Clareus Scientific Science and Engineering
Volume 1 Issue 1 June 2024

Analysis and Design of Road Tunnel in Lower Himalayas

Citation: Ankur Chauhan., et al. "Analysis and Design of Road Tunnel in Lower Himalayas". Clareus Scientific Science and Engineering 1.1 (2024): 37-47.

Article Type: Case Study

Received: May 13, 2024

Published: July 15, 2024

Ankur Chauhan[1]*, Altaf Usmani[2] and JT Shahu[3]

[1]*Research Scholar, Dept. of Civil Engineering, Indian Institute of Technology Delhi, Delhi 110016, India*

[2]*Deputy General Manager, Engineers India Limited, New Delhi 110066, India*

[3]*Professor, Dept. of Civil Engineering, Indian Institute of Technology Delhi, Delhi 110016, India*

***Corresponding Author:** Ankur Chauhan, Research Scholar, Dept. of Civil Engineering, Indian Institute of Technology Delhi, Delhi 110016, India.

Abstract

This paper discusses a case history of a highway tunnel located in lower Himalayas. The site information was collected primarily from geological mapping and limited site investigations carried out from tunnel portal due to inaccessible terrain conditions. The tunnel alignment, cross section, and elevations were assessed, evaluated, and designed based on the geological and geotechnical parameters ascertained from field and laboratory investigations. Numerical analysis was carried out to study stress-deformation behavior under varying ground conditions and multiple stages of excavation along with wedge stability for given joint conditions. Displacements obtained numerically were compared along with field monitoring values for crown and wall. Finally, a sensitivity analysis is carried out to ascertain impact of strength parameters on deformation and yielding of excavated tunnel face.

keywords: Numerical analysis; Geotechnical investigations; Geological mapping; Road tunnel; Field monitoring

Introduction

The design and construction practices in mountainous tunnels with varying ground conditions have always been a challenge for the designers and contractors. One of many challenges is to find a way to carry out geotechnical investigations at site due to uneven and harsh terrain conditions and inaccessibility of proposed alignment passing through critical locations. The foremost step to start with these projects is to identify and explore suitable alignment options based on investigations involving geological, geophysical, geo technical, and hydrogeological exploration and available site information from nearby quarries, if any. With the basic principle of establishing road connectivity between strategic and critical areas in hilly regions, road tunnels are an important means to convert large distances into small ones, thereby reducing the travel time alongside ensuring all weather connectivity of these locations. In order to further ensure the most suitable alignment of these tunnels, multiple alignment options are studied and optimized with respect to geotechnical, geological, and hydrological aspects.

This paper presents a case history of a road/highway tunnel project passing through varying ground conditions, located in lower Himalayan mountain ranges, having a maximum and minimum overburden of 1520 m and 50 m respectively, above the proposed tunnel crown level. The proposed tunnel site was explored and investigated by an extensive\site-specific investigation including geological mapping and geotechnical investigations through borehole drillings and laboratory testing. The adopted tunnel geometry is a modified horseshoe shaped cross-section with diameter of around 13 m (Figure 1). The invert portion of the tunnel slopes down towards one portal to allow the drainage during construction and finished stages of tunnel. Initially tunnels support was estimated with empirical tunnel support chart devised by Barton et al. (1993). The tunnel cross-sections were subsequently analyzed using two- dimensional (2D) finite-element code RS2 with the same support system to check the stability of the tunnel in terms of displacements and stresses along with optimization in support as required to stabilize the tunnel cross section (Usmani et al. 2013). Further, these results were cross-verified using discontinuity-controlled stability analysis, to ascertain formation of unstable wedges. Since the tunnel is two laned with provision of bi-directional traffic, the methodology based on the plain strain concept was suitable for these conditions. Further the nominal plain strain analysis can also predict the surface settlements caused by tunnel construction in a suitable manner (Franzius and Potts 2005). However, due to varying ground conditions and weak zones, it was more prone to instability due to intersecting joints and block failures. Therefore, changing ground conditions were analyzed by simulating the different ground types along with structural features like joints, faults etc. in finite element software tool, RS2 to study the overall effect of different ground types, and joints etc.

