Middle-East Journal of Scientific Research 17 (2): 148-156, 2013
ISSN 1990-9233

DOI: 10.5829/idosi.mejsr.2013.17.02.11901

The Effect of Load Split in Tunnels on the Stability of Slopes

Ashkan Fallah Vazirabad and Vahid Hosseinitoudeshki

Department of Civil Engineering,
Zanjan Branch, Islamic Azad University, Zanjan, Iran

Abstract: The slope stability of rocks is an important problem in geotechnical engineering. This paper shows the results of geological studies and the analysis of stability variation of basalt slopes due to step-by-step releasing stress to excavated tunnels in these slopes. Local rock masses have been classified by Rock Mass Rating (RMR), Tunnelling Quality Index (Q) and Geological Strength Index (GSI). Preliminary data of 2D finite element analysis program are provided by information and classifications mentioned above. First the stability of various slopes are analyzed by 2D finite element program and in case of slopes stability, variation of strength reduction factor (SRF) is studied through modeling tunnels with different radius and releasing stress step by step. The results show that SRF decreases as slope's angle and radius of excavated tunnels increases. Also, in most cases value of SRF is decreased by releasing stress in tunnels. In other words an increase in releasing stress reduces slopes stability.

Key words: Load split · Strength reduction factor · Tunnel · Slope

INTRODUCTION

Stability of slopes and excavation of rocks is one of the important problems in underground structures engineering. This problem is noticed in both the design and construction stages and a number of methods are used in calculation of slope stability and tunnel excavation of rocks at the moment [1]. Kinematical, limit equilibrium and numerical analysis are usually recommended for appraisal of rock. The movement of bodies without illustration of the forces that move them is defined as Kinematical analysis. Shear strength and the failure surface, the effects of pore water pressure and the influence of external forces like seismic accelerations are described by equilibrium analysis. In contrast, we perform numerical analysis such as finite element methods to verify outcomes obtained from kinematical and equilibrium analysis [2].

Empirical and numerical methods are generally preferred methods in designing underground engineering structures. Rock engineers and engineering geologists mostly favor empirical methods due to their practicality. Since RMR (Geomechanics classification system) and Q (Tunnelling Quality Index) have achieved a universal acceptance in designing tunnel supports and rock mass classification system, many engineers prefer to use them [3].

In primary design stage of a project, when detailed information of rock mass is very little, rock mass classification systems are very valuable tools for engineers. In contrast, to provide initial estimates of support requirements using several rock mass classification systems is suggested to visualize the composition and specifications of rock mass [4].

These classification systems were originally acquired from many tunneling case studies and they have been applied to many construction designs. Stress redistributions, support performance and deformations around the tunnel cannot sufficiently be calculated by these empirical methods [3, 5]. So, it should be noted that these methods need more particular attention when they are being used. Especially, determination of their values according to the rock mass is very susceptible to the field observations when they are analyzed. The field studies contained geological mapping, core drilling and discontinuity surveying and geotechnical descriptions [6, 7]. In other words, numerical methods such as finite elements method are very dependent on the strength

Corresponding Author: Ashkan Fallah Vazirabad, Department of Civil Engineering, Zanjan Branch, Islamic Azad University, Zanjan, Iran.

parameters of rock masses, which are used as initial input data in the finite element models. So, it is of great importance to remember that both methods mentioned above should be used carefully and their parameters should be determined as close as possible to field data. In summary, significant results on the ground control can be generated by providing reliable input parameters to finite element method [3, 5].

Geology, Field and Laboratory Studies: Fig. 1 defines the initial modeling of the slopes. First of all, model is determined in such dimensions to avoid stress concentration's effect in excavated zone and the minimum dimension is measured 3-4 times bigger than tunnel's biggest diameter (In this study the largest size for diameter of tunnel is 14 meters) [8]. Therefore, as seen in Fig. 1, BC, DC, FE equals 50 m and rock slope length (ED) equals 150 m constantly in all models. Slopes with the following angles, 30, 35, 40, 45, 50, 55, 60, 65 and 70 have been studied in separate models. For instance Fig. 1 shows a slope with dip of 40 degree.

In every model with changing slope's angle, dimensions of X and Y will be subject to change in order to fix rock slope's length and this can be calculated through geometric equations easily.

In each model tunnels with the following radius, 3, 4, 5, 6 and 7 m have been modeled and separately have been analyzed. The distance between centers of all tunnels (O) and center of rock slope (n), En = nD = 75m, is constantly 25 m (On = 25 m), as shown in Fig. 2.

The rock information studied in this paper's models is the basalt rock examined in Boztepe dam site in Turkey [2]. This initial data contains properties of intact rock which is presented in Table 2.

