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Evaluation of Effective Parameters on the Underground Tunnel Stability Using BEM

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Abstract

There are various parameters that affect stability and expansion of failure zones in under pressure tunnels. Among the important parameters that affect failure zones around the tunnels are cohesion and internal friction angle of the rock mass. In addition, the cross sectional shape is the considerable point in failure distribution around the tunnels. The stress analysis method is one of the applicable methods for evaluating stability and recognizing failure zones in underground tunnels. On the other hand, numerical stress analysis method, because of obtaining results with simplicity and desirable accuracy, is one of the best methods in stability evaluation of the tunnels. The Boundary Element Method (BEM) has unique advantages in stability analysis of infinite continuums. In this paper, using Hoek-Brown failure criteria and also BEM, failure zones around the tunnels with various section shapes are studied numerically and the effects of cohesion and internal friction angle of rock mass on formation and distribution of failure radii are evaluated. It deserves mentioning that the behavior of the rock mass around the tunnels is assumed to be elastic and the formulations are based on plane strain.

***Key Words*:** Tunnel; failure zone; Hydrostatic; BEM; Hoek Brown criteria; stability.

1 Introduction

The first step in designing any kind of maintenance system in underground tunnels is to study stability condition of rock mass around them. The purpose of stability analysis in the underground structures is to predict stability condition of their various parts in order to determine states which are: 1) stable without support system, 2) stable with support system and 3) permanently unstable.

Methods for studying stability in tunnels are classified into three categories: experimental, laboratory and analytical. Stress analysis method is one of the most important analytical methods which have the advantages of simplicity and high speed of operations**.** Generally, the regions around the tunnel where the ratio of strength to stress (strength factor) is smaller than safety factor and its stability is controlled by installation of support system are called failure zone. This zone is different for various shapes of tunnel section and statuses of in situ stresses.

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Since 1980, boundary element method has been used for analyzing rock mechanic problems. Banerjee & Butterfield and Crouch & Starfield studied opening model in unlimited space using BEM in 1981 and 1983 respectively [1, 2]. At that time, i.e. from 1980 to 1983, Hoek & Brown presented a criterion known as Hoek-Brown Failure Criteria for intact rocks.

Their completed failure criteria for all rocks in the intact and jointed states were presented in 1992 [3].

In spite of extensive researches and studies performed in the field of stability evaluation of tunnels, there are some ambiguities in recognizing failure zones around different sections. In this paper, at first failure distribution around a tunnel under hydrostatic pressure based on Hoek-Brown failure criteria is evaluated, then the effects of rock strength parameters (cohesion and friction angle) and the tunnel depth on failure radius are studied and some approximated equations are suggested for estimating failure radius for various tunnel sections.

2. Rock Mass Strength

One of the most important problems in designing all underground spaces (such as tunnels) is to know the strength parameters of the rock mass. Although rock mass strength around the shallow tunnels (such as road and railway tunnels) is affected by structural and weathering conditions, in underground deep tunnels, determination of rock mass strength requires the results obtained from field investigations, laboratory tests, and analytical studies.

In this study, Roclab software, version 4 is used to determine rock mass strength parameters. This software is based on Hoek-Brown Failure Criteria which yields values of strength (compression and tension), deformation modulus, cohesion, and internal friction angle of the rock mass. General equation of Hoek-Brown failure criteria is as follows [4]:

$$\sigma_1 = \sigma_3 + \sigma_{ci}(m_b \frac{\sigma_3}{\sigma_{ci}} + s)^a \tag{1}$$

Where σ_1 stands for major principle stress, σ_3 indicates minor principle stress (confined stress), σ_{ci} represents uniaxial compression strength of intact rock mass, m_b is fixed factor dependent on properties of in situ rock mass, and s and a represent coefficients which depend on jointed condition of rock mass.

And s, a, and m_b are fixed coefficients of Hoek-Brown failure criteria which can be calculated by Roclab software. It is worth mentioning that, to determine these coefficients, Geological Strength Index (GSI), obtained on the basis of other rocks engineering classification indices i.e. RMR and Q, is used. The value of GSI varies from 5 for very weak rocks to 100 for intact rocks [5].

Appling $\sigma_3=0$ in equation (1), uniaxial decreased compression strength of rock for different condition of jointed rocks is obtained as follows:

$$\sigma_c = \sigma_{ci}.s^a \tag{2}$$

In this regard, if $\sigma_1=0$, and equation (1) is solved in terms of σ_3, uniaxial tension strength of rock will be according to equation (3):

$$\sigma_t = \frac{1}{2}(-\sigma_{ci}{}^{(1/a)-1} m_b + \sqrt{m_b^2 \sigma_{ci}^{2((1/a)-1)} - 4\sigma_{ci}^{1/a} s}\ ) \tag{3}$$

In 1983, Hoek showed that for brittle rocks uniaxial tension strength of rock equals biaxial tension strength[6]. It means that if $\sigma_1 = \sigma_3 = \sigma_t$, the results of equation (3) will be converted into a simple form as follows:

$$\sigma_t = \frac{-s\sigma_{ci}}{m_b} \tag{4}$$

In figure (1), the curve of Hoek-Brown failure criteria is shown. In fact, equations (2) and (3) are respectively intersection points of Hoek-Brown curve with directions of major and minor principle stresses.