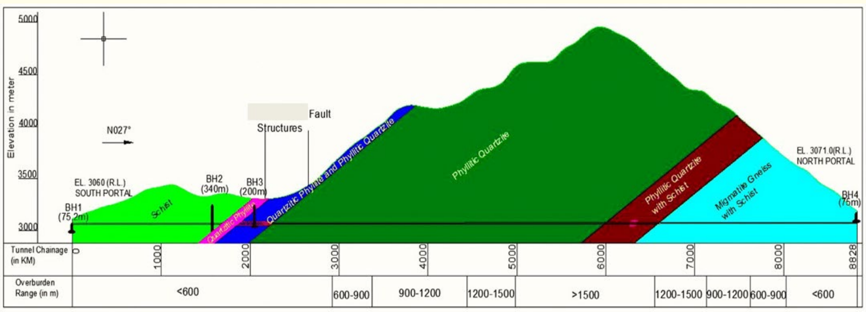

Figure 1: Tunnel layout and geological distribution.

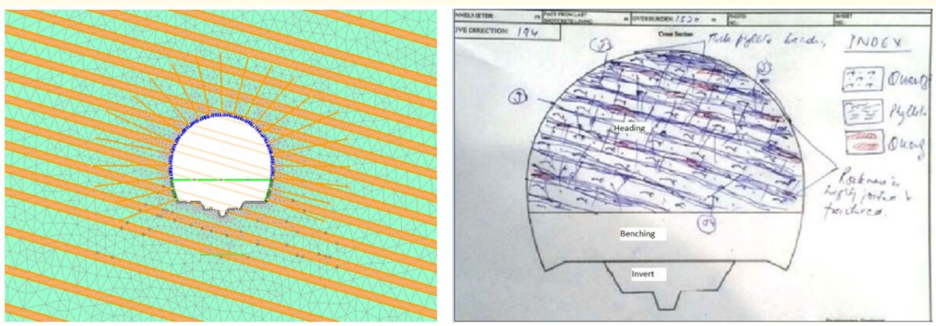

Figure 2: Excavation steps in tunnel section along with quartzite (green) and phyllite (orange) bands.

Site Investigations

The site investigation program included bore hole drilling (BH) at tunnel portal locations where access was available along with laboratory testing. It involved 4 boreholes (BH1, BH2, BH3 and BH4) as shown in Figure 1 of varying depths at portal locations in order to capture the ground information. The geological mapping was carried out extensively at portals and other accessible locations near the tunnel alignment. The rock mass encountered was medium to highly foliated with layers of quartzite and phyllite rocks. The distribution of quartzite and phyllite varies with about 0.5-2 m thick phyllite bands between 2-5 m thick quartzite bands. Thus, mixed face conditions were encountered at tunnel face which made it more complex and challenging. Based on geological mapping, quartzite available at site was found to have average GSI value of around 70 which is why it is referred to as good rock on the face and phyllite with average GSI value of 55 to call it as fair rock, which was used for modelling alternate bands of quartzite (good) and phyllite (fair) rockmass. These foliations introduce a marked anisotropy in the ground and require due attention while implementing the rock support system. Further several probe hole drillings were carried out from tunnel portal and face (Figure 2) to understand the quality of rockmass ahead of the face. This actually helped a lot during tunnel construction to plan for the supports and groundwater mitigation in advance thereby enabling a safe and workable environment in the tunnel. Based on the geological mapping and face logs, tunnel stretch (Figure 1) was divided into multiple ground types showing different behaviors under varying stress conditions, for which suitable support classes were recommended to cater for different ground and behavior types. The design approaches adopted followed both empirical as well as numerical analysis to study the excavation response and arrive at support system. Extensive monitoring plan were also developed and implemented at site in order to accommodate the risk into design and also verify the support system and its suitability in the tunnel. Final lining was installed after a significant period of monitoring and ensuring no further change in instrument readings is observed over a period of time.

Ground elevation along the tunnel stretch varies from an elevation of around of 50.0 m towards the valley area to around 1520.0 m near the deepest portion of the hill. These topographical conditions at site pose a wide range of design and construction related challenges for designers and contractors respectively. The rockmass quality Q system classification (Grimstad and Barton 1993) was adopted for the quantification of supports for the rock mass. Due to high overburden areas, significantly higher horizontal stresses, maximum (S_H) of around 60 MPa with an orientation of N 194° E and a minimum (S_h) of around 40 MPa, which acts normal to S_H, was calculated. The higher magnitudes of stresses are contributing to the fact that these readings correspond to the deepest point of the tunnel in the mountain, and consequently, the irregular topography of the site supports the stress levels and high stress ratios. The geological conditions of the project site consisted of mainly two rock mass layers of differing thicknesses, i.e., the alternate bands of quartzite and phyllite rock masses. Unconfined compression strength (UCS) tests on 20 samples and 5 tensile strength test were carried out on samples selected from four bore holes. Based on the site investigation data results, UCS values adopted in design was 70 MPa (avg. value) for quartzite and around 50 MPa (avg. value) for phyllite rock mass near the tunnel level. The average rockmass density was estimated to be 27 kN/m^3.