According to the studies by [2] performed on this rock, RMR [9], Q [10] and GSI [4] values are indicated in Table 3. Other valuable data are also available from this research.

Estimation of Rock Mass Properties and Finite Element 2D Program's Input: One of the significant stages in geotechnical studies is determining rock mass properties for finite elements 2D program's input. In-situ studies are very expensive, time-consuming and demanding. Though in this paper, Roclab software from [11] is applied to estimate rock mass properties. The following statements directly provided from Roclab user guide PDF file, define program's function briefly.

Roclab is a software program for determining rock mass strength parameters, based on the generalized Hoek-Brown failure criterion. One of the major obstacles which are encountered in the field of numerical modeling for rock mechanics is the problem of data input for rock mass properties.

The rock mass properties determined by Roclab can be used as input for numerical analysis programs such as Phase2 (finite element stress analysis and support design for excavations) or Slide (limit equilibrium slope stability analysis) [11].

Comparison of Hoek-Brown and Mohr-coulomb's failure curves of the studied basalt is shown in Fig. 3. Input data and output results obtained from comparing failure criteria mentioned above is seen in Table 4.

It should be noticed that rock mass constants can be calculated by [12] suggested equations (i.e. mb, s and a).

$$m_b = m_i \exp(\frac{GSI-100}{28-100}) \tag{1}$$

$$s = \exp(\frac{GSI-100}{9-3D}) \tag{2}$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - e^{-20/3}\right) \tag{3}$$

The value of disturbance in the rock mass affects equation D factor which is related to the method of excavation (e.g. Smoothness of blasting), the amount of equation D is presented in Table 4 [12].

Strength of Rock Masses: Many researchers suggest several empirical equations to estimate the strength of rock masses (σ_{cmass}, Mpa) on the basis of RMR, Q, GSI values which the most used ones are offered in Table 5. It should be mentioned that equation (7) outcome is not used in calculating the average strength value of the rock masses (σ_{cmass}, Mpa) if it was too high when compared to the other strength values [2, 3, 6, 13-17].

The strength of the basalt rock mass (σ_{cmass}) calculated by [2] from equation (14) is 10.6 Mpa which corresponds to (σ_{cmass}) calculated by the Roclab program (σ_{cmass}=10.608Mpa).

Deformation Modulus of Rock Masses: As we mentioned previously, in-situ determination of deformation modulus of rock mass (E_{mass}) is also very expensive, time-consuming and demanding. So researchers prefer to use empirical methods to estimate E_{mass} [3].

Table 1: Value of X, Y and center of tunnel's coordinate (x_0, y_0)

Value of gradient	Value of X, Y and (x_0, y_0)			
α (degree)	X (m)	Y (m)	x_0 (m)	y_0 (m)
30	230.0	125.0	102.5	65.8
35	222.9	136.0	97.1	72.5
40	214.9	146.4	91.4	79.0
45	206.1	156.1	85.4	85.4
50	196.4	164.9	79.1	91.4
55	186.0	172.9	72.5	97.1
60	175.0	179.9	65.8	102.5
65	163.4	185.9	59.0	107.4
70	151.3	191.0	52.2	111.9

Table 2: Material properties of basaltic rocks.

Properties	Value of parameters
Uniaxial compressive strength (σ_{ci}, Mpa)	40.64
Young's modulus (E, Gpa)	30.91
Poissom's ratio (v)	0.27
Unit weight (γ , KN/m^3)	25.55

Table 3: Rock mass classifications of the basaltic rock masses.

Unit	RMR	Q	GSI
Basalt	56.3/51.3	1.03	48
Rock mass quality	Fair rock	Poor rock	-

Table 4: Geomechanical properties of the basaltic rock masses.

input		output	
Parameters	Value of parameters	Rock Mass Parameters	Value of parameters
Hoek Brown Classification		**Hoek Brown Criterion**	
Uniaxial Compressive Strength of Intact rock (σ_{ci} ,Mpa)	40.64	m_b	3.903
GSI	48	s	0.0031
m_i	25	a	0.507
Disturbance Factor (D)	0	Tensile strength (σ_t,Mpa)	-0.032
		Uniaxial compressive strength of rock mass (σ_c ,Mpa)	2.177
		Strength of rock mass (σ_{cmass} ,Mpa)	10.608
		Deformation modulus (E_{rm} ,Mpa)	8390.43
		Mohr-Coulomb Fit	
		Friction angle (ϕ ,deg)	52.77
		Cohesion (c,Mpa)	0.744

Several common empirical equations are offered in Table 6 to estimate Deformation modulus of rock mass [2, 3, 6, 13-17]. Deformation modulus of rock mass (E_{mass}) value was obtained by [2], using several empirical equations and calculating their average as 7.96 Gpa while the program-calculated result is 8.39 Gpa.

