Periodica Polytechnica
Civil Engineering

60(1), pp. 11–20, 2016
DOI: 10.3311/PPci.7923

RESEARCH ARTICLE

The Dangerous Condition of Ground during High Overburden Tunneling (A Case Study in Iran)

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Received 19-01-2015, revised 31-05-2015, accepted 22-06-2015

Abstract

Knowledge of the ground condition and its hazards can play an important role in the selection of support and suitable excavation method in underground structures. Water transport tunnel is one of the most important structures with regard to the goal of excavation, special conditions and limitations considered in the design and execution of them. Beheshtabad Water Conveyance Tunnel with 64930 meters length, 6 meters final diameter is the largest water Conveyance tunnel in Iran. Because of high over burden and weak rock in the most of tunnel path, the probable hazardous of the ground condition such as squeezing and rock burst must be studied. Squeezing stands for large time-dependent convergence during tunnel excavation. This phenomenon occurs in weak rocks and deep conditions. Besides, the height of overburden in some of the zone tunnel is about 1200 meters. The occurrence of this phenomenon is always together with the instantaneous release of strain energy stored in the rock materials, causing the harm to the personal equipment and the collapse of underground structures. The existence of high thickness overburden in some the zones of this project indicates the high potential of rock burst hazard. In this research, the length of the tunnel has been partitioned into sections using the interpreted geological, geophysical studies and borehole data. After evaluating rock burst and squeezing potential with alternative analytical and experimental methods for each section, the results of different methods were compared with each other. Results predict low to moderate squeezing potential and moderate to high rock burst potential for some panels of the tunnel.

Keywords

Central plateau of Iran · Dangerous condition of ground · rock burst · squeezing · Beheshtabad Water Conveyance Tunnel

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1 Introduction

Tunnels are one of the vital arteries that, because of excessive expenses spent for their introduction and also derangement of passing traffic as a result of perfect demolition or serious damages, need the observation of technical geotechnical considerations in design and performance. Zayandehrud River is the only permanent river in the Central Plateau of Iran. Water demand in this area is constantly growing due to population growth, key industries, withdrawal of ground water tables and reduction of its quality. So, Beheshtabad Tunnel, by transporting 1070 millions of cube meters of water per year to Iran central plateau, is considered in order to eliminate the shortages in the parts of drinking water, industry and agriculture. This plan, consisting of a dam with 184 meters height and water transport tunnel with the length of about 65 km and 6 meters diameter, is expected to be the longest water transport tunnel in Iran.

In this research, firstly, the tunnel was panelled by using the interpretation of geological, geophysical studies and boreholes. Then, the squeezing and rock burst potential were studied through empirical and analytical methods for each panel. Finally, the results were compared with each other.

1.1 Literature Review

The rock burst and squeezing are two main modes of underground instability caused by overstressing of the ground. Both modes are generally related to continuous ground. Squeezing can occur both in massive (weak and deformable) rocks and in highly jointed rock masses as a result of overstressing. It is characterized by yielding under the redistributed state of stress during and after excavation [1]. The squeezing can be very large; deformations as much as l7% of the tunnel diameter have been reported in India [2]. According to the unexpected geotechnical hazards during tunnelling, Singh et al., Goel et al., Jethwa et al., Hoek and Marinos have studied the squeezing phenomenon for deep tunnels in weak rocks and derived some criteria to recognize it [2–6].

In most criteria, the overburden load plays an important role in developing the squeezing conditions. Furthermore, when an excavation for a deep underground tunnel or chamber is undertaken in a strong and brittle rock, the change in stress results in dynamic damage to the adjacent rock. This is referred to as rockburst or break ways. Such rock bursts are a major hazard for the safety of engineers and engineering equipment, as well as affecting the shape/size of the structure [7]. Hoek and Brown, Myrvang and Grimstad, Hatcher, Haramy, Qiao and Tian, Wang and Park and Amberg have been working to identify rock burst in deep tunnels with brittle rocks [8–14].