It is worth mentioning that in most studies and numerical equations dominant on rock mechanics and geotechnical problems, to determine failure criteria, the cohesion and internal friction angle of the rock mass parameters have been considered. As shown in figure (1), to determine these values (Mohr-Colomb failure Criteria factors), by fitting smooth lines on Hoek-Brown curve, one can obtain cohesion (C) and internal friction angle of the rock mass (Φ) according to equations (5) and (6):

$$C = \frac{\sigma_{ci}\left[(1+2a)s + (1-a)m_b\sigma_{3n}\right](s + m_b\sigma_{3n})^{a-1}}{(1+a)(2+a)\sqrt{1 + \left(6am_b(s + m_b\sigma_{3n})^{a-1}\right) \big/ \left((1+a)(2+a)\right)}} \tag{5}$$

$$\phi = \sin^{-1}\left[\frac{6am_b(s + m_b\sigma_{3n})^{a-1}}{2(1+a)(2+a) + 6am_b(s + m_b\sigma_{3n})^{a-1}}\right] \tag{6}$$

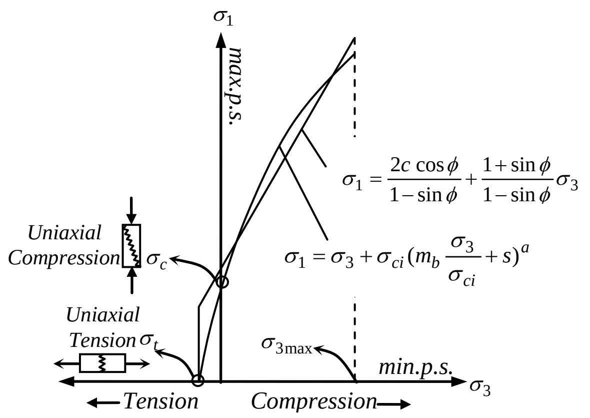


Figure 1. Relation between Hoek-Brown major and minor principle stresses and equivalency with Mohr-Colomb Criteria [4]

Where $\sigma_{3n}= \sigma_{3max}/\sigma_{ci}$. Determining compression and tension strengths of rock mass, strength factor around tunnel with regard to its analytic results can be obtained.

3. Numerical Studies

In order to evaluate failure zone in tunnels and study effective parameters, a tunnel made of jointed dolomite with GSI=65 and unit weight of γ=2.85 ton/m^3 has been considered in depth of 100 meters under ground level. Rock mass properties are shown in intact condition (without joint) in figure (2).

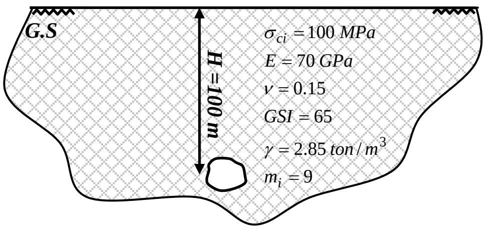


Figure 2. Properties of rock mass around tunnel

Where, m_i is related to properties of in situ rock mass in intact condition. For comparative study of shape section in failure behavior, and with regard to extensive use of regular geometric sections, tunnel with circular, horseshoe, quadrangular and rectangular section shapes has been considered (figure 3).

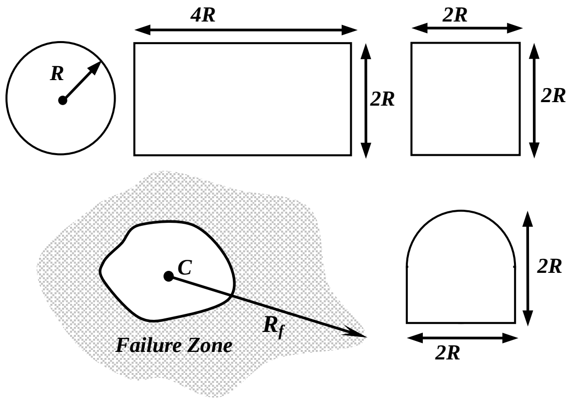
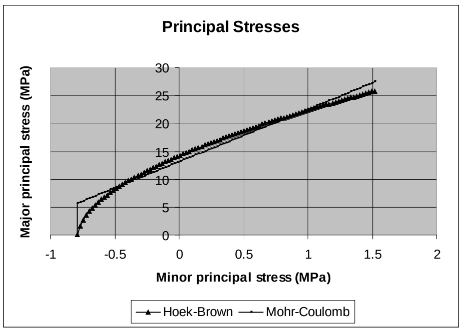