Analysis Approach

The stability analysis of tunnel was carried out using finite-element code RS2.0 (RS 2D) along with staged excavation technique and with rock support installation. The major joint sets were also modelled in RS2 to check their impact on excavation stability. Further as per given rockmass conditions, discontinuity-controlled failures were very critical and thus it is crucial to examine the inherent stability of potential wedges and sufficiency of the installed support system. Thus wedge stability analysis was carried out using numerical tool UNWEDGE, which is based on the block theory (Goodman and Shi 1985), for various sections and orientations of the tunnel, assuming the applicable joint sets obtained from site mapping as given in Table 1. Friction angle from shear resistance along joints was considered as 30° for the fair and good rock mass conditions based on the investigation results while negligible cohesion is assumed for both rock mass classes. The rock mass properties were interpreted using generalized Hoek-Brown failure criterion (Hoek and Brown 2018) and same was adopted for carrying out FEM analysis. The criterion adopted is as stated below.

$\sigma_1 = \sigma_3 + \sigma_{ci} [m_b (\sigma_3/\sigma_{ci}) + s]^a$,

where σ_1 and σ_3 are the major and minor principal stresses respectively and m_b = reduced value of material constant given as $m_b = m_i$ exp [(GSI-100)/(28-14D)]

s = exp[(GSI-100)/(9-3D)]

$a = (1/2) + (1/6) \times [e^{-GSI/15} - e^{-20/3}]$,

D= disturbance factor, and

GSI= geological strength index of rockmass.

Different tunnel supporting measures like rock bolts and shotcrete were incorporated sequentially in the FEM model to control the tunnel lining displacements within allowable limits. Various analyses were carried out considering different cross sections and conditions for the tunnel to verify its stability. This paper study presents the results for analysis of the tunnel cross-section corresponding to maximum overburden (1520 m). An analysis was performed for each section with an unsupported excavation to determine the convergence behavior of the rock mass. The tunnel excavation was analyzed accordingly with inclusion of support systems. The maximum allowable displacement corresponding to the excavation of fair to good rockmass was considered to be 1% of B, where B is the excavated width of the tunnel. Thus, for this tunnel, with the excavated height/span being 14 m, the allowable displacement would be around 140 mm. Also, for fair rockmass conditions, a displacement of 1% was acceptable. Various rock properties such as deformation modulus, unconfined compressive strength, density, and Poisson's ratio for different classes were deduced from laboratory tests results, further correlated with Q values to derive the rock mass parameters. All other parameters were calculated based on Hoek-Brown criteria (Hoek et al. 2018) using Rocscience Software, RocLab. The rock mass parameters considered in the FEM analysis are given in Table 2. Further overburden pressure at the tunnel location was used to calculate resultant vertical stresses and different in situ stress ratios (K) were used to calculate variation of major and minor in situ stresses.

S. No.	***Feature***	***Dip Direction***	***Dip Angle (^{0})***
1	Joint 1(bedding plane)	N 220°	20
2	Joint 2	N 80°	80-90
3	Joint 3	N 350°	80
4	Joint 4	N 60°	50

Table 1: Joint Sets for UNWEDGE Analysis.

Parameter		***Q Value***	
Barton's Q value		4 ≤ Q ≤ 10	Q > 10
Uniaxial compressive strength (MPa)		50	70
Deformationmodulus (MPa)		7100	16700
Hoek-Brown Criteria	mb	1.403	5.138
	s	0.007	0.036
	a	0.504	0.501
Poisson's ratio		0.3	0.25
Bulk unit weight (kN/m$^{3)}$		27	27
In situ stress ratio		1.5	1.5

Table 2: Rock Mass Parameters.