2 Beheshtabad Water Conveyance Tunnel

Beheshtabad Water Conveyance Tunnel, about 65 kilometre length and 6 meter width, is one of the biggest water supplying projects for transporting water to the central plateau of Iran. This tunnel is located near Ardal City with east north-west south direction. From the entrance to 17 km of the tunnel, it is located in Zagros Zone and its output is in Sanandaj-Sirjan Zone. This tunnel is expected to transfer water to resolve water deficiencies and shortcomings for industrial and agricultural use in the central plateau of Iran, 1070 cubic million meters annually [15].

Most important problems in the path of this tunnel refer to its cross within numerous fractures, resulting in many problems and troubles during drilling and in the stages of maintenance coverage of tunnel.

With regard to 19 boreholes in the tunnel path, tunnel has been panelled to 16 sections. Engineering geological properties for each panel are summarized in Table 1. The rock engineering classification is shown in Table 2 [16].

Referring to Table 1, it can be seen that the classification grading by Q system is lower than that by the RMR for the same type rock. That is because Q system takes the high stress field into consideration, and to some extent, it causes the rock mass instability.

Regarding researches in the studied area, stability analysis and leakage quantity investigation have been conducted. Rahimdel and et al. proposed the primary support for tunnel section based on geology section and rock masses of the tunnel using RMR, Q and VNIMI methods. The results based on VNIMI method are given in Table 3 [17].

Rafiee and et al. [15] used the Fuzzy Analytical Hierarchy Process (FAHP) to support the estimation of tunnel. In this study, regarding the numerical analysis (finite difference program FLAC2D), six support systems were considered as the decision alternative are shown in Table 4 and support cost, factor of safety, applicability, time, displacement and mechanization were considered as the criteria. Calculations showed that the alternative "E" should be selected as the optimum support system to satisfy the goals and objectives of Behashtabad Tunnel.

3 Squeezing

The magnitude of tunnel convergence, the rate of deformation and the extent of the yielding zone around the tunnel depend on the geological and geotechnical conditions, the in-situ state of stress relative to rock mass strength, the groundwater flow and pore pressure, and the rock mass properties [18]. The increase in movement velocity and displacement magnitude often vary in the tunnel face depending on geological conditions, the principal stress orientations and the tunnel shape [19]. Squeezing is, therefore, synonymous with yielding and time-dependence; its cost depends on the excavation and support techniques adopted. If the support installation is delayed, the rock mass moves into the tunnel and stress redistribution take place around it. On the contrary, if deformation is restrained, squeezing will lead to long-term load build-up of rock support.

For the evaluation of the potential of squeezing, empirical and semi-empirical methods have been introduced via deferent researchers. These methods are explained below.

3.1 Prediction of Squeezing

3.1.1 Empirical Approaches

The empirical approaches are essentially based on classification schemes. Two of these approaches are mentioned below in order to illustrate the uncertainty still surrounding the subject, notwithstanding its importance in the tunnelling practice.

3.1.1.1 Singh et al. Approach This method, which is based on the results of 39 case histories, by collecting data on rock mass quality Q, overburden and height, proposes that squeezing potential is predictable by using Eq. (5) and Table 5 [2].

$$H = 350Q^{1/3} \tag{1}$$

Where H is the overburden and Q is the rock mass quality classification.

3.1.1.2 Goel et al. approach A simple empirical approach developed by Goel et al. is based on the rock mass number N, which is defined as stress-free Q as follows [3].

$$N = (Q)_{SRF=1} \tag{2}$$

Where N is the rock mass number, $(Q)_{SRF=1}$ is rock mass quality classification with SRF equals to 1 and SRF is stress reduction factor.

This is used to avoid the problems and uncertainties in obtaining the correct rating of parameter SRF in Barton et al. Q. Considering the tunnel depth H, the tunnel span or diameter B, and the rock mass number N from 99 tunnel sections, Goel et al. plotted the available data on a log-log diagram (Fig. 1), between N and $H \times B^{0.1}$ [3].

3.1.2 Semi-Empirical Approaches

The common starting point of all these methods for quantifying the squeezing potential of rock is the use of the “competency factor”, which is defined as the ratio of uniaxial compressive strength of rock/rock mass to overburden stress. Two of such methods are briefly discussed below.