Figure 3. Assumed tunnel sections for determining failure zones

3.1. Results of Roclab Program

All necessary information for recognizing failure zone around the tunnels such as uniaxial compression and tension strengths in weak conditions are obtained from the output of Roclab Program. For the assumed rock mass, Hoek-Brown curve with Mohr-Colomb fitting lines are shown in figure (4.a.). From horizontal and vertical axes of this diagram, uniaxial tension (σ_t=-0.793784MPa) and uniaxial compression strengths (σ_c=14.1972 MPa) of the assumed rock mass have been specified.

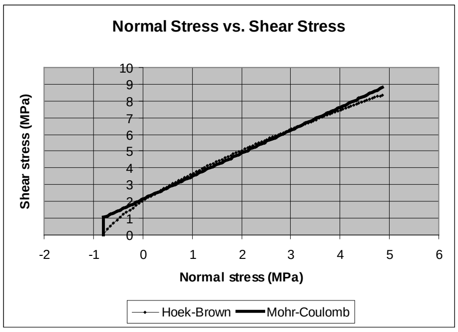
Figure (4.b.) represents cohesion value (C) and internal friction angle (Φ), obtained from Mohr-Colomb fitting curve.



(a)

Hoek Brown Classification:
sigci100Mpa
GSI65
mi9
D0
i70000

***Failure Envelope Range**:*
Application Tunnels
sig3max 1.52067 MPa
Unit Weight 0.0285 MN/m3
Tunnel Depth 100 m



(b)

Hoek Brown Criterion:
mb 2.57854
s 0.0204681
a 0.501975

Mohr-Coulomb Fit:
c 2.13771 MPa
phi 53.8938 degrees
Rock Mass Parameters:
Sigt -0.793784 MPa
Sigc 14.1972 MPa

Figure 4. Determination of necessary parameters for dolomite rock

4. Evaluation of Failure Zones

Main purpose of this research is to study failure zones and determine their radius distribution around underground tunnels. In this regard, software based on BEM was provided so that in addition to the stress analysis of the tunnel, its failure zones were determined. For more information about BEM see references [7-11].

This computer program is prepared in MATLAB software and compiled in two main parts. The first part (the longer part of the software) consists of six subprograms. This part includes an input file which obtains its values from discretization of the zones around the tunnel with three-node boundary elements, and stress analysis file for determining unknown parameters of the problem.

The second part of the software, receiving the results of the first part (stresses) and considering assumed safety factors, tension and compression strengths of the rock mass, type of tunnel section, and also assuming type of stress condition of the tunnel surrounding (including major and minor principle stresses or vertical, horizontal or shear induced stresses), evaluates and determines tension and compression failure zones around the tunnel graphically.

4.1. Verification

To compare the results of boundary elements analysis with analytical responses [12], figure (5) is given. In this figure, the total displacement values in different distances from the circular tunnel due to pressures of gravity (γH), in two different modes of soil lateral pressure coefficient (*k*), are compared with analytical solutions and EXAMINE^{2D} software results[13]. Figure (5) also shows how the loading is applied.

Tension strength of the rock masses is much lower than their compression strength. For the mentioned rock mass in figure (4), compression strength was estimated to be 18 times more than tension strength. This fact shows that in underground tunnels (especially deep tunnels) failure occurs in tension zones and contour curves of minor principle stress indicate tension behavior around the tunnel.

It is worth mentioning that, in underground deep tunnels, although the maximum value is compression stress, due to high compression strength of rock mass, minor stress curves are dominant for determining critical failure zones and strength factors.

Figures (6.a.) to (6.d.) show failure zones around the various sections of all kinds of tunnel for the under study rock mass. In these figures, the most critical condition of failure zones (which is of tension failure), has been marked with grey color. In circular tunnel, maximum failure radius was observed in angles of 45 degrees and the failure is expanded in these directions too. Horseshoes tunnel in crest shows a condition similar to that of circular tunnel; however, it is more critical than that. Formation of failure zones in this kind of geometry (which is a combination of circle and quadrangle) follows its geometry, i.e. failure in walls and floor occur along with expansion of failure in quadrangular section (figure 6.c.) and is the same to circular section in crest. Among the sections under study, the rectangular section in crest and base, because of its larger opening in length compared with width, has the most expansive failure zone. This section has the least expansion of failure in its width, but in crest and base, the failure has extended in surrounding to 4R.

Although maximum value of stress is found in corners, in rectangular and quadrangular sections, there is not the most critical value of stress concentration, because this stress is compression type and with regard to high compression strength of rock mass, there is little failure in these zones. These results are applicable when effective in situ stresses are imposed on tunnel across principle axes.