Corresponding to tunnel crown level at the chosen location, high major (SH) in the range of 61 MPa and the minor principal stresses (Sh) in the range of 41 MPa were calculated. The overburden stress at the site of about 41 MPa corresponding to around 1520 m deep tunnel near chainage 5500. The major and minor in-situ stress ratios at site are about 1.5 to 1.0 from which, a stress ratio of 1.5 was finalized for design. The in-situ stress ratios largely depend on the orientation of the tunnel and hence optimized as per the depth and the direction of the section modelled. Therefore, in this analysis, the stress ratio 1.5 and 1 was used for both rock classes as tabulated in Table 2. During numerical modelling, initially rock support system as per Q-system support chart were modelled which were further modified based on stress analysis results in order to stabilize the tunnel. The various parameters governing the Q value of rockmass are:

$$Q = [RQD/J_n] \times [J_r/J_a] \times [J_w/SRF]$$

Where, the six parameters are:

RQD = Degree of jointing (Rock Quality Designation).
Jn = Joint set number.
Jr = Joint roughness number.
Ja = Joint alteration number.
Jw = Joint water reduction factor.
SRF = Stress Reduction Factor.

Tunnel		
Good (Q >10) and Fair (4<Q<10)	Bolt: 8 m long; spacing 1.20m C/C; shotcrete:250 mm thick (wire mesh reinforced)	Bolt: 10 m long; spacing 1.20m C/C; shotcrete:250 mm thick (wire mesh reinforced)

Note: C/C- center to center spacing.

Table 3: Rock Support Type.

Parameters	***Rock bolt***	***Shotcrete***
Diameter (mm)	25	-
Young’s modulus (MPa)	200,000	31,600
Tensile capacity (MN)	0.19	-
Poisson’s ratio	NA	0.24

Note: NA: Not Applicable.

Table 4: Support Elements.

Excavation Sequence

Tunnel design started with simulation of sequential excavation steps: (1) Defining initial in-situ stress condition before excavation (2) ground softening of the material within the tunnel by 30%, to simulate excavation of initial rounds of top heading (3) further softening by 20% with spraying of sealing layer of shotcrete 50mm (4) further softening by 20% installation of support systems like rock bolts and shotcrete 1st layer 100mm (with reduced/young properties) (5) simulate completion of top heading by removal of material with final layer of shotcrete 100mm (with hardened properties) (6) tunnel bench excavation and support (full bench), and (7) excavation and support of invert of the tunnel. In order to simulate 3D excavation effects in two-dimensional analysis, concept of core replacement (step wise material softening) is used. The details of support system used in model and their properties are presented in Table 3 and Table 4.

Monitoring Plan

The philosophy of the tunnel design here is based on sequential excavation, followed by installation of support systems in stages and finally validation of design by geotechnical monitoring. Displacements of the tunnel periphery were recorded for all considered sections using 3D optical targets installed at the reference points on the shotcrete faces. For each target, the three-dimensional measurements (X, Y, Z) of the displacement vector can be recorded, and relative movement are measured. All the instrumentation and monitoring data were recorded real time and stored in excel worksheet format so that it can be processed and plotted for review and verification. Figure 3 demonstrates the position of 3D optical targets (T1- T5) in the tunnel sections, where T1, T2, T3 are part of tunnel crown and T4, T5 are on tunnel side walls. These instruments were installed at 50-m intervals along the tunnel length, and each section was monitored on a regular basis. Later, the monitoring frequency was modified as per displacement variation.

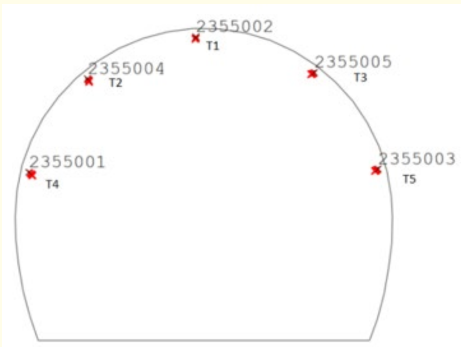

Figure 3: Monitoring arrangement inside tunnel.

Result and Discussion

Figure 4 and Figure 5 show the stage-wise vertical and horizontal displacements at tunnel crown and side walls obtained from numerical analysis for mixed ground conditions with an overburden of 1520 m, respectively. It was observed that majority of the vertical displacement at crown (T1) happened during heading excavation. Also, there is incremental trend in vertical displacements at the crown during bench excavation. Similarly, the horizontal displacement at the wall (T4 and T5) increases during benching and invert excavations. A comparison of Figure 4 (a and b) and Figure 5 (a and b) concludes that vertical and horizontal displacement values at tunnel crown and side walls are significantly high in poor zone of mixed rock conditions than the good zone.

