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Static and Dynamic Assessment of Tunnel Rock Supports in Weak Rock

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Abstract. Tunnel rock support design can be very intriguing as it involves a high-risk factor and any improper analysis may lead to severe financial loss. Conventional empirical methods along with the numerical analysis provide a better interpretation of the expected problems during the construction. In this study, an approach has been made to understand the effectiveness of various supports in weak rock conditions. The data from the literature is taken as input for modeling using the finite element software PHASE^2 of RocScience for both static and dynamic loading. The performance of the supports has been evaluated in terms of stress distribution, plastic deformation to suggest an optimum support system for the available data. For the continuum model, a rock mass devoid of joints was considered and an equivalent Mohr-Coulomb failure criterion was assumed for it. Joints were introduced in the dis-continuum model by providing interface elements. Dynamic analysis has been performed with the pseudo-static and time response method. The results indicate that the combined use of rock bolts and shotcrete as support provides a reliable support system under various static and dynamic forces in tunnel supports.

Keywords: Static and dynamic analysis; Continuum and Dis continuum modeling; Tunnel rock support; Hoek-Brown failure criteria; Mohr-Coulomb failure criteria.

1 Introduction

Tunnel construction in rock is challenging as several factors, which are not readily visible comes into consideration for the design. Technological advancements along with the empirical relations can help design safe and economic tunnel support. The unpredictable behavior in rocks due to the anisotropy and discontinuities requires an effective support system with reinforcements, lining, etc. to avoid huge losses and complexities [1]. The degree of anisotropy such as joints, faults, fissures, etc., strongly affects the mechanical performance of the rock mass [2]. The detection of rock mass disturbances near the tunnel face and the surrounding soil is compulsory to prepare a safe and reliable tunnel support design. The empirical relations obtained from the experience of several field experiments, establish the quantitative categories to classify these rock masses according to their properties such as RQD, RMR, GSI, RMi [3, 4, 5]. For an appropriate calculation with a more reliable result, numerical models can be utilized with various inputs from empirical relations to produce a meaningful knowledge about the ground conditions [6]. Both the methods are very much sensitive to the input parameters; hence they should be accurately and carefully measured [7].

Weak Rocks such as claystone, siltstone, coal, limestone, soft slate, shale have higher possibilities of squeezing and weathering as they are generally a transition between hard rocks and non-cohesive soils. A rock mass whose uniaxial compressive strength (generally less than 50MPa) is less than one-third of the in-situ stress, is considered a weak rock [1].

Performance analysis of the support system in dynamic (seismic) loading in addition to static loading is necessary for a seismically vulnerable region. This paper aims to compare the performance tunnel for various support systems in weak rock concerning both static and dynamic loadings.

2 Empirical Approach

2.1 Rock Mass Classification

The purpose of rock mass classification is to develop a systematic way of describing and grouping rocks of similar behavior without any ambiguity and determine the support requirements for tunnels. The rock masses can be classified based on 3 major factors: i) intact rock properties ii) discontinuity characteristics iii) boundary conditions. Several classification systems have been developed based on field investigations and experience. Some of the common classification systems are the Rock Mass Rating (RMR), Q-system, Geological Strength Index (GSI).

Barton et al. (1974) [8] proposed the Q-system of rock mass classification which can be calculated as follows:

$$Q = \frac{RQD}{Jn} * \frac{Jr}{Ja} * \frac{Jw}{SRF}$$

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, and SRF = stress reduction factor.

The data reported in [9] is used for the analysis. As the detailed geotechnical study was not considered along tunnel alignment, the average Q value was obtained considering the minimum and maximum value of rock mass.