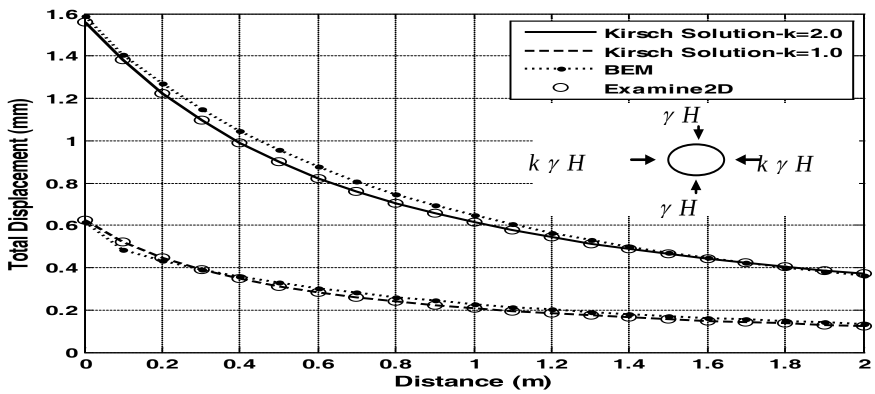
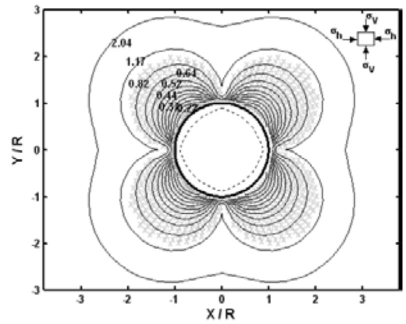
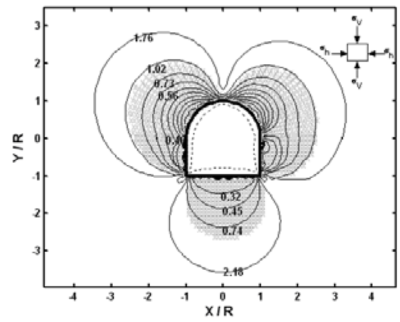


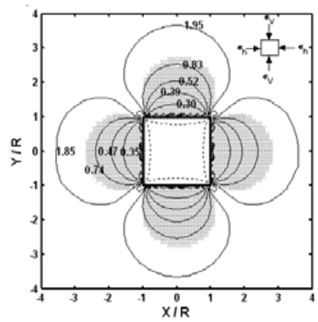
Figure 5. Comparison of total displacement values in different distances from the tunnel wall between analytical Solutions [12] , EXAMINE^{2D} software[13] and BEM(present study)



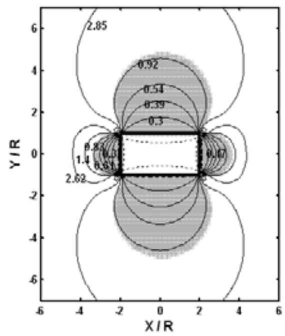
a. Circular Tunnel Under Hydrostatic Pressure



b. Horseshoe Tunnel Under Hydrostatic Pressure



c. Square Tunnel under Hydrostatic Pressure



d. Rectangular Tunnel Under Hydrostatic Pressure

Figure 6. Failure zones around various sections of tunnel under hydrostatic compression

5. Effects of Cohesion and Internal Friction Angle

The most important effective parameters in rock masses strength and comprehensively expansion of the failure zones are cohesion and internal friction angle of the rock material around the tunnels. These parameters, which are dependent on rock shear strength and are resulted from Mohr-Colomb failure curve, indicate jointed texture of rock masses. It means that by changing the direction and depth of joints, these parameters give different values.

In figure (7) the behavior of cohesion(C) and internal friction angle (Φ) of the rock mass under study versus GSI is shown. As seen, the behavior of these parameters varies in different condition of rock mass texture. With improving and decreasing the joints, the cohesion C increases while the friction angle (Φ), in higher values of GSI, decreases. Therefore, because of sensitivity and deference of the behaviors of these parameters in higher values of GSI, their effects on rock mass texture in these areas are studied and compared (figure 7).