Figure 6 (a) demonstrates the correlation of predicted vertical displacement at tunnel crown with observed field monitoring values from monitoring instruments. It clearly reflects that the vertical displacement increases with declining rock quality i.e. Q values, however relative difference is less for observed values (as some of the displacement will be lost initially till installation of instruments). Figure 6 (b) presents the correlation of horizontal displacement of tunnel wall caused by excavation with field monitoring values for the given overburden and ground conditions. It clearly reflects that in the side walls, the horizontal displacement also increases with declining rock quality i.e., Q values. However, it is important to note that observed field values are not following any explicit increasing or decreasing trend and generally fall around a nominal range, which support the notion that rockmass at site is not as varying as much as it was anticipated in design, although mixed ground conditions are prominently occurring which results in that variation. It is further observed that predicted values of vertical displacements are of the order of 100 mm while the observed field values are around 70-75mm which further supports that considering the mixed face conditions in numerical analysis is a conservative yet realistic approach which shall be adopted in such projects.

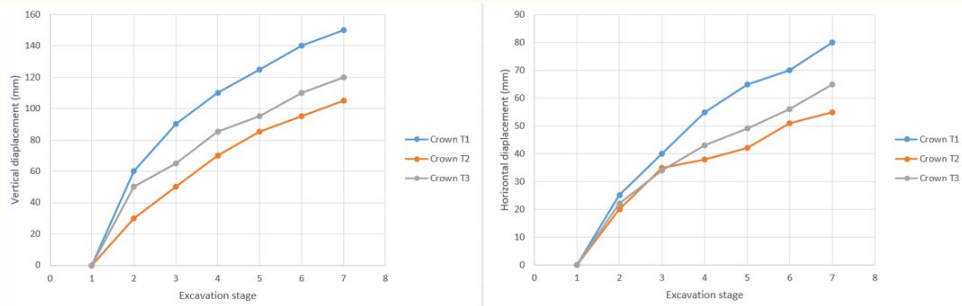

Figure 4: (a) Vertical Displacement trend at tunnel crown with 1520-m overburden (b) Horizontal displacement trend at tunnel crown with 1520-m overburden.

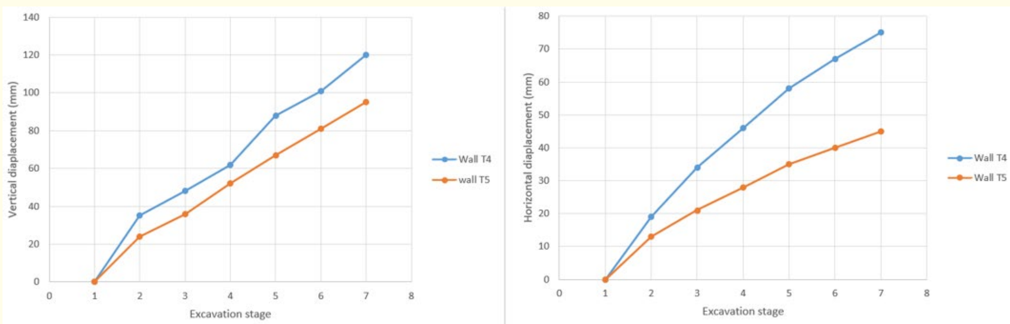

Figure 5: (a) Vertical Displacement plots at tunnel side wall with 1520-m overburden (b) Horizontal displacement plot tunnel side wall with 1520-m overburden.

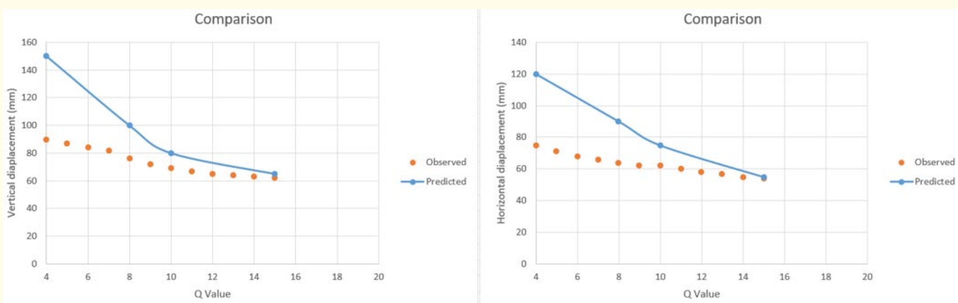

Figure 6: (a) Predicted versus observed maximum vertical displacement in crown; (b) predicted versus observed maximum horizontal displacement in wall.