In-Situ Stress: The following equation (24) for calculating amount of horizontal stress suggested by [18].

$$\sigma_h = \frac{v}{1-v}\sigma_v + \frac{\beta EG}{1-v}\left(H+1000\right) \tag{4}$$

Where β=8×10-6/°C (coefficient of linear thermal expansion), G=0.024°C/m (geothermal gradient), ν is the Poisson's ratio, E_{mass} is deformation modulus of rock mass, MPa.

The vertical stress was assumed to increase linearly with depth due to its overburden weight, as follows.

$$\sigma_h = \gamma H \tag{5}$$

Horizontal to vertical field stress ratio is Phase2 program's input in field stress properties section which is calculated as 1.774. $\left(K_0 = \frac{\sigma_h}{\sigma_v}\right)$

Numerical Modeling: Phase2 computer software, a 2D Finite Element Program is a leading and powerful tool used for modeling soil, rock and structural behavior, in the fields of geotechnical, geomechanics and in civil and mining engineering. The software provides a 2D base by using Hoek-Brown failure criterion [6, 19].

After modeling, material properties input Tables 3 and 4 are used. Remember that only one type of rock is used in all models of this paper. It should also be noted that failure criterion of Generalized Hoek-Brown is applied. After defining material, meshing stage is performed and horizontal to vertical in-situ stress ratio is defined. As said in section 2, at first, slopes stability is merely analyzed.

Table 5: The suggested empirical equations for estimating σ_{cmass}.

Author	Equations number	Equations	Notes
Hoek and Brown (1980)	(4)	$\sigma_{cmass} = \sigma_{ci}\sqrt{e^{\frac{(RMR-100)}{9}}}$ (MPa)	
Yudhbir et al. (1983)	(5)	$\sigma_{cmass} = \sigma_{ci} e^{7.65\frac{(RMR-100)}{100}}$ (MPa)	
Ramamurthy (1986)	(6)	$\sigma_{cmass} = \sigma_{ci} e^{\frac{(RMR-100)}{18.75}}$ (MPa)	
Goel (1994)	(7)	$\sigma_{cmass} = \frac{5.5\gamma Q'^{1/3}}{B^{0.1}}$ (MPa)	γ is the density of rock mass ($^{t}/_{m^3}$)
Kalamaris and Bieniawski (1995)	(8)	$\sigma_{cmass} = \sigma_{ci} e^{\frac{(RMR-100)}{24}}$ (MPa)	
Bhasin and Grimstad (1996)	(9)	$\sigma_{cmass} = \left(\frac{\sigma_{ci}}{100}\right) 7\gamma Q^{1/3}$ (MPa)	For hard rocks (σ_{ci}>100 MPa, Q>10) γ is the density of rock mass ($^{t}/_{m^3}$)
Sheorey (1997)	(10)	$\sigma_{cmass} = \sigma_{ci} e^{\frac{(RMR-100)}{20}}$ (MPa)	
Trueman (1998)	(11)	$\sigma_{cmass} = 0.5\sigma_{ci} e^{0.06RMR}$ (MPa)	
Aydan and Dalgic (1998)	(12)	$\sigma_{cmass} = \frac{RMR}{RMR + \beta(100 - RMR)}\sigma_{ci}$ (MPa)	$\beta = 6$
Barton (2000)	(13)	$\sigma_{cmass} = 5\gamma(Q\frac{\sigma_{ci}}{100})^{1/3}$ (MPa)	
Hoek et al. (2002)	(14)	$\sigma_{cmass} = \sigma_{ci}\frac{(m_b + 4s - a(m_b - 8s))(m_b/4 + s)^{a-1}}{2(1+a)(2+a)}$ (MPa)	

σ_{cmass} = is the strength of rock mass (MPa)

σ_{ci} = is the strength of intact rock (MPa)

RMR = rock mass rating.

Q = rock mass quality.

GSI = geotechnical strength index.

Q' = $(\frac{RQD}{J_n})(\frac{J_r}{J_a})$

m_b, a and **s** = rock mass constants.

Table 6: Proposed equation for calculating deformation modulus of rock mass (E_{mass}).