Table 1. Estimation of Average Q-value

Poor rock mass			Fair rock mass			Average Q
Parameter of Q	Rating	Q_{min}	Parameter of Q	Rating	Q_{max}	*$(Q_{min} * Q_{max})^{1/2}$*
RQD	35		RQD	65		
J_n	91		J_n	4		
J_r	1	0.38	J_r	1	1.625	0.785
J_a	2		J_a	2		
J_w	1		J_w	1		
SRF	5		SRF	5		

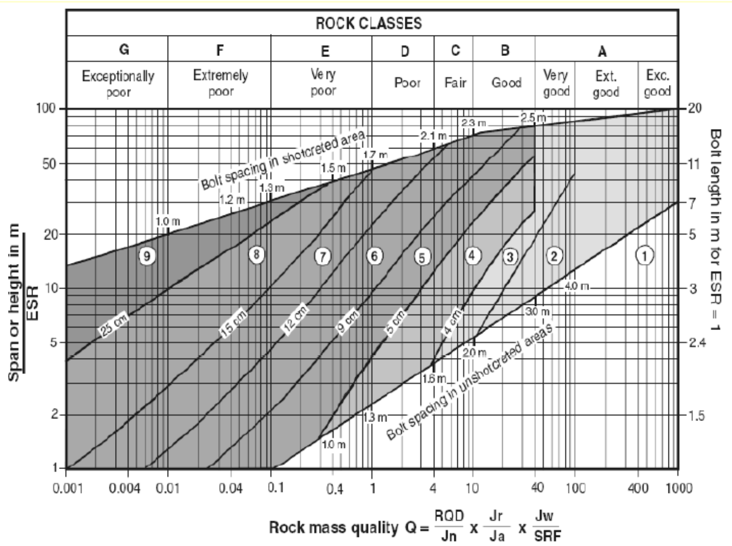

Fig. 1. Q chart for support requirement

From Fig. 1., support categories lie in the fourth region. Spacing of bolts = 1.5 m, Length of bolts = 3 m, Diameter of bolts = 1.9 cm, Thickness of shotcrete = 10 cm for Q = 0.785. For the given value of Q, rock mass lies in a very poor class [10].

3 Numerical Approach

The tunnel supports have been analyzed using Phase^2 9.0 [11], a 2D elastoplastic finite element program capable of calculating stresses, displacements, yielded elements of the plastic zone in underground structures. Generally, the continuum model is utilized with a rock mass devoid of joints and equivalent Mohr-Coulomb failure criterion for such studies. But for this paper, analysis is performed on a dis-continuum model with joints introduced by interface elements. Modeling of rock mass behavior is carried out considering it to be of a strain-softening case with the Generalized Hoek-Brown failure criteria. Half of the peak values of the strain-softening case are taken as residual parameters. Parameters used in the model are given in Table 2 and Table 3 [9]. The mesh is discretized as 6-nodded triangular elements of 1413 number and it consists of 2991 nodes. Based on properties defined at each node, Phase^2 solves the problem using the Gaussian elimination technique. Sigma 1 is the horizontal stress and Sigma 3 is the vertical stress for in-situ stress conditions. Stress field due to gravity exists in the in-situ stress condition. The proposed tunnel is inverted-D shaped of 3.5*3.5 m and located at 35 m depth. The set of joints at 60° inclination at 1 m spacing has been assumed. The geometry of the model is taken as 24.5*24.5 m. A comparative study among tunnel supports viz. unsupported, rock bolt, shotcrete, and combination of rockbolt and shotcrete are carried out for both static and dynamic loading. The stresses, total displacements, and yielded elements are obtained for the crown (A), walls (B & C), and invert (D) of the tunnel (Fig. 2).