Tab. 1. Rock engineering geological characteristics for each tunnel section [16]

Section	Kilometer (m)	Rock mass	Overburden (m)	Density (gr/cm^3)	UCS (MPa)	RQD
I	5941 - 7800	Limestone with dolomite	600	2.530	65 - 75	95 - 100
II	7800 - 8116	Marl stone	781.58	2.968	20 - 40	95 - 100
III	8116 - 10790	Lime stone and Marl stone	1205.5	2.509	65 - 75	95 - 100
IV	10790 - 12129	Marl stone and con-glomerate	340	2.488	70 - 90	95 - 100
V	12129 - 15492	Mud stone and con-glomerate	294	2.450	30 - 45	95 - 100
VI	15492 - 17574	Weathered and altered andesitic	285	2.491	20 - 30	50 - 60
VII	17574 - 18013	Crushed limestone and Marly limestone	327	2.651	20 - 40	40 - 50
VIII	18013 - 20862	Marly and shale limestone	349	2.464	20 - 30	50 - 85
IX	20862 - 21730	Marl and Shale	477	2.733	25 - 35	85 - 90
X	21730 - 24174	Marl and Shale	621	2.646	20 - 40	85 - 90
XI	24174 - 29030	Alteration of massive limestone	654.45	2.646	40 - 50	75 - 85
XII	29030 - 31604	Shaly limestone	381	2.651	25 - 60	25 - 60
XIII	31604 - 34912	Melonitic limy sand stone with quarts lenses	335.6	2.667	10 - 30	25 - 45
XIV	34912 - 37490	Melonitic limy sand stone with quarts lenses	481	2.690	25 - 50	25 - 50
XV	37490 - 37892	Limestone and dolomite	571	2.690	50 - 80	90 - 100

Tab. 2. **Table 2.** Rock engineering classification of the studied tunnel [16]

Tunnel Section	RMR		Q	
	Value	Rating	Value	Rating
I	54 - 55	Fair	1.65 - 2.67	Poor
II	60 - 64	Good	1.35 - 4	Poor
III	53 - 60	Fair	1.1 - 2	Poor
III	57 - 60	Fair	1.35 - 3	Poor
IV	50 - 71	Fair	2.4 - 13.3	Poor - Fair
V	56 - 61	Fair	2.3 - 9	Poor - Fair
VI	58 - 69	Good	3.92 - 9	Fair
VII	55 - 60	Fair	3.4 - 9	Poor - Fair
VIII	57 - 59	Fair	4.3 - 9	Fair
IX	19 - 21	Poor	0.006 - 0.015	Exceptionally Poor - Extremely poor
X	23 - 28	Poor	0.006 - 0.02	Exceptionally Poor - Extremely poor
XI	18 - 20	Poor	0.37 - 6	Fair
XII	50 - 64	Fair	2.1 - 6	Poor - Fair
XIII	50 - 57	Fair	0.95 - 2	Poor
XIV	49 - 59	Fair	1.1 - 3	Poor
XV	30 - 35	Poor	0.2 - 0.4	Poor

Tab. 3. Primary support estimation for tunnel rock masses

Rock mass	Primary support
Limestone with dolomite, marl stone, mud stone and conglomerate	Using rock bolt or shotcrete lining by 5 cm in Thickness.
Crushed limestone and marly limestone, Marly and shale limestone and Shaly limestone	Application of rock bolt 2.5 m in length with 1 × 1 distance together and shotcrete lining by 5 cm or more in Thickness with mesh and rock bolt

Tab. 4. Explanation of Model Notations [15]

Support system (Alternative)	Explanation
A	Supporting by shotcrete lining by 25 cm in thickness together with IPE18
B	Supporting by shotcrete lining by 30 cm in thickness together with IPE16
C	Supporting by shotcrete lining by 20 cm in thickness together with wire mesh
D	This system is the combination of shotcrete with steel fibre by 20 cm in thickness
E	Application of rock bolt 3 m in length with 1 × 1 distance together with shotcrete lining by 10 cm in thickness
F	Application of rock bolt 3 m in length with 2 × 2 distance together with shotcrete lining by 20 cm in thickness

Tab. 5. Classification of squeezing behaviour according to Singh et al.