Researchers	Equation no.	Equation	Notes
Bieniawski (1978)	(15)	$E_{mass} = 2RMR - 100\ (GPa)$	For RMR > 50
Serfim and Pereira (1983)	(16)	$E_{mass} = 10^{\frac{(RMR-10)}{40}}\ (GPa)$	For RMR < 50
Grimstad and Barton (1993)	(17)	$E_{mass} = 25LogQ\ (GPa)$	For Q > 1
Mitri et al. (1994)	(18)	$E_{mass} = E_i[0.5\left(1 - \left\{cos\pi\ \frac{RMR}{100}\right\}\right)]\ (GPa)$	
Hoek and Brown (1998)	(19)	$E_{mass} = \sqrt{\frac{\sigma_{ci}}{100}}10^{(\frac{GSI-10}{40})}\ (GPa)$	For poor rock ($\sigma_{ci} < 100\ Mpa$)
Read et al. (1999)	(20)	$E_{mass} = 0.1(\frac{RMR}{10})^3\ (GPa)$	
Barton (2002)	(21)	$E_{mass} = 10Q_c^{1/3}\ (GPa)$	
Hoek et al. (2002)	(22)	$E_{mass} = (1 - \frac{D}{2})\sqrt{\frac{\sigma_{ci}}{100}}10^{(\frac{GSI-10}{40})}$	
Hoek and Diederichs (2006)	(23)	$E_{mass} = E_i(0.02 + \frac{1}{1 + e^{(60+15D-GSI)/11}})$	

σ_{ci} = is the strength of intact rock (MPa)

RMR = rock mass rating.

Q = rock mass quality.

GSI = geotechnical strength index.

Q_c = rock mass quality rating or normalized $Q = Q\frac{\sigma_c}{100}$

E_i = Young's modulus.

D = disturbance factor.

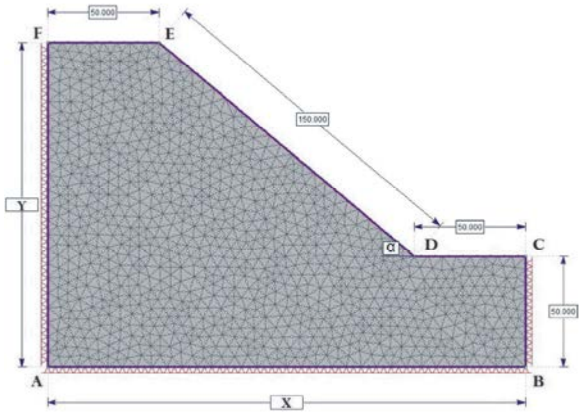

Fig. 1: Numerical modeling of rock slope (in this Fig. $\alpha = 40°$).

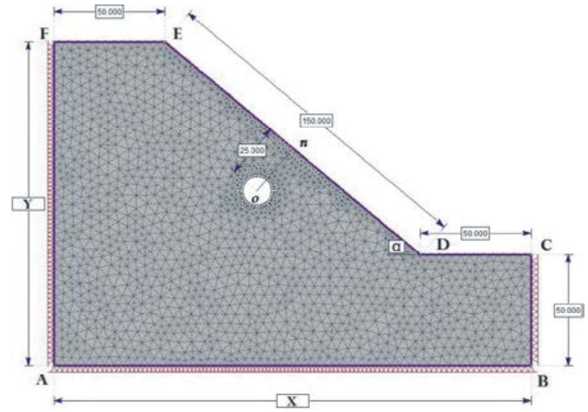

Fig. 2: Numerical modeling of excavated tunnel in rock slope (in this Figure, $\alpha = 40°$ and radius = 7 m).