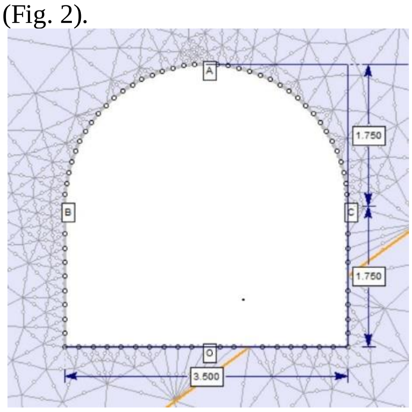

Fig. 2. Inverted-D Shaped tunnel (All dimensions are in meters)

Table 2. Details of rockbolt and shotcrete

Support system	Young's modulus (MPa)	Poisson's ratio	Tensile capacity residual (MN)
Shotcrete	30000	0.2	-
Rockbolt	200000	-	0.1

Table 3. Details of the rock mass properties

Type of rock	Sandstone
Unit Weight (kN/m^3)	26.5
Young's Modulus (MPa)	18500
Poisson's Ratio	0.14
Uniaxial compressive strength (MPa)	3.124
m_b	2.301
s	0.002
a	0.509
Residual m_b	1.1505
Residual s	0.001
Residual a	0.2545

4 Static Analysis

For the static analysis using Phase2, the stresses Sigma1, Sigma3, yielded elements, and the total displacements are mentioned in Table 4. Additionally, the results of the analysis are depicted in Figure 3 to Figure 6.

Induced stresses at invert are less than that of sidewalls and crown before and after support installation; however, total displacements at invert is higher. The negative minor principal stresses (tensile stresses) in unsupported condition has been corrected by the installation of supports. There has been a decrease in total displacement at all the concerned locations with the installation of supports, but no significant change in displacement could be observed for support systems with shotcrete alone or combined shotcrete and rockbolt. The yielded elements have reduced from 508 to 443 on shotcrete application. The overall trend for the given gravity-induced stress conveys that the support from shotcrete alone can suffice for a weak rock for the mentioned properties.