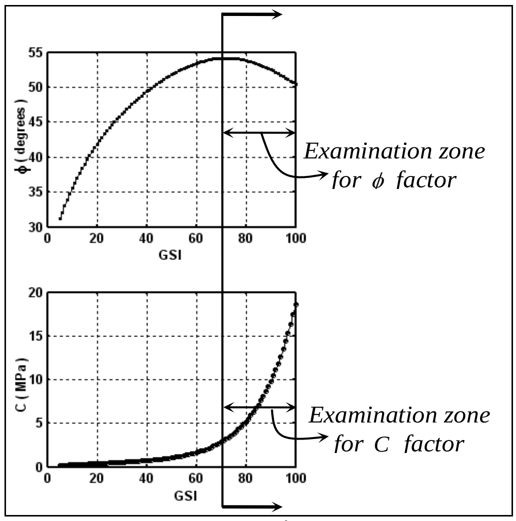
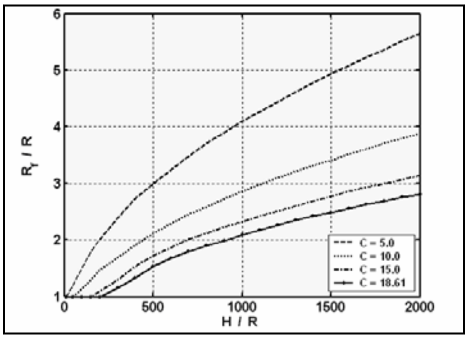


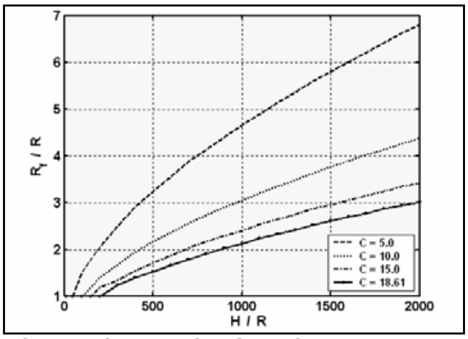
Figure 7. Parameters C and Φ in various conditions of rock mass textur

5.1 Effects of Cohesion (C)

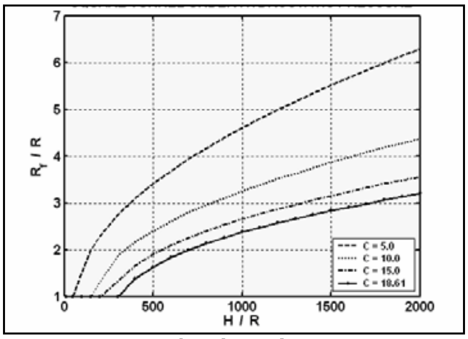
The rock mass has been studied for different tunnel section shapes and depths (figure 8). The effect of cohesion has been studied in four values of 5, 10, 15, and 18.61Mpa. The value of 18.61Mpa is the maximum cohesion of the rock mass under study and belongs to its intact condition i.e. GSI=100.



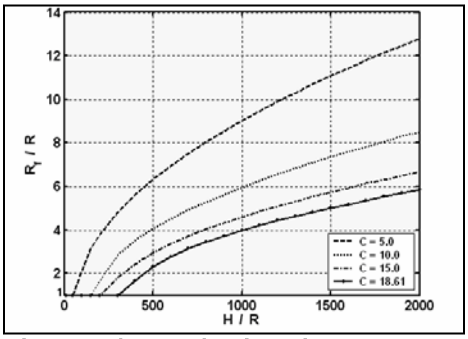
a. Circular Tunnel Under Hydrostatic Pressure



b. Horseshoe Tunnel Under Hydrostatic Pressure



c. Square Tunnel Under Hydrostatic Pressure



d. Rectangular Tunnel Under Hydrostatic Pressure

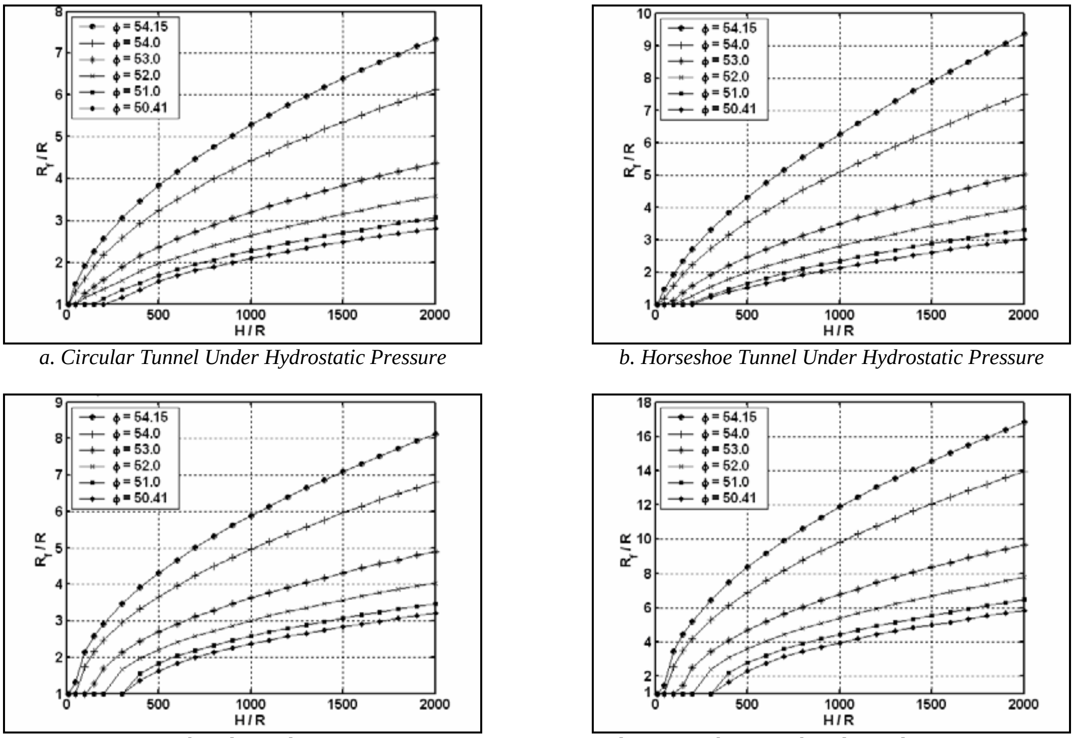
Figure 8. Expansion of failure radius in various cohesion values