	Predicted		***Observed***	
	Max. total displacement	position	Max. total displacement	Position
Mixed Rockmass	165	crown	94	crown

Table 5: Average Displacement Value for Different Rock Classes.

Table 5 demonstrates the correlation between the observed and predicted displacement results during sequential tunnel excavation at maximum overburden around chainage 5500m and same is being compared with nearest monitoring results. The results of the predicted displacements and plastic zone can be seen in Figure 7.

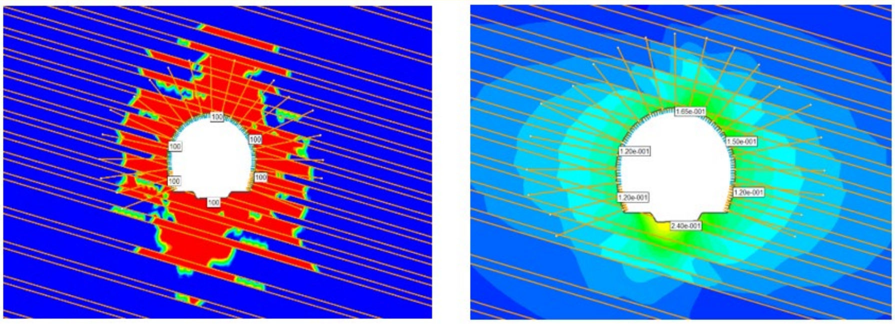

Figure 7: Demonstration of yield zone and total displacements post excavation for the mixed rock conditions.

It is to be noted that observed displacements shall be dealt cautiously, as they highly depend on parameters like installation time of the instrument after excavation, if any damage during nearby excavation, and precision of the readings. Moreover, the predicted values are derived from a simplified plasticity-based numerical modelling. Including these limitations, such comparisons can produce quite crucial conclusions. It can be deduced from Table 5 results that the observed readings are almost 40% less than the predicted readings for a mix of fair and poor rockmass, reflecting the need to review the input parameters adopted for analyses.

In order to incorporate the discontinuity-controlled failures in the tunnel due to existing joint sets and tunnel orientation, a discontinuity analysis was performed with the shear strength parameters of joints as stated before. Unwedge is a numerical tool to carry out this analysis using joint sets and tunnel axis orientation as inputs and identifying the possible sliding wedges into the tunnel, which can be further supported to achieve the desired factor of safety. Rock support similar to Table-3 was modelled in supported case of Unwedge analysis. Table 6 shows the critical results of the unwedge analyses. These results clearly shows that appropriate support system is necessary to ensure long term stability against such failures.

Wedge type	***Factor of safety (Without support)***	***Factor of safety (With support)***	***weight (ton)***	***height (m)***
Crown	0.1	3.41	19.18	3.8
Walls	0.00	1.68	23.9	4.6

Table 6: Summary of UNWEDGE analysis.

Sensitivity Analyses

This project essentially involved significant variations in ground conditions, which includes the mixed face (varying rock) conditions all along the alignment. This becomes extremely critical at high overburden zone due to high magnitude of in-situ stresses along with need to identify critical parameters of rockmass which play a significant role in deformations and forces in support elements. This sensitivity analysis also assists in establishing effectiveness of site practices like support optimization in order to contain the deformations under fair/poor rockmass conditions. The numerical model is simulated using FEM software RS2 considering the rockmass with joints to simulate the actual realistic ground profile. An effort is being made to see the sensitivity of output results (deformations and plastic zone thickness) with respect to input parameters (cohesion, friction and deformation modulus). It is based on the studies performed by Barton (Barton et al. 2002) to correlate the effect of pre-injection/grouting on the material properties of rockmass. It was concluded that Q-parameters, Q-values, moduli, velocities, permeabilities and all other ground parameters improved significantly due to pre-injection/grouting performed at pressures ranging from 5-10 MPa. Typically, illustration shows that the Q value increased from 0.5 (before grouting) to around 17 post injection which is also improving other Q parameters like joint roughness thereby enhancing the shear strength of rockmass. Typical illustration of pre-injection is shown in Figure 8.