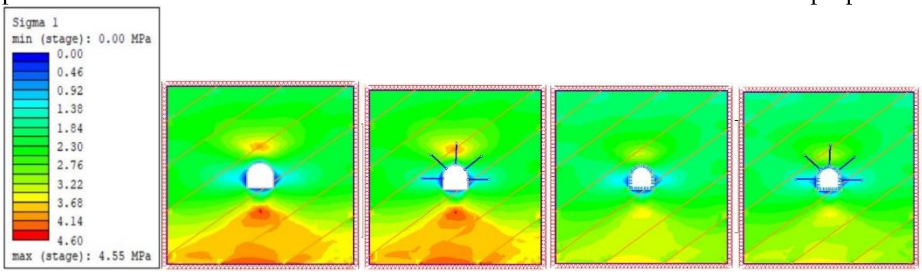

Fig. 3. Stress (σ_1) distribution before and after support installation

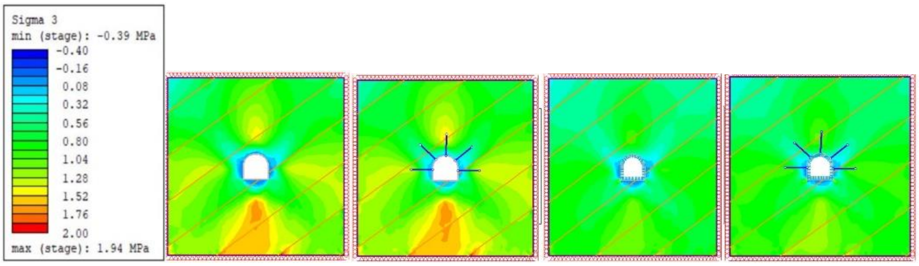

Fig. 4. Stress (σ_3) distribution before and after support installation

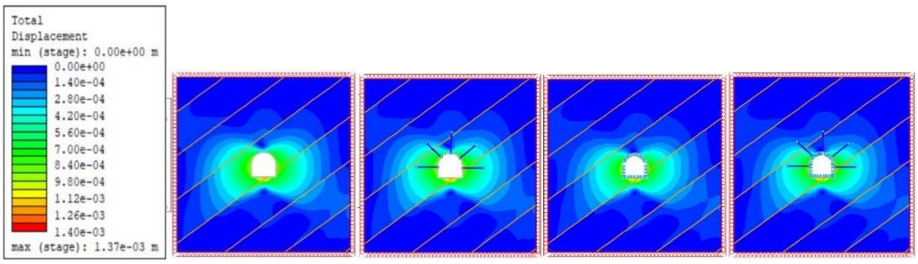

Fig. 5. Total displacement profile before and after support installation

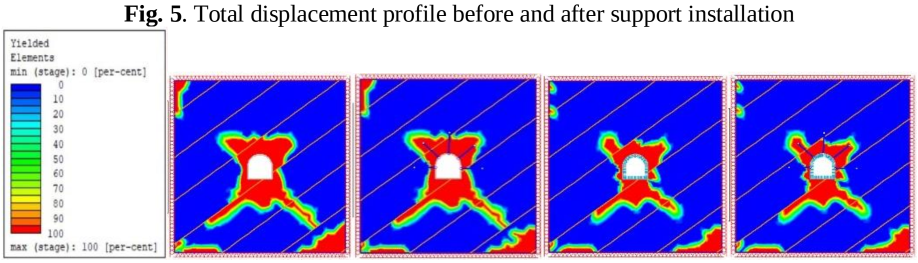

Fig. 6. Percentage of yielded mesh elements before and after support installation

Table 4. Summary of static analysis

Support System	σ_1 (MPa)		σ_3 (MPa)		Total Displacement (mm)		Yielded elements
Unsupported	A	0.92	A	-0.04	A	0.7	508
	B	0.46	B	-0.04	B	0.98	
	C	0.23	C	-0.04	C	0.91	
	O	0	O	-0.11	O	1.33	
Rockbolt	A	0.61	A	0	A	0.63	502
	B	0.61	B	0	B	0.91	
	C	0.37	C	0	C	0.91	
	O	0.14	O	0	O	1.33	
Shotcrete	A	2.7	A	0.63	A	0.06	443
	B	0.74	B	0.1	B	0.72	
	C	0.46	C	-0.08	C	0.72	
	O	-0.1	O	-0.08	O	1.14	
Shotcrete + Rock-bolt	A	2.75	A	0.55	A	0.06	443
	B	0.76	B	0.06	B	0.72	
	C	0.76	C	0.06	C	0.72	
	O	-0.1	O	-0.11	O	1.2	

5 Dynamic Analysis

5.1 Pseudo-static analysis

Pseudo static analysis has been performed considering horizontal seismic coefficient ($k_H = 0.3$) and downward vertical seismic coefficient ($k_V = 0.2$), which are within the typically exploited values. The vertical seismic coefficient is considered in the downward direction as it produces maximum stresses along the tunnel periphery when acting in the direction of gravity [12]. Seismic coefficients apply additional body force on each mesh. The stresses Sigma1, Sigma3, elements yielded from the plastic zone and the total displacements for the considered combination of supports and loading are extracted in Table 5 and are represented in Figure 7 to Figure 10.

Induced stresses at the crown are maximum before and after support installation. The total displacements on right side walls (C) are more as the direction of k_H is taken towards the right. There has been a decrease in total displacement at all the concerned locations with the installation of shotcrete or combined shotcrete and rockbolt. The yielded elements have reduced from 477 to 421 on shotcrete application. It has to be highlighted here similar to the previous case, shotcrete with given properties is sufficient to control displacements and yielded elements in the given weak rock exposed to pseudo-static loading.