In low cohesion values (C=5Mpa), compared to other values, the failure zones and their radial expansion(R_f) start in depths less than 50R for various sections, but with increase in cohesion, these depths reach to approximately 200R to 300R. As it can be seen, with changing the cohesion from 5 Mpa to 10 Mpa, failure radius changes decreased considerably, but from 10 Mpa to 15 Mpa, failure radius changes decreased in comparison to the previous state. This means that the distance of curves for C=5 Mpa to C=10 Mpa is more than that of C=10 Mpa to C=15 Mpa for various sections of tunnel. It could be concluded that in cohesions close to intact state of rock, the change in failure radius is low and when it is far from intact state, expansion of failure radius increases considerably.

5.2 Effects of Internal Friction Angle (Φ)

Because of great significance of Φ parameter in/ great sensitivity of Φ parameter to failure behavior, its effects are studied in six various values (figure 9). These values belong to falling curvature in figure (7). The maximum value ($\Phi=54.14^o$) is for turning point of the curvature and the minimum value ($\Phi = 50.41^o$) is the friction angle of the intact rock mass. As seen in various tunnel sections, the importance of this parameter, even in its low changes**,** is clear. For example with a 0.15^o change in Φ value, the gradient of failure curves in various depths decrease to 17%.

The decrease in percentage of failure curves gradient versus changes of Φ is presented in table (1). Considering the values of this table and figure (9), it can be concluded that with a decrease in Φ value, the failure curves gradient decreases too. It means that with decreasing internal friction angle of rock mass, the curve gradient decreases too, but does not have the same rate in various depths and various sections. In lower depths, decreasing rate is higher, and with increasing the tunnel depth, curves descending gradient diminishes too. So it can be stated that the friction angle changes, in lower tunnel depths, are more effective in improving failure behavior.



a. Circular Tunnel Under Hydrostatic Pressure *b. Horseshoe Tunnel Under Hydrostatic Pressure*

c. Square Tunnel Under Hydrostatic Pressure *d. Rectangular Tunnel Under Hydrostatic Pressure*

Figure 9. Expansion of failure radius in various condition of Φ

Table 1. Decrease percentage of failure curve gradient (R_f / H) in various depths for equal changes of Φ

TUNNEL SECTION	*H*	*500R-1000R*			*1000R-1500R*			*1500R-2000R*		
	ϕ°	*54-53*	*53-52*	*52-51*	*54-53*	*53-52*	*52-51*	*54-53*	*53-52*	*52-51*
CIRCULAR TUNNEL		*29.4*	*20.2*	*10.4*	*31.5*	*19.0*	*17.6*	*29.8*	*20.3*	*13.9*
HORSESHOE TUNNEL		*33.5*	*21.3*	*16*	*35.4*	*20.7*	*16.9*	*36.8*	*23.6*	*20.0*
SQUARE TUNNEL		*28.4*	*13.9*	*7.5*	*31.0*	*18.8*	*12.5*	*30.5*	*20.3*	*14.9*
RECTANGULAR TUNNEL		*28.0*	*15.2*	*7.8*	*29.6*	*17.8*	*13.9*	*29.9*	*18.3*	*14.9*

6. Subject Generalization

The ratio of major effective in situ stress (Max (σ_h,σ_v)) to uniaxial compression strength (σ_{ci}) can be considered as a main index for the evaluation of tunnel stability. A failure radius versus various states of in situ stresses has been estimated approximately in figure (10). These estimated equations for various section shapes are presented as follows:

Circular Tunnel:

$$\frac{R_f}{R} = -1.7366(\frac{\sigma_v}{\sigma_{ci}})^2 + 4.4239(\frac{\sigma_v}{\sigma_{ci}}) + 0.8777 \qquad (7)$$