H	Type of behaviour
$> 350Q^{1/3}$	Squeezing conditions
$< 350Q^{1/3}$	Non squeezing conditions

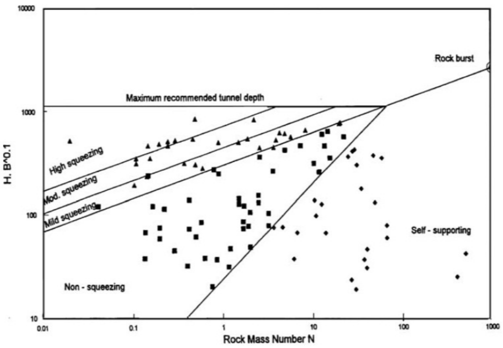

Fig. 1. Goel et al.'s approach for predicting squeezing conditions [3]

3.1.2.1 Jethwa et al. Approach As mentioned above, the degree of squeezing is defined by Jethwa et al. [4] on the basis of Eq. (3) and Table 6:

$$N_c = \sigma_{cm}/P_0 = \sigma_{cm}/\gamma H \qquad (3)$$

Where σ_{cm} is rock mass uniaxial compressive strength, P_0 is in situ stress, γ is rock mass unit weight and H is the tunnel depth below surface.

Tab. 6. Classification of squeezing behaviour according to Jethwa et al.

N_C	Type of behaviour
0.4 >	Highly squeezing
0.4 - 0.8	Moderately squeezing
0.8 - 2	Mildly squeezing
> 2	Non squeezing

3.1.2.2 Aydan et al. approach Aydan et al. [20], based on the experience of tunnels in Japan, proposed to relate the strength of the intact rock σ_{ci} to the overburden pressure γH by the same relation as (3), implying that the uniaxial compressive strength of the intact rock σ_{ci} and that of the rock mass σ_{cm} are the same. The fundamental concept of the method is based on the analogy between the stress-strain response of rock in laboratory testing and tangential stress-strain response around tunnels. As illustrated in Fig. 2, five distinct states of the specimen during loading are experienced, at low confining stress σ_3 (i.e., $\sigma_3 \leq 0.1\sigma_{ci}$). The following relations, as defined, give the normalized strain levels η_P, η_s and η_f [20].

$$\begin{aligned} \eta_P &= \varepsilon_P/\varepsilon_e = 2\sigma_{ci} - 0.17, \\ \eta_s &= \varepsilon_s/\varepsilon_e = 3\sigma_{ci} - 0.25, \\ \eta_f &= \varepsilon_f/\varepsilon_e = 5\sigma_{ci} - 0.32 \end{aligned} \qquad (4)$$

Where ε_P, ε_s and ε_f are the strain values shown in Fig. 2, as ε_e is the elastic strain limit.

Based on a closed form analytical solution, which has been developed for computing the strain level ε_Θ^a around a circular tunnel in a hydrostatic stress field, the five different degrees of squeezing are defined as shown in Table 7. In this Table, ε_Θ^a is the tangential strain around a circular tunnel in a hydrostatic stress field [20], whereas ε_Θ^e is the elastic strain limit for the rock mass.

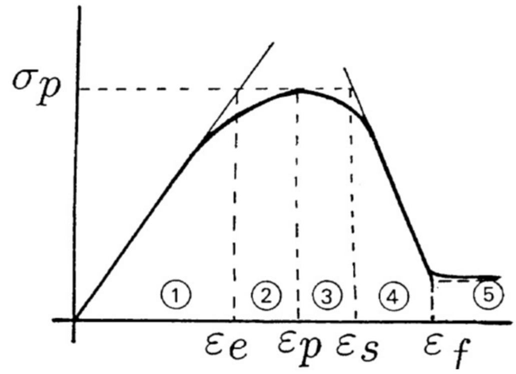

Fig. 2. Idealized stress-strain curve and the associated states for squeezing rocks

Tab. 7. Classification of squeezing behaviour according to Aydan et al.