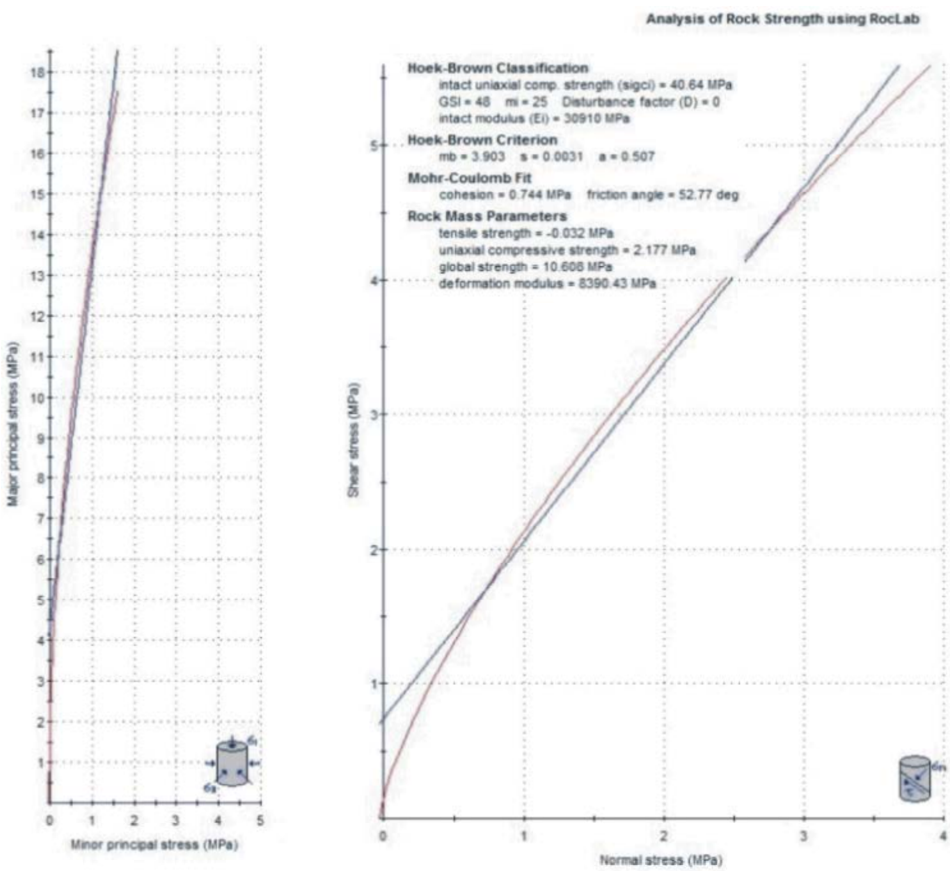

Fig. 3: Comparison of Hoek-Brown and Mohr-coulomb‘s failure curves.

A sample of computed model is shown in Fig. 4. Also, obtained Strength Reduction Factor (SRF) of each analyzed slope is shown in Fig. 5 graph.

The required SRF is dependent on the subsequences of losses related to property, lives and reconstruction costs if the slope failed. SRF also depends on the reliability of design parameters. The minimum SRF suggested by different regulations vary from 1.2 to 1.5.

Assuming minimum value of SRF as 1.5, it is completely obvious that all slopes are stable according to Fig. 4.

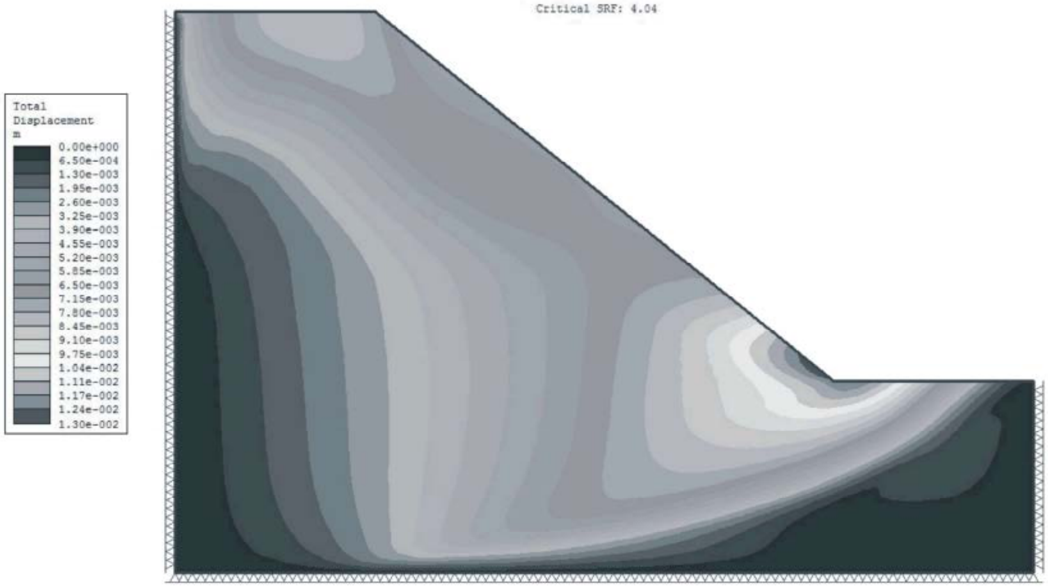

Fig. 4: Amount of slope's critical SRF before tunnel excavation (in this Fig α = 40° and SRF=4.04).

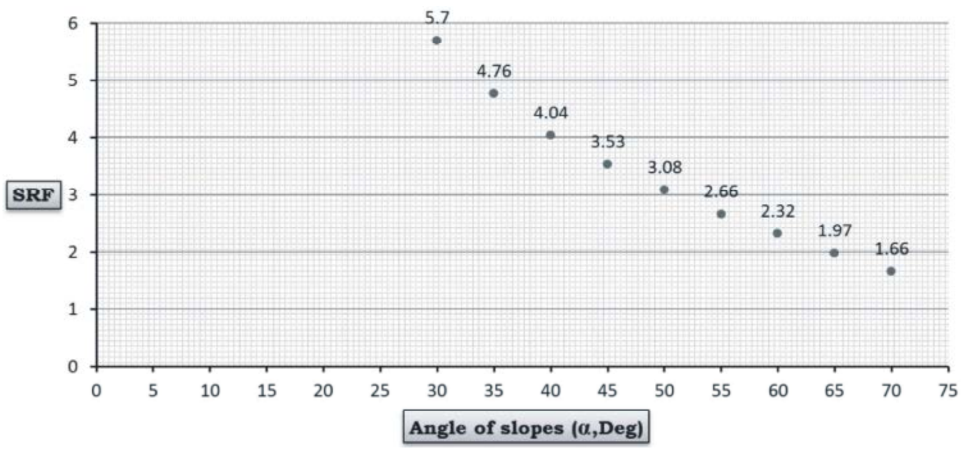

Fig. 5: Value of Strength Reduction Factor (SRF) of each analyzed slope.

