.32 mm, 19.98 mm and 34.42 mm at crown, spring line and invert, respectively, which were found again lowest deformation in *Q-system* design method among the others.

While calculating the difference of rock mass deformation between the two parts, the tunnel deformations at crown, spring line and invert was found to be 1.16 mm (14.2%), 1.13 (6.0%) and 9.83 mm (40.0%), respectively which were comparatively less in first part of this phase, which comprises of consecutive excavation of initial and modified tunnel.

6.3. Appropriate method for widening of tunnels

As the study was carried out on different empirical approaches for design of excavation and support system, the Q-System introduced by Barton [3] and further optimised in 2002 [36], was considered appropriate for widening of tunnel in high stress conditions found from two separate phases as carried out in section 6 and 6.2. The method is suitable as it considers the equivalent dimensions of initial tunnel and modified tunnel during definition of the support system. This facility is not available in any other design approaches. Furthermore, the efficacy of *Q-system* is further increased as the method considers the effective in-situ stress and overburdened height in excavation and support design evaluation for modified tunnel. The stress around the tunnel cavity and rock mass deformation obtained after simulation through *Q-System* at execution of 30,000 time-steps are illustrated in Figure 14.

Furthermore, once the initial tunnel is excavated in specific site conditions, then, this can be taken as a pilot tunnel, wherein, all the rock mass characteristics comprising of rock mass behaviour, rock condition, joint pattern are then available. Therefore, it is viable to use the Q-Method instead of any monitoring system as it has been tried in Lowari tunnel.

7. Conclusions

The demand for enlargement of existing underground structures will be definitely required in order to meet the necessary increase in the demand of infrastructure development. In this context, it is mandatory to reduce the knowledge gap in widening the underground structures, in general, and also to develop a deep understanding of the application of the methods available for designing and particularly widening the underground structures. To get through with this phenomenon, the case of the widening of Lowari Tunnel located in Pakistan was opted to be used for simulation of different available empirical design methods in order to examine the rock mass behaviour in each method. The tunnel model based on Lowari Tunnel profile both for initial and widened tunnel was used in FLAC 3D with site specific condition to evaluate the rock mass behaviour using New Austrian Tunnelling Method (*NATM*), Rock Mass Rating (*RMR*), Rock Mass Quality (*Q*), Rock Mass Index (*RMi*), Rock Structure Rating (*RSR*), Geological Strength Index (*GSI*), and Basic Quality Index (*BQ*) Method. Among the different design approaches, the Rock Quality Index (*Q-system*) was found to be the most suitable method for application in the widening of tunnels keeping in view the deformation control of rock mass. The numerical simulation and cost analysis results shows that, this method is more economical as compared to other methods as it suggests optimised reinforcement and support system to be used. Majority of empirical methods ignored the effective tunnel width, in-situ stress conditions and overburden heights which are considered as major elements in tunnel stability and support determination. Only *Q-system*, RMR_{14} and RM_i cater for the stress effect, wherein, *Q-System* was found highly appropriate in this regard. Through this study, it will be of great help to have an appropriate technique in widening of tunnel in high in-situ stress conditions. Furthermore, the size and width of any widened tunnel can be best evaluated in future on the basis of this study.

Declarations

Author contribution statement

Babar Khan: Conceived and designed the experiments; Performed the experiments; Analyzed and interpreted the data; Wrote the paper.

S. Muhammad Jamil: Conceived and designed the experiments; Contributed reagents, materials, analysis tools or data.

Turab H. Jafri: Performed the experiments; Analyzed and interpreted the data; Contributed reagents, materials, analysis tools or data; Wrote the paper.

Kamran Akhtar: Contributed reagents, materials, analysis tools or data.

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Competing interest statement

The authors declare no conflict of interest.

Additional information

No additional information is available for this paper.

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