Theoretical expression	Squeezing degree
ε_Θ^a / ε_Θ^e ≤1	Non-squeezing
1≤ ε_Θ^a / ε_Θ^e ≤ η_p	Light-squeezing
η_p ≤ ε_Θ^a / ε_Θ^e ≤ η_s	Fair-squeezing
η_s ≤ ε_Θ^a / ε_Θ^e ≤ η_f	Heavy-squeezing
ε_Θ^a / ε_Θ^e ≥ η_f	Very heavy squeezing

3.1.3 Analytical-Theoretical Approaches

3.1.3.1 Barla and International Society of Rock Mechanics (ISRM) Approaches The squeezing potential in these methods can be expected in accordance to Table 8 by considering the values of tangential stress (σ_Θ), uniaxial compressive strength (σ_{cm}) and the maximum stress (σ_1) [18].

Tab. 8. Classification of squeezing behaviour according to Barla and ISRM approaches [18]

Evaluation Method		Squeezing degree
ISRM (σ_θ / σ_{cm})	Barla (σ_{cm} / σ_1)	
< 1	> 1	Non-squeezing
1 - 2	1 - 0.4	Light-squeezing
2 - 4	0.4 - 0.2	Fair-squeezing
> 4	0.2 >	Heavy-squeezing

3.2 Evaluation of Squeezing Potential in Beheshtabab Water Conveyance Tunnel

The results of assessing squeezing potential for the zone of the tunnel, in which there was the occurrence of this phenomenon using different criteria, have been shown in Fig. 3. To study the result of different criteria, the percentage of each category of the studied squeeze zones was calculated as shown in Table 9. In average, 69, 23, 5 and 3 percent of total panels were in none, light, moderate and heavy squeezing conditions, respectively. So, most sections of the tunnel were in none squeezing potential.