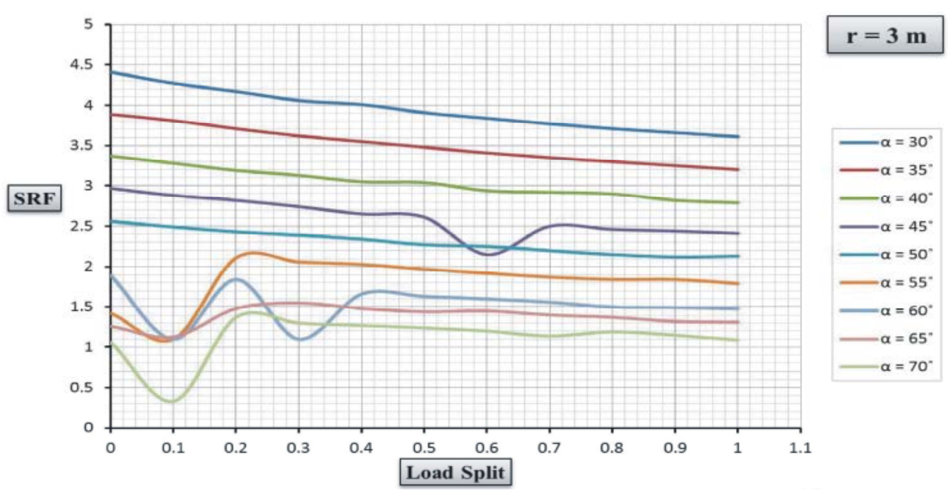

Fig. 6: Slopes behavior in stress reduction stages (load split) at radius 3 m.

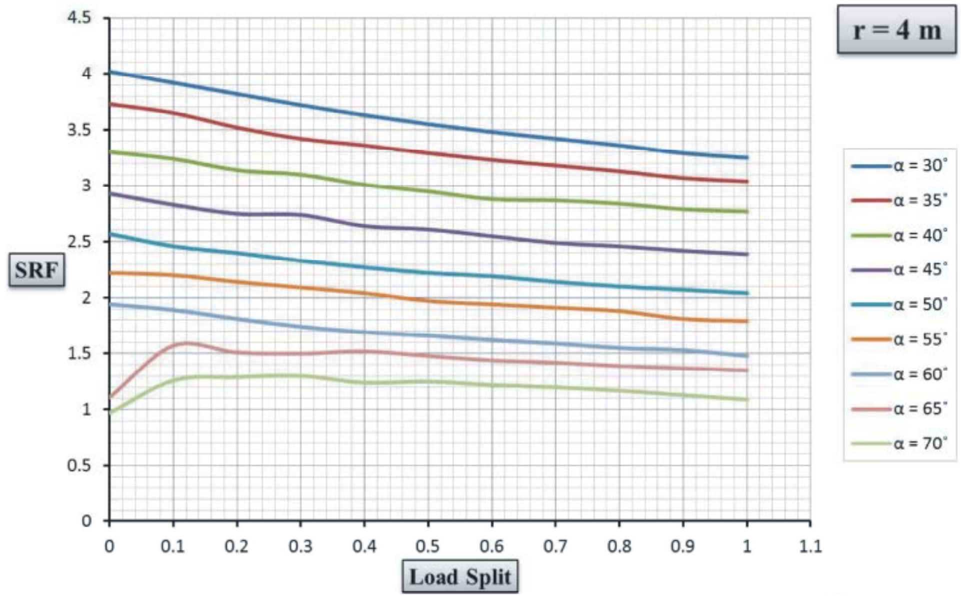

Fig. 7: Slopes behavior in stress reduction stages (load split) at radius 4 m.