Horseshoe Tunnel:

$$\frac{R_f}{R} = -1.5012(\frac{\sigma_v}{\sigma_{ci}})^2 + 4.6448(\frac{\sigma_v}{\sigma_{ci}}) + 0.8494 \qquad (8)$$

Square Tunnel:

$$\frac{R_f}{R} = -2.359(\frac{\sigma_v}{\sigma_{ci}})^2 + 5.5135(\frac{\sigma_v}{\sigma_{ci}}) + 0.8516 \qquad (9)$$

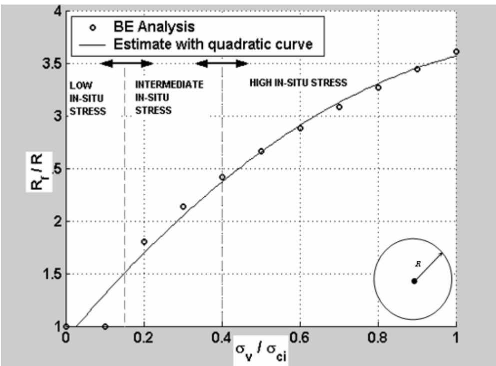
Rectangular Tunnel:

$$\frac{R_f}{R} = -5.0163(\frac{\sigma_v}{\sigma_{ci}})^2 + 12.0472(\frac{\sigma_v}{\sigma_{ci}}) + 0.6685 \qquad (10)$$

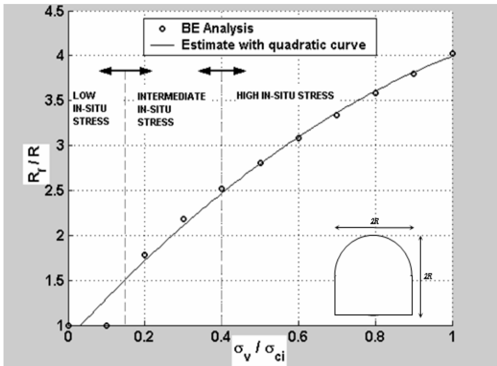
Experimental classification for values σ_v/σ_{ci} shows that around the tunnel, for values of in situ stresses, if $\sigma_v/\sigma_{ci} \leq 0.15$, there is no failure and behavior of tunnel is linear and there is no need to support system [14]. This result can be obtained for circular, horseshoes, and quadrangular sections too, but as shown in figure (10.d.), in rectangular section, failure has penetrated to 2.5R into surrounding tunnel.

In medium in situ stresses i.e. $0.15<\sigma_v/\sigma_{ci}\leq 0.4$, around tunnel, lamination phenomenon has occurred parallel to stresses σ_v, and equipped support system is needed. As depicted in figures (9.a.) to (9.d.), in this stress range, the behavior of failure radius is relatively linear and a suitable support system for its stability could be designed according to experimental theory. In major in situ stresses value ($\sigma_v/\sigma_{ci} > 0.4$), the installation of support system is expensive and in special cases ($\sigma_v/\sigma_{ci} > 0.6$), the maintenance of the tunnel will usually be impossible. This fact can be concluded from figures so that for values $\sigma_v/\sigma_{ci} > 0.4$, estimated curves have more curvature than the previous intervals and it means that prediction of the expansion of failure zone, in high values of in situ stresses, is difficult.

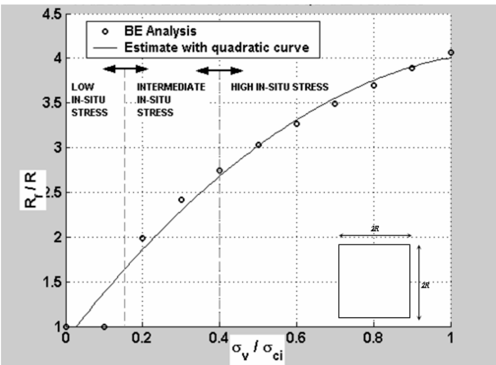
Figure (11) shows the effects of tunnel section type on failure radius expansion. It shows that circular, horseshoes, quadrangular and rectangular section shapes are more suitable for decreasing failure radii, respectively.



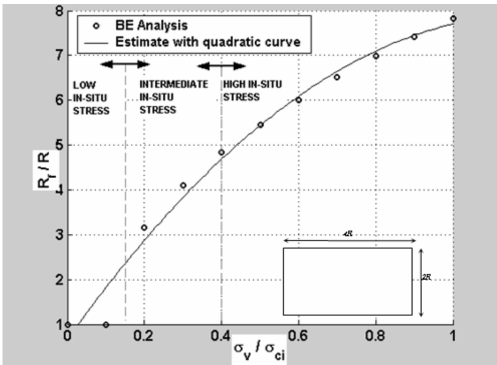
a. Failure Radius Estimating in Circular Tunnels



b. Failure Radius Estimating in Horseshoe Tunnels



c. Failure Radius Estimating in Square Tunnels



d. Failure Radius Estimating in Rectangular Tunnels

Figure 10. Failure behavior in various sections of tunnel for various values of insitu stresses