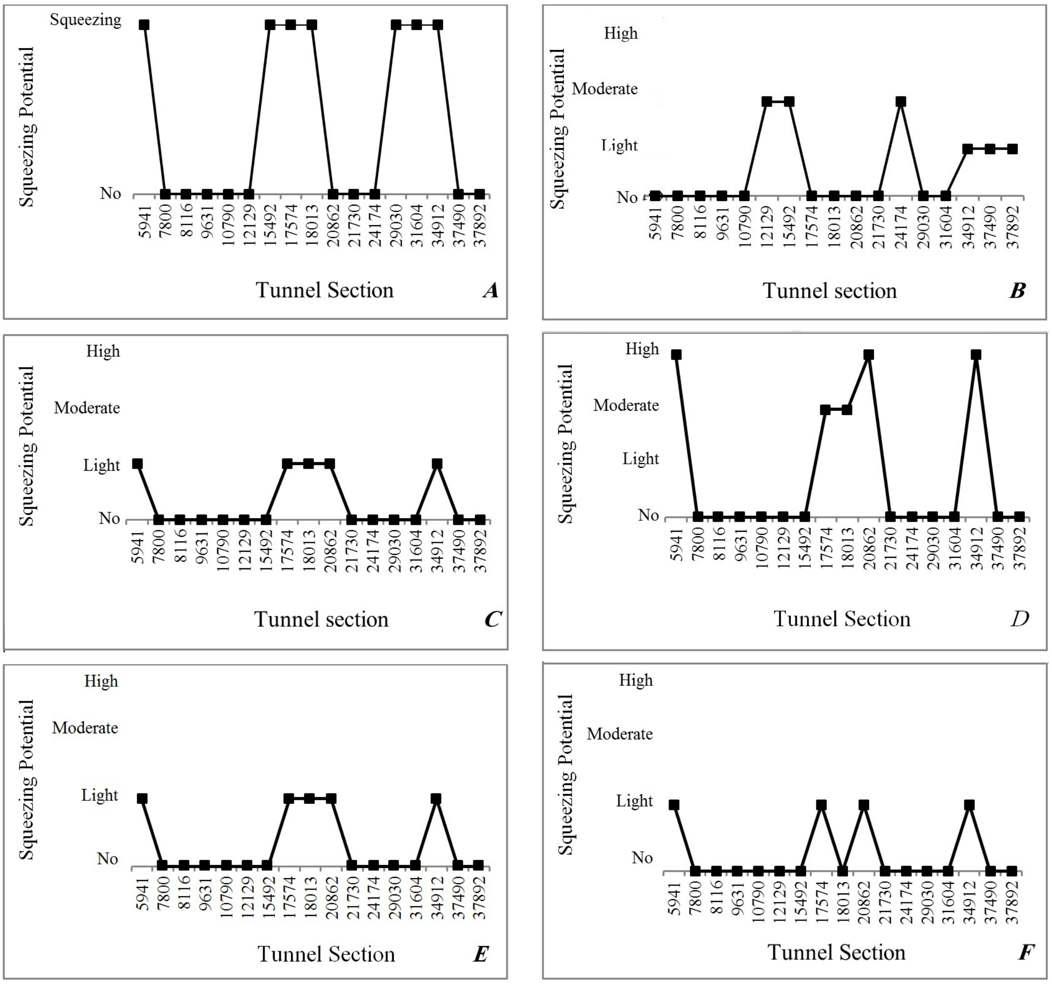

Fig. 3. The results of the squeezing potential using Singh (A), Goel (B), Jethwa (C), Aydan (D), Barla (E) and ISRM (F) criteria

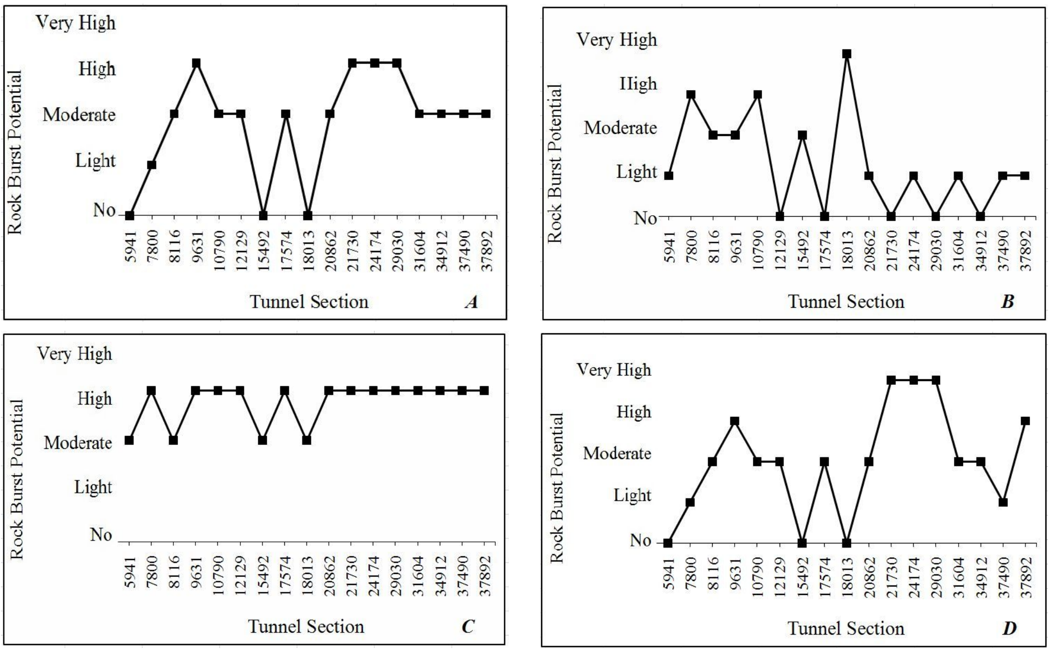

Fig. 4. The results of the rock burst potential using the method of stresses (*A*), linear elastic criterion (*B*), brittleness coefficient (*C*) and tensile stress (*D*) criteria.