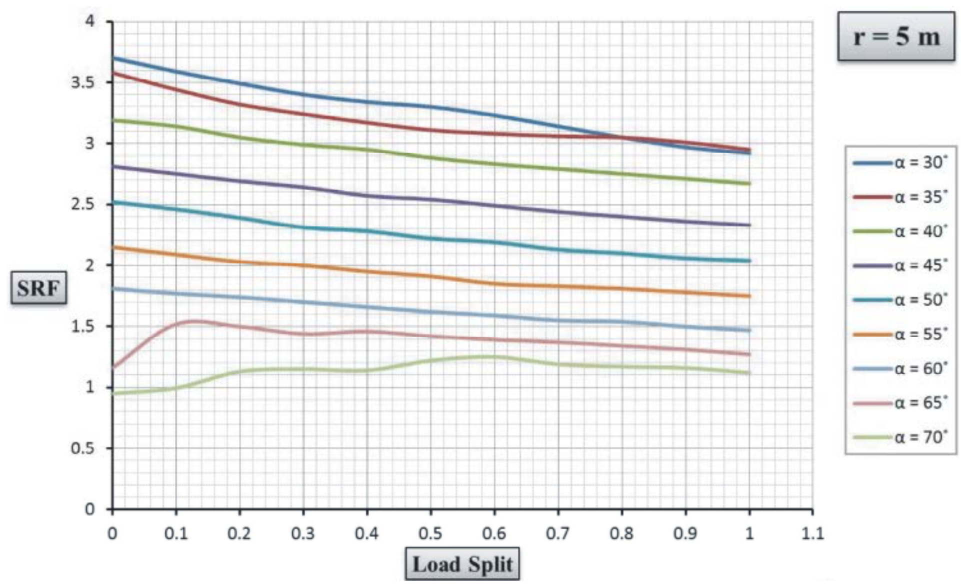

Fig. 8: Slopes behavior in stress reduction stages (load split) at radius 5 m.

After examining slopes stability, tunnels will be modeled. Fig. 2 shows the model up to this stage.

In each slope, tunnel with supposed radius is modeled and each radius with 10% stress releasing stage has been analyzed. For instance, as seen in Fig. 2, at angle of 40 degree, tunnel has been modeled with 7 m radius. Then at first stage, 10% value of stress was released and after that its effect on slope stability and SRF change was recorded. This process was repeated with 20, 30, 90 and 100% stress releasing under same circumstances and SRF was also recorded. Above mentioned process was repeated for each following radius 3, 4, 5, 6 and 7 m respectively.

According to the following graphs, in every constant radius, function of slopes has been evaluated so that in each radius, different slopes behavior in stress reduction stages are shown in Fig. 6, 7, 8, 9, 10.

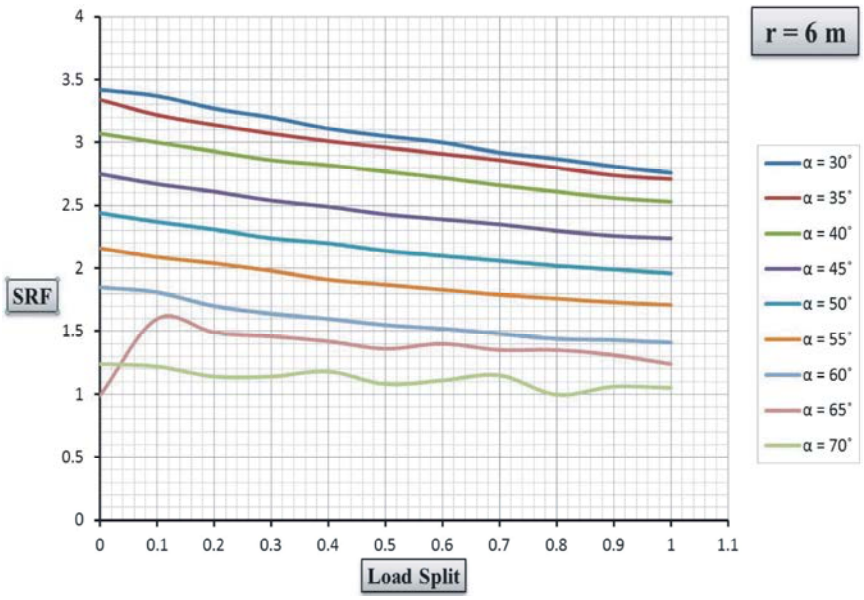

Fig. 9: Slopes behavior in stress reduction stages (load split) at radius 6 m.