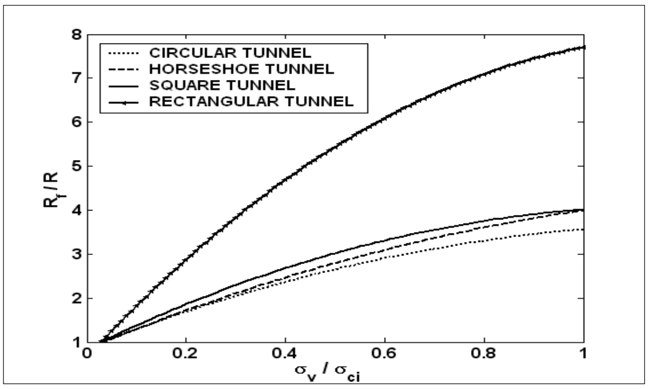


Figure 11.Comparison of tunnel section type in expansion of failure

7. Conclusions

Tunneling is one of the processes which entails engineering judgments during its construction and always has high risk level. Therefore, precise and accurate evaluations on the project, before construction stages, can reduce risks and prevent incorrect engineering judgments. Incorrect estimation of a parameter will affect the behavior of the tunnel considerably and negligence of it will lead to irreparable damages. Therefore, accurate studies, continual evaluations, and use of various parameters are the most primary actions which should be considered before starting the tunnel construction. Determination and prediction of failure zones are important in defining the behavior of the tunnels, in terms of either efficiency or economy.

The results obtained from this research are classified as follows:

1) The failure in rock masses occurs in tension zones due to low tension strength and opening resulting from construction of the tunnel, and minor principle stress indicates its behavior.
2) By trying to prevent wide opening like rectangular section, failure zones around tunnels can be limited up to 50%.
3) According to failure expansion up to 2.5 times its radius, circular section is the best section for undergoing hydrostatic pressures.
4) Cohesion and internal friction angle is very effective on failure behavior of rock masses. So it is suggested to determine these parameters accurately and avoid excavating tunnels in rock masses with too jointed texture.

References

[1] Banerjee, P.K. and Butterfield , R.. (1981). "Boundary Element Methods in Engineering Science"; McGraw-Hill, London, Russian Translation, Mir Publishers, (1984), Moscow; Chinese Translation, National Defense Press, Beijing, (1988).

[2] Crouch, S.L. and Starfield, A.M. (1983). "Boundary elements method in solid mechanics", Unwin Hyman Ltd , London.

[3] Hoek, E. and Marions, P. (2007). "A brief history of the development of the Hoek-Brown failure criterion"; Soil and Rocks, No. 2, pp.1-13.

[4] Hoek, E., Carranza-Torres, C. and Corkum, B. (2002). "Hoek-Brown failure criterion-2002 edition"; Vancouver Canada/ Minneapolis,USA/ Toronto, Canada.

[5] Hoek, E., Kaiser, P.K. and Bawden, W.F. (1995). "Support of underground excavation in hard rocks"; A.A.Balkeman/Rotterdam/Brookfield.

[6] Hoek, E. (1983)."Strength of jointed rock masses"; 23rd; Ranking Lecture; Geotechnique 33(3), 187-223.

[7] Asgari Marnani, J. and Panji, M. (2008) . "Boundary Elements Methods, Theory and Application" ; Newshanegar Pub. ; Iran [in Persian].

[8] Asgari Marnani, J. and Panji, M. (2008). "Examination of Lateral Factor ' K' Effect on Tunnel Zone of Influence Using Boundary Elements Methods", Nedaye Daneshgah, Islamic Azad University Central Tehran Branch, Research Deputy, 24, 51-61[in Persian].

[9] Brebbia,C.A. and Dominguez, J. (1990). "Boundary Elements an introductory course"; Computational Mechanics publications.

[10] Panji, M. (2007). "Stress analysis of infinite and semi infinite continuum media using BEM", M.Sc. thesis in Civil Engineering, Islamic Azad University, Central Tehran Branch[in Persian].

[11] Sauter, S. A. and Schwab, C. (2010). "Boundary Element Methods", Springer series in Computational Mathematics.

[12] Brady, B. H. G. and Brown, E. T. (1993). "Rock Mechanics: for underground mining", 2nd Ed., London: Chapman & Hall.

[13] Rock Science Inc., University of Toronto, "EXAMINE 2D". (1998). User's Manual.

[14] Martin, C.D., Kaiser, P.K. and McCreath, D.R. (1999)."Hoek-Brown parameters for predicting the depth of brittle failure around tunnels", Canadian Geotechnical Journal, Vol. 36 , pp. 136-151.