Tab. 9. The results of the squeezing potential in Beheshtabad Water Conveyance Tunnel

Percentage of tunnel sections in each squeezing condition				Evaluation criteria
Non	Light	Moderate	High	
59	41	0	0	Singh
65	17	17	0	Goel
72	28	0	0	Jethwa
72	0	11	17	Aydan
72	28	0	0	Barla
75	25	0	0	ISRM

4 Rock Burst

A rock burst is one of the most complicated dynamic geological phenomena, with intricate mechanisms and numerous affecting factors, which accounts for the difficulty of predicting its characteristics. In the past few years, many methods of forecasting rock bursts have been proposed, including the assessment of rock mechanics, stress detection and modern mathematical theories.

The prevention of rock bursts is one of the key problems in the construction of deep tunnels in which rock burst prediction is a basic problem. In the construction of underground engineering, it is of great importance for the safety and the optimization of support measures to make correct and timely predictions of the possibility, as well as the scope and intensity of rock bursts in the rock mass surrounding the excavated ground.

4.1 Rock Burst Prediction

Regarding the available and valid references, comprehensive researches have been carried out in the classification and evaluation of rock burst phenomenon. In most of them, linear elastic criterion, method of Tensile Stress, method of Brittleness Coefficient and Method of Stresses have been used for rock burst prediction [7, 21–35]:

4.1.1 Linear Elastic Criterion

Linear elastic energy stored in rock before reaching the peak strength can be defined by the Eq.(5) [21].

$$LE = \frac{\sigma_c^2}{2E} \tag{5}$$

Where LE is the linear elastic energy (MPA), E is unloading tangent elastic modulus of rock, and σ_c is uniaxial compressive strength. Rock burst potential is predictable by using Table 10.

4.1.2 Method of Tensile Stress

Rock burst predictions using this method can be defined by Eq.(6). Rock burst potential is predictable by using Table 11 [13].

$$T_s = \frac{\sigma_\theta}{\sigma_c} \tag{6}$$

Where σ_{Theta} is the tensile stress, and σ_c is the uniaxial compressive strength.

4.1.3 Method of Brittleness Coefficient

This method evaluates the tendency of rock burst through the brittleness coefficient of Rocks (β). This coefficient is defined as the ratio of σ_c over σ_t (σ_c and σ_t are the uniaxial compressive strength and the tensile strength of the rock, respectively), i.e., $\beta = \sigma_c/\sigma_t$. In general, the grater β, the higher the rock burst tendency (see Table 12) [22].

4.1.4 Method of Stresses

Method of stresses combines the lithological character of a rock mass (including tensile and compressive strength) to judge the possibility that rock burst can take place. This method introduces two factors of α and β to serve as criteria. α and β are defined, respectively, as the ratio of the rocks uniaxial compressive strength (σ_c) over the major principle geo-stress (σ_1), i.e., $\alpha = \sigma_c/\sigma_1$ and as the ratio of the rocks uniaxial tensile strength, σ_t, over σ_1, i.e., $\beta = \sigma_t/\sigma_1$. Because the index of the uniaxial compressive can be determined easily, the value of α is generally used for a criterion having the following Table [22].

4.2 Evaluation of Rock Burst Potential in Beheshtabad Water Conveyance Tunnel

The results of the rock burst potential assessing for the zone of the tunnel in which the occurrence of this phenomenon was achieved using different criteria, as shown in Fig. 4. To study the Different criteria results, the percentage of each category of studied rock burst zones was calculated as shown in Table 14.

Regarding Table 14, Linear elastic criterion predicts no rock burst potential for more sections of the tunnel, while Tensile Stress and Stresses methods assume the major sections of tunnel to be in the fair rock burst potential. According to brittleness coefficient, all tunnel sections are unfortunately in heavy rock burst condition. In average, 16, 13, 31, 34 and 6 percent of total panels are in none, light, moderate, heavy and very heavy rock burst conditions, respectively. So, most sections of tunnel are in moderate to high rock burst condition. To have a better comparison, the obtained results have been shown in Fig. 5. Also, to better understand, the results were given in Table 15. Regarding Fig. 5 and Table 15, more of the sections are in high squeezing potential condition. So, in this tunnel, the squeezing potential is more important than the rock burst. These results are in agreement with high overburden and weak sedimentary rock masses in these sections.