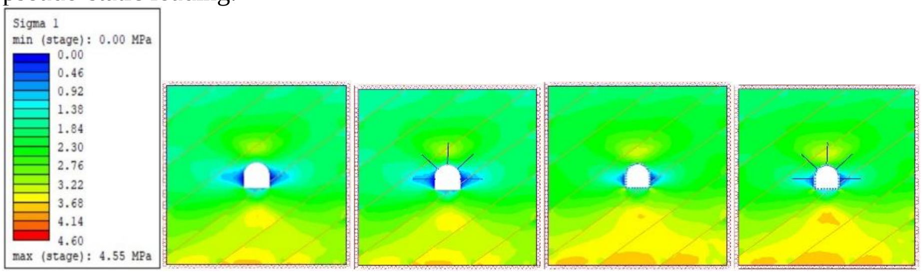

Fig. 7. Stress (σ_1) distribution before and after support installation

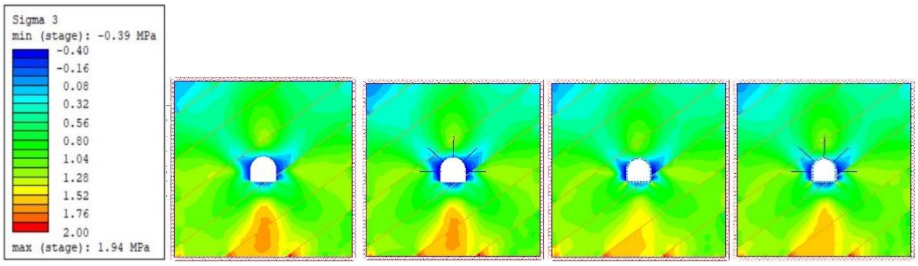

Fig. 8. Stress (σ_3) distribution before and after support installation

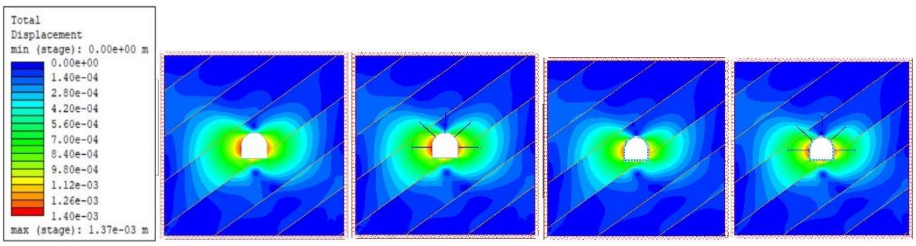

Fig. 9. Total displacement profile before and after support installation

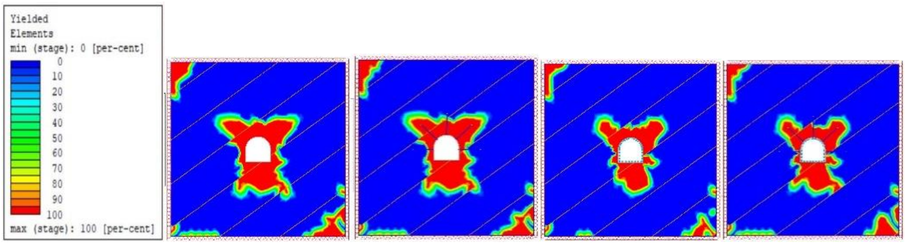

Fig. 10. Percentage of yielded mesh elements before and after support installation

Table 5. Summary of pseudo-static analysis

Support System	σ_1 (MPa)		σ_3 (MPa)		Total Displacement (mm)		Yielded elements
Unsupported	A	1.7	A	0.38	A	0.315	477
	B	0.45	B	0	B	0.9	
	C	0.45	C	0	C	0.81	
	O	1.2	O	0.19	O	0.765	
Rockbolt	A	1.7	A	0.38	A	0.315	475
	B	0.45	B	0	B	0.855	
	C	0.45	C	0	C	0.81	
	O	1.2	O	0.19	O	0.765	
Shotcrete	A	3	A	0.76	A	0.08	421
	B	0.5	B	0	B	0.72	
	C	0.5	C	0	C	0.68	
	O	1.75	O	0.29	O	0.52	
Shotcrete + Rockbolt	A	2.75	A	0.67	A	0.08	440
	B	0.75	B	0	B	0.72	
	C	0.75	C	0	C	0.68	
	O	1.25	O	0.19	O	0.68	

5.2 Time response analysis

For the time response analysis, the data of the Nepal earthquake of the magnitude of Mw = 7.8 that occurred in the year 2015 is utilized. The acceleration time history has been shown in Figure 11 and Figure 12.