To simulate the ground improvement post grouting, the FEM model was checked by improving shear parameters and deformation modulus in stages; from 10% to 30% to see the impact on the displacements and plastic zone. As expected from the findings of Barton, It was interesting to see the impact of increasing shear strength parameters and deformation modulus on the displacements and plastic zone thickness, which clearly shows the reduction in displacements (more sensitive to increase in deformation modulus post grouting) and reduction in plastic zone (more sensitive to shear parameters of rockmass), results are summarized in Table 7 and pictorial form is depicted in Figure 9. It can be clearly visible in Figure 9 that as compared to initial state shown in Figure 7, the deformations are significantly lowered, and plastic zone is also reduced due to improvement in rockmass and joint parameters thereby improving the overall Q value of rockmass. These results support the argument that ground improvement techniques strengthen the shear and strength parameters of rockmass which in-turn results in significant improvement in deformations and yielding zones around the excavation boundaries. Hence, ground improvement techniques like consolidation and compaction grouting can give much better results under poor/fair/ ground conditions where there is high probability of grout intake due to high permeability. Therefore, what was concluded experimentally is also verified by numerical analysis and results are showing the same trend, also the lower values observed from field monitoring instruments also encourages the fact that pre-injection/grouting has worked out positively at site and hence less displacements were recorded.

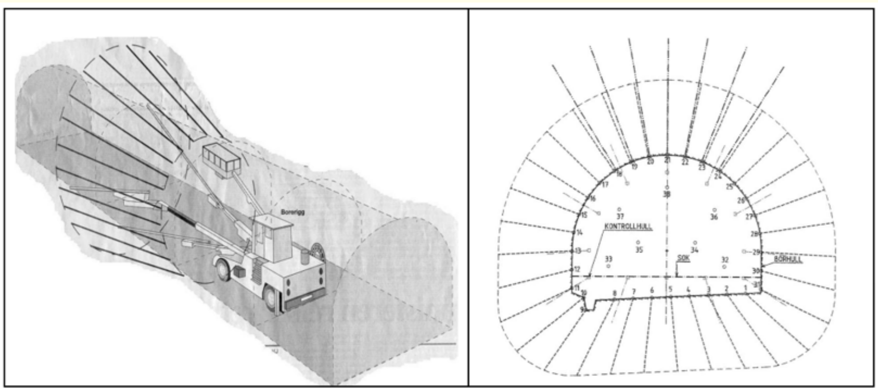

Figure 8: Conceptual pre-injection screens, which may vary in length from 20 to 30 m, (Barton 2011).

Output Parameter	***Cohesion (% increase)***			***Friction Angle (% increase)***			***Deformation modulus (% increase)***		
	10%	20%	30%	10%	20%	30%	10%	20%	30%
Total displacement (% Reduction)	4%	7%	12%	3%	8%	11%	8%	15%	22%
Yield zone thickness (% Reduction)	10%	18%	25%	5%	8%	12%	4%	7%	11%

Table 7: Summary of sensitivity analyses.