Tab. 10. Classification of Rock burst behaviour according to linear elastic criterion

50 >	50 - 100	100 - 150	150 - 200	200 <	*LE* (MPa)
Very Low	Low	Moderate	High	Very High	Rock burst potential

Tab. 11. Classification of Rock burst behavior according to the Method of Tensile Stress

0.3>	0.3 - 0.5	0.5 - 0.7	0.7 - 0.9	0.9 <	T_S
Non	Low	Moderate	High	Very High	Rock burst potential

Tab. 12. Classification of Rock burst behaviour according to the method of brittleness coefficient

40<	40 - 26.7	26.7 - 14.5	14.5 >	β
Non	Low	Moderate	High	Rock burst potential

Tab. 13. Classification of Rock burst behaviour according to the Method of Stresses

10 <	10 - 5	5 - 2.5	2.5 >	α
Non	Low	Moderate	High	Rock burst potential

Tab. 14. The results of the rock burst potential

Percentage of tunnel sections in each of rock burst conditions					
Non	Light	Moderate	High	Very high	Evaluation criteria
18	6	53	23	0	Stresses
29	35	18	12	6	Linear elastic criterion
0	0	12	88	0	Brittleness coefficient
18	12	41	12	17	Tensile Stress

Tab. 15. The results of squeezing and rock burst potential in the tunnel sections

Percentage of tunnel sections in each of rock burst and Squeezing conditions (%)					
	Non	Light	Moderate	High	Very High
Squeezing	69	23	5	3	0
Rock burst	16	13	31	34	6

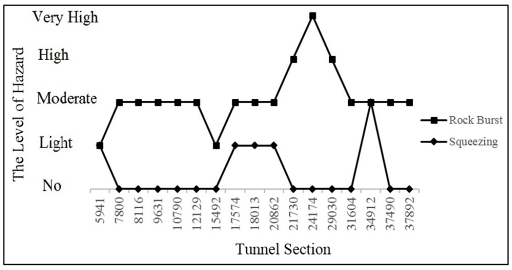

Fig. 5. Comparison of the squeezing and rock burst potential results

5 Conclusions

Squeezing and rock burst potential were addressed in this article using different empirical, semi-empirical and analytical approaches. The results showed that empirical and analytical methods were almost accommodated with each other. In squeezing potential research, according to Singh, Jethwa, Barla and ISRM approaches, a great numbers of tunnel sections fell into non-squeezing potential category. Aydan and Goel criteria, similar to the recently mentioned approaches, have predicted moderate to heavy squeezing potential for a small percentage of sections. Based on our researches, the results showed that 69, 23, 5 and 3 percent of total panels were in none, light, moderate and heavy squeezing conditions, respectively. Thus, the rock masses in this tunnel path were in none to light squeezing potential. In rock burst potential research, according to forbear Linear Elastic Criterion that predicted moderate rock burst potential for all sections, 16, 13, 31, 34 and 6 percent of total panels were in none, light, moderate, heavy and very heavy rock burst conditions noticeability by referring back other methods of Tensile Stress, Tensile Stress and Method of Stresses. So, the rock masses in this tunnel path were in moderate to high rock burst potential. According to the precise prediction of this phenomena, it is not possible to have a safe environment during the deep exploration and mining. So, some necessary measure of prevention are proposed:

1. The construction methods can be improved. The impact of blasting vibration should be minimized as far as possible to avoid bringing about various factors inducing rock burst.
2. Rock can be strengthened by grouting to change the mechanical properties of the wall rock. Grouting bolt nets and plastic bolts can also be applied to the underground chamber or wall rock.
3. In very poor squeezing conditions, using heavy support and monitoring the displacements of the roof and bottom of the tunnel and using flexible support in moderate to high squeezing conditions are essential.

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