Dynamic analysis of the four different support systems has been done in two steps: undamped and damped. Earthquake loading is applied at the top of the model as time acceleration history. The viscous boundary condition at the base of the model is provided with an absorbent boundary such that the incoming pressure and shear waves effect is consumed. The vertical boundaries are set to transmit boundary conditions, ensuring no reflection of outgoing waves at these boundaries. After the completion of undamped dynamic analysis, the values of Rayleigh damping constants are adjusted to $\alpha_m = 0.0043$ and $\beta_k = 0.45$ to achieve a Rayleigh damping of 5%. Using the obtained values of the Rayleigh damping coefficient, damped dynamic analysis is performed again, the results of which are depicted in Figure 13 to Figure 16. The stresses Sigma1, Sigma3, yielded elements of the plastic zone, and the total displacements for the considered combination of supports and loading conditions are mentioned in Table 6.

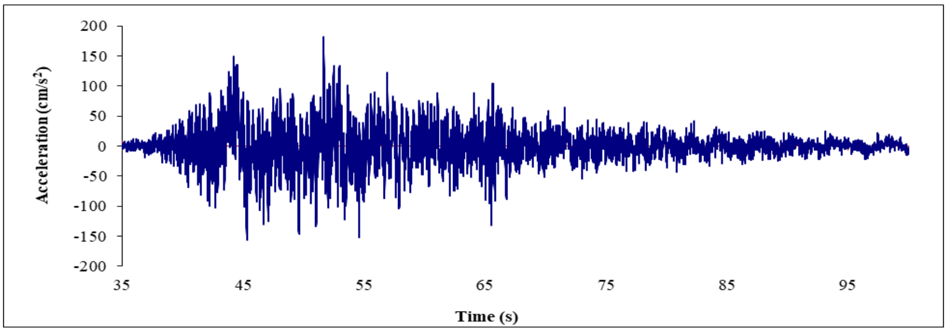

Fig. 11. The horizontal component of the acceleration time history of the Nepal earthquake, 2015 (Source: USGS).

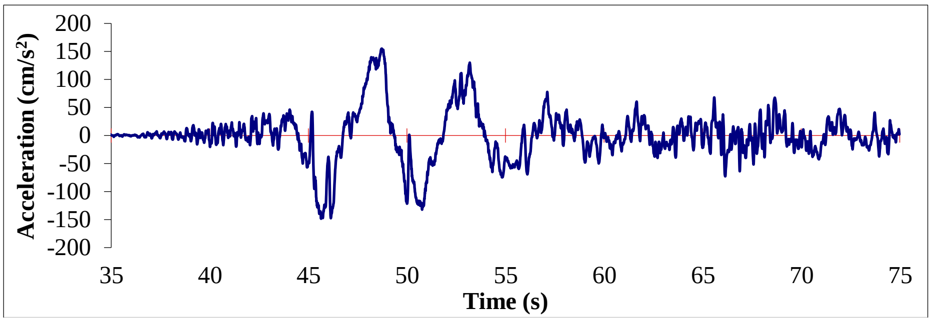

Fig. 12. The vertical component of the acceleration time history of the Nepal earthquake, 2015 (Source: USGS).

Stresses at the crown have been maximum before and after support installation; however, displacements on the right-side walls (C) were more due to the nature of load application. The development of negative minor principal stresses is corrected on the installation of shotcrete alone, at the concerned locations. Displacements reported are considerably high in all the cases as all the elements of the model have yielded because of the strain accumulation due to dynamic loading. No specific trend in displacement could be observed before and after the installation of support. There has been a 100% yielding of plastic elements in all the cases. From the analysis below, it can be inferred that shotcrete can alone suffice as the optimum support condition for the given weak rock properties in time response analysis.