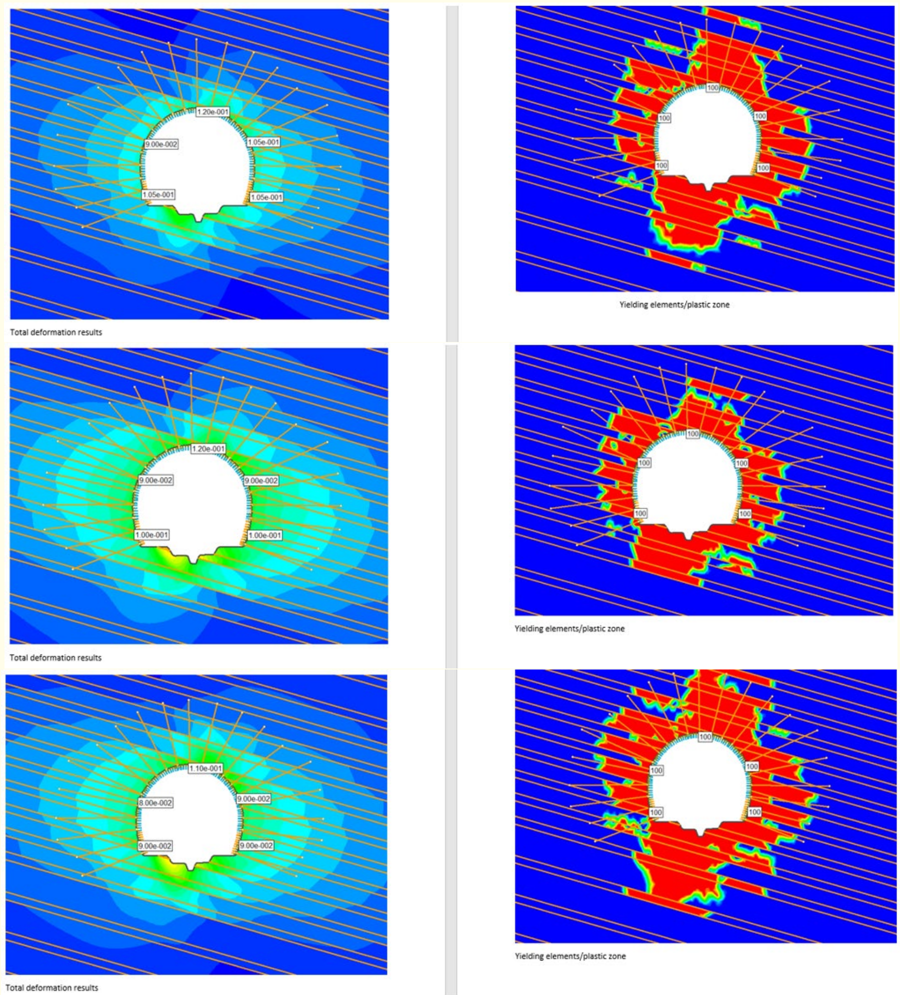

Figure 9: Sensitivity analyses showing variation in displacements and yield zone (a) Cohesion increased by 30% (b) friction angle increased by 30% (c) Deformation modulus increased by 30%.

Summary & Conclusions

Based on the results of the study, it was observed that more than 50% vertical displacement at the tunnel crown happened during top heading and further increased continuously during benching. Similarly considerable increase in horizontal displacement were also observed during benching. It was noted that predicted displacements based on the adopted design parameters values were generally higher than observed displacement values for fair and poor rock mass conditions. One of the reasons for this difference could be the delay in monitoring instrument installation which sometimes give rise to this variance which calls for due consideration of it during interpretating the monitoring results from field observations. Another reason for this variation can be the pre-injection/grouting performed at site which improved the shear and strength parameters of rockmass. The sensitivity analyses showed that critical ground parameters can impact the stability of the tunnel cross-section which was derived experimentally by Barton in 2002 and same is being numerically verified in this paper. Cohesion is an important shear parameter which is sensitive to the yielding zone thickness and can help in reduction of plastic zone thickness and reduction in rock bolt lengths. Deformation modulus was also observed to be sensitive to deformations/displacements occurring around the tunnel and forces/stresses on the support elements. Thus, improving rock mass conditions using grouting or other methods can help in minimizing displacements and plastic zone around and along the tunnel profile.

References

1. Franzius JN and Potts DM. “Influence of mesh geometry on three-dimensional finite-element analysis of tunnel excavation”. Int. J. Geomech 5.3 (2005): 256-266.
2. Goodman RE and Shi G. “Block theory and its application to rock engineering”. Prentice Hall, Englewood Cliffs, NJ (1985).
3. Grimstad E and Barton N. “Updating the Q-System for NMT”. Proc., Int. Symposium on Sprayed Concrete—Modern Use of Wet Mix Sprayed Concrete for Underground Support, Norwegian Concrete Association, Oslo, Norway (1993): 46-66.
4. Hoek E and Brown ET. “Hoek brown failure criterion 2018 edition”. J. rock mechanics and geotech (2018): 445-463.
5. Barton N. “An Engineering Assessment of Pre-Injection in Tunnelling”. Indian Rock conference by ISRMTT, Roorkee (2011).
6. Barton N, Buen B and Roald S. “Strengthening the case for grouting -Pre-Grouting for Tunnels in Jointed Rock”. Tunnels and Tunnelling International (2002): 34-39.
7. RS2 [Computer software]. Toronto, Rocscience.
8. RocLab [Computer software]. Toronto, Rocscience.
9. UNWEDGE [Computer software]. Toronto, Rocscience
10. Usmani A., et al. “Interaction mechanism between two large rock caverns”. Int J Geomech ASCE 15.1 (2013): 06014014.