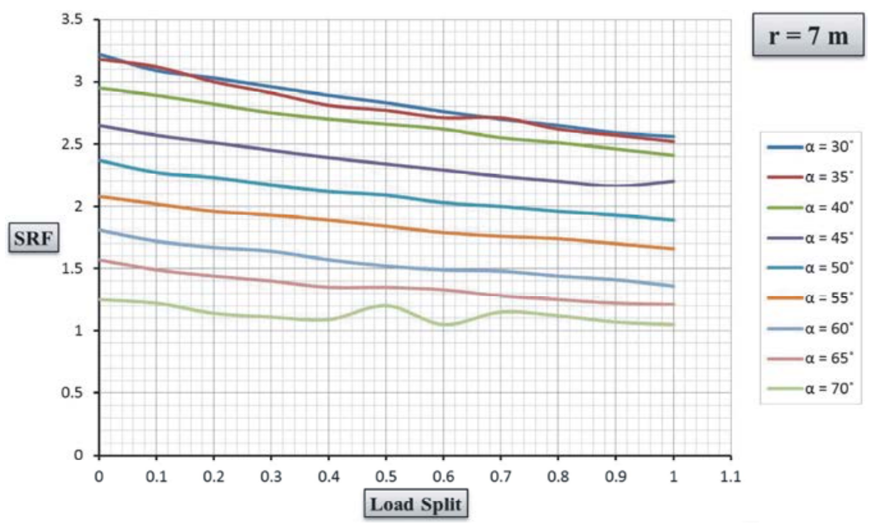

Fig. 10: Slopes behavior in stress reduction stages (load split) at radius 7 m

CONCLUSIONS

This paper's objective is to study the effect of released stress' value on excavated tunnels in slopes and those slopes stability. Here the results follow.

- An increase in slopes angle reduces SRF value and slope stability.
- An increase in radius of excavated tunnels in slopes reduces SRF value and slope stability.
- A change in SRF is relevant to stress releasing process, in a way that an increase in releasing stress reduces value of SRF and slopes stability.

REFERENCES

1. Hoek, E. and J.W. Bray, 1981. Rock slope engineering, 3rd ed. Institute of Mining and Metallurgy. pp: 358.
2. Gurocak, Z., S. Pranshoo and M. Zaman, 2007. Empirical and numerical analyses of support requirements for a diversion tunnel at the Boztepe dam site, eastern Turkey. Eng. Geol., 91: 194 - 208.
3. Basarir, H., A. Ozsan and M. Karakus, 2005. Analysis of support requirements for a shallow diversion tunnel at Guledar dam site, Turkey. Eng. Geol., 81: 131-145.

4. Hoek, E., P.K. Kaiser and W.F. Bawden, 1995. Support of underground excavation in hard rock. Balkema, Rotterdam. pp: 215.
5. Karahan, E., 2010. Design of excavation and support systems for the cubukbeli tunnel in Antalya, Turkey. pp: 116.
6. Genis, M., H. Basarir, A. Ozarsan and E. Balaban, 2007. Engineering geological appraisal of the rock mass and preliminary support design, Dorukhan tunnel, Zonguldak, Turkey. Eng. Geol., 92: 14-26.
7. David Hume, C., 2011. Numerical validation and refinement of empirical rock mass modulus estimation. Eng. Geol. Canada, Ontario. pp: 169.
8. Hoek, E. and E.T. Brown, 1980. Underground excavations in rock. Inst. Min. Metall. London. pp: 527.
9. Bieniawski, Z.T., 1989. Engineering rock mass classifications. Wiley, New York.
10. Barton, N.R., R. Lien and J. Lunde, 1974. Engineering classification of rock masses for the design of tunnel support. Rock Mech., 4: 189-239.
11. Rocscience, 2002. Roclab user's guide. Rocscience Inc. Toronto, Ontario, Canada.
12. Hoek, E., C. Carranza-Torres and B. Corkum, 2002. Hoek-Brown failure criterion - 2002 Edition. In: Hammah, R. Bawden, W. Curran, J. Telesnicki, M. (Eds.), Proceedings of Narmstac 2002, Mining Innovation and Technology.
13. Sonmez, H. and R. Ulusay, 2002. A discussion on the Hoek-Brown Failure.
14. Criterion and suggested modifications to the criterion verified by slope stability case studies. Turkey. Ankara. Geo. Eng., 26: 77- 99.
15. Hoek, H., M.S. Diederichs, 2005. Empirical estimation of rock mass modulus. Rock Mech., 43: 203-215.
16. Ozsan, A., H. Basarir and M. Cilsal, 2006. Engineering Geological Investigations.
17. Along the Ankara subway extension. IAEG. pp: 586.
18. Kaya, A., F. Bulut, S. Alemdağ and A. Sayin, 2011. Analysis of support requirements for a tunnel portal in weak rock: A case study from Turkey. Sci. Res., 31: 6566-6583.
19. Khabbazi, A., M. Ghafoori, G.R. Lashkaripour and A. Cheshomi, 2012. Estimation of the rock mass deformation modulus using a rock classification system, Geomechanics and Geoengineering, iFirst, pp: 1-7.
20. Sheorey, P.R., G. Murali Mohan and A. Sinha, 2001. Influence of elastic constants on the horizontal in-Situ stress. Rock Mech., 38: 1211-1216.
21. Cai, M., 2008. Influence of stress path on tunnel excavation response - numerical tool selection and modeling strategy. Tunnel Tech., 23: 618-628.