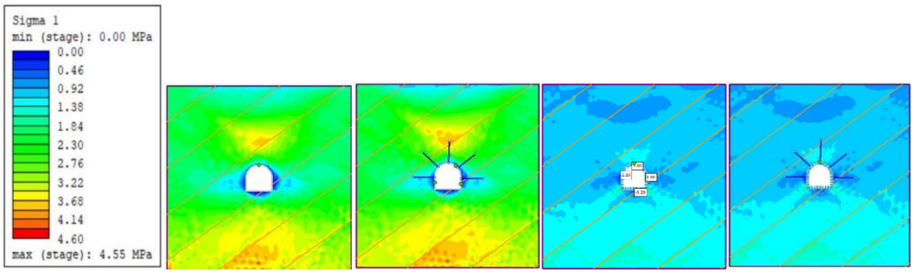

Fig. 13. Stress (σ_1) distribution before and after support installation

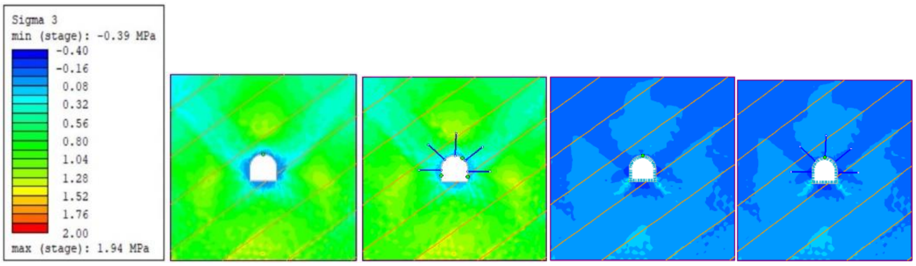

Fig. 14. Stress (σ_3) distribution before and after support installation

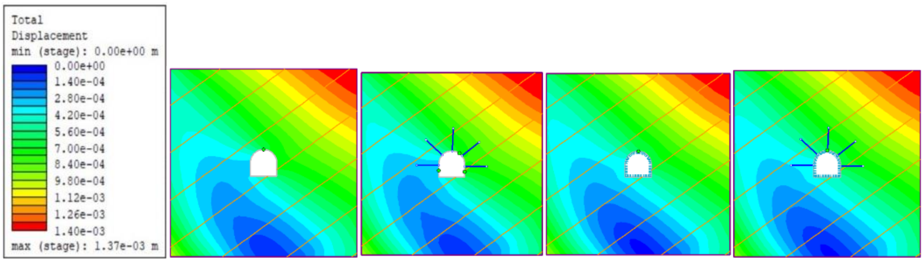

Fig. 15. Total displacement profile before and after support installation

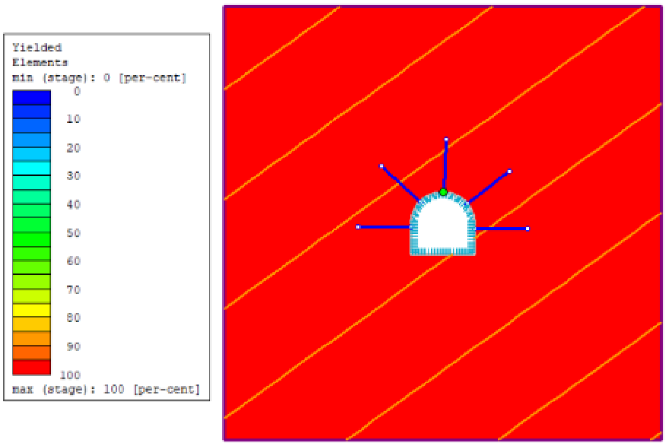

Fig. 16. Percentage of yielded mesh elements on rockbolt and shotcrete installation

Table 6. Summary of time response analysis

Support System	σ_1 (MPa)		σ_3 (MPa)		Total Displacement (mm)		Yielded elements
Unsupported	A	0.77	A	-0.04	A	297	1305
	B	0.51	B	-0.04	B	165	
	C	0.51	C	-0.04	C	363	
	O	0.26	O	-0.04	O	231	
Rockbolt	A	0.68	A	-0.04	A	297	1305
	B	0.42	B	-0.04	B	165	
	C	0.42	C	-0.04	C	363	
	O	0.16	O	-0.04	O	231	
Shotcrete	A	3.8	A	1.6	A	306	1305
	B	2.2	B	0.95	B	204	
	C	0.6	C	0.3	C	306	
	O	-0.2	O	-0.35	O	204	
Shotcrete + Rockbolt	A	3.8	A	2.25	A	306	1305
	B	0.6	B	-0.35	B	204	
	C	0.6	C	-0.35	C	306	
	O	-0.2	O	-0.35	O	238	

6 Conclusions

The weak rock mass was characterized by the geotechnical investigations reported in [9]. Based on values obtained from the empirical method (Q-system) of rock mass classification, numerical analysis using Phase^2 was performed for different loading conditions. The dis-continuum analysis provides a better insight into the rock mass behavior for any condition. The highly conservative results in the time response analysis is due to the high ground displacement amplitude considered for this model. Generally, the displacement variation is quite low in actual field conditions. Among the four support systems proposed in the paper for the loading conditions, shotcrete outperformed other support conditions in terms of stresses, displacements, and yielded elements. The analysis represents that, the safety of the structure is not directly related to the amount of support provided. The behavior of the rock mass, to any kind of support system, is entirely dependent on its properties. Therefore, a proper combination of the empirical and numerical methods can provide us with an estimation of the response of the rocks to the tunnel construction. Nevertheless, the validation of the observed results requires observational data from the field.

References

1. Hoek, E.: Tunnel support in weak rock. In Proc. Reg. Keynote address, Symposium of Sedimentary Rock Engineering, pp. 20-22. Taipei, Taiwan (1998).
2. Cai, M., Horri, H.: A constitutive model of highly jointed rock mass. Mechanics of Materials 13(3), 217-246 (1992).
3. Koçkar, M.K., Akgün, H.: Methodology for tunnel and portal support design in mixed limestone, schist and phyllite conditions: a case study in Turkey. International Journal of Rock Mechanics and Mining Sciences 40(2), 173-196 (2003).
4. Özsan, A., Başarır, H.: Support capacity estimation of a diversion tunnel in weak rock. Engineering Geology 68(3-4), 319-331 (2003).
5. Kaya, A., Bulut, F., Sayin, A.: Analysis of support requirements for a tunnel portal in weak rock: A case study from Turkey. Scientific Research and Essays 6(31), 6566-6583 (2011).
6. Basarir, H., Ozsan, A., Karakus, M.: Analysis of support requirements for a shallow diversion tunnel at Guledar dam site, Turkey. Engineering Geology 81(2), 131-145 (2005).
7. Basarir, H.: Engineering geological studies and tunnel support design at Sulakyurt dam site, Turkey. Engineering Geology, 86(4): 225-237 (2006).
8. Barton, N., Lien, R., Lunde, J.: Engineering classification of rock masses for the design of tunnel support. Rock Mechanics 6(4), 189-236 (1974).
9. Khadka, S.S., Maskey, R.K.: Stability analysis and design of rock support for tunnel and cavern of Kathmandu University Geo-Lab. Journal of Science, Engineering and Technology 13(I), 1-19 (2017).
10. Singh, B., Goel, R. K.: Tunneling in weak rocks. 5th edn. Elsevier Geo-Engineering Book Series, (2006).
11. Phase2 (2012), A 2D finite element program for calculating stresses and estimating support around the underground excavations, Geomechanics Software and Research. Toronto, ON: RocScience Inc.
12. Bhasin, R.: Effect of earthquake on rock support in tunnels. Tunneling Asia, New Delhi (2013).