Structural Engineering and Mechanics, *Vol. 60, No. 5 (2016) 809-828*
DOI: http://dx.doi.org/10.12989/sem.2016.60.5.809

Comparison of Hoek-Brown and Mohr-Coulomb failure criterion for deep open coal mine slope stability

Cemalettin O. Aksoy*[1], Guzin G. Uyar[2] and Yılmaz Ozcelik[2]

[1]*Department of Mining Engineering, DokuzEylul University, Faculty of Engineering, İzmir, Turkey*
[2]*Department of Mining Engineering, Hacettepe University, Faculty of Engineering, Ankara, Turkey*

(Received March 18, 2016, Revised August 9, 2016, Accepted September 21, 2016)

Abstract. In deep open pit mines, slope stability is very important. Particularly, increasing the depths increase the risks in mines having weak rock mass. Blasting operations in this type of open pits may have a negative impact on slope stability. Several or combination of methods can be used in order to enable better analysis in this type of deep open-pit mines. Numerical modeling is one of these options. Many complex problems can be integrated into numerical methods at the same time and analysis, solutions can be performed on a single model. Rock failure criterions and rock models are used in numerical modeling. Hoek-Brown and Mohr-Coulomb terms are the two most commonly used rock failure conditions. In this study, mine planning and discontinuity conditions of a lignite mine facing two big landslides previously, has been investigated. Moreover, the presence of some damage before starting the study was identified in surrounding structures. The primary research of this study is on slope study. In slope stability analysis, numerical modeling methods with Hoek-Brown and Mohr-Coulomb failure criterions were used separately. Preparing the input data to the numerical model, the outcomes of patented-blast vibration minimization method, developed by co-author was used. The analysis showed that, the model prepared by applying Hoek-Brown failure criterion, failed in the stage of 10. However, the model prepared by using Mohr-Coulomb failure criterion did not fail even in the stage 17. Examining the full research field, there has been ongoing production in this mine without any failure and damage to surface structures.

Keywords: numerical modeling; controlled blasting; yielding criteria; deep open pit; slope stability

1. Introduction

Empirical and observational methods, limit equilibrium method and stress-strain analysis method can be used in slope stability analysis. Empirical methods are based on the results of model or equations created using previous experiences. However, the rock mass can show different properties in different locations. Therefore, it is unable to use empirical formulas accurately everywhere. It is necessary to know the shear strength of discontinuities in the rock mass in the limit equilibrium method. Determining the shear strength of discontinuities is an expensive and long-term process. In limit equilibrium method, sliding surface should be predicted earlier. Although 2D plane sliding analysis can be performed easily in this method, in case of analysis of

*Corresponding author, Professor, E-mail: okay.aksoy@deu.edu.tr

http://www.techno-press.org/?journal=sem&subpage=8 ISSN: 1225-4568 (Print), 1598-6217 (Online)

3D rock mass discontinuities, analytical method of Londe, Vigier *et al.* (1969) and graphical method proposed by John (1968) can be used. The most important parameter used in the kinematic analysis is discontinuities. Safety coefficient can be used in this type of analysis (Hoek and Bray 1981). Stress-Strain analysis, however, takes into account the strength and deformation properties of the rock mass. The most important element in this method is to determine the most accurate material model and failure condition representing rock mass deformation properties accurately. This determination phase requires considerable experience. As will be apparent from the descriptions, the first two methods mentioned above are used to analyze slope stability. However, in mining operations, it is not enough simply to examine the slope stability. In Stress-Strain analysis method, which is the third method, it is possible to integrate a lot of parameters into the model. Thus, not only the slope stability analysis, it is also possible to see the risks that may occur in the structure of the surrounding rock and the surface. In this study, although static modeling studies are given, dynamic processes such as earthquakes, blasting vibration, can be examined by numerical modeling. Today's technology allows it. Today, numerical modeling is the easiest way for providing three-dimensional solutions to the problems. There are many methods in numerical modeling. The most commonly used are the finite elements, finite differences and boundary elements. Feng and Hudson (2008) clearly demonstrated the principles of numerical modeling (Fig. 1). As can be seen from Fig. 1, the numerical modeling is an easy method incorporating long term observations. It is possible to integrate many parameters (time, material model, failure criteria etc.) into the numerical models. In 3D numerical modeling studies, the possible effects before and after excavation and effects of excavation area can be analyzed. Moreover, the performance of precautions taken against these effects can be examined. Also, system sensitivity after integration of dynamic loads to the model can be analyzed (Chen, Zhu *et al.* 2016, Castiati, Chassiakos *et al.* 2015).

The success of the design in rock engineering is closely associated with making accurate characterization of the rock mass and the correct identification of rock mass behavior. It is possible to define the behavior of many materials in the software used for numerical modeling. The most commonly used material behavior are Hoek-Brown, Mohr-Coulomb, Soft Soil Creep, Hardening

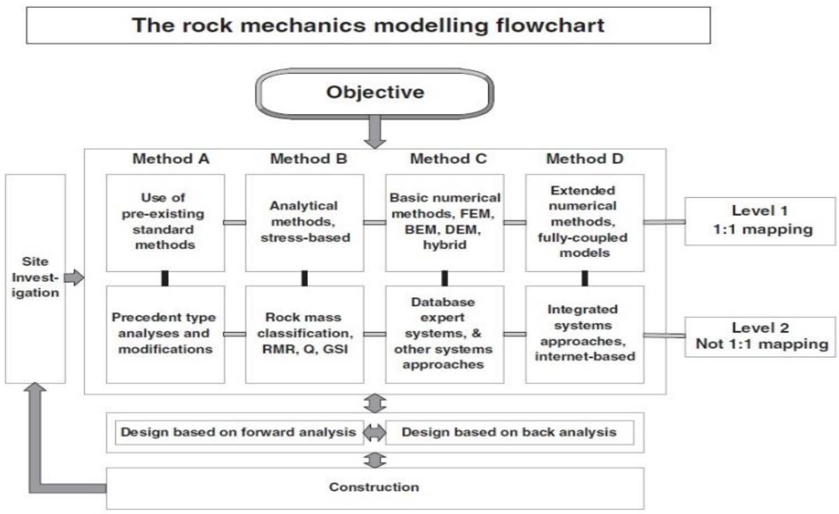


Fig. 1 Flow chart of modeling types in rock engineering design (Feng and Hudson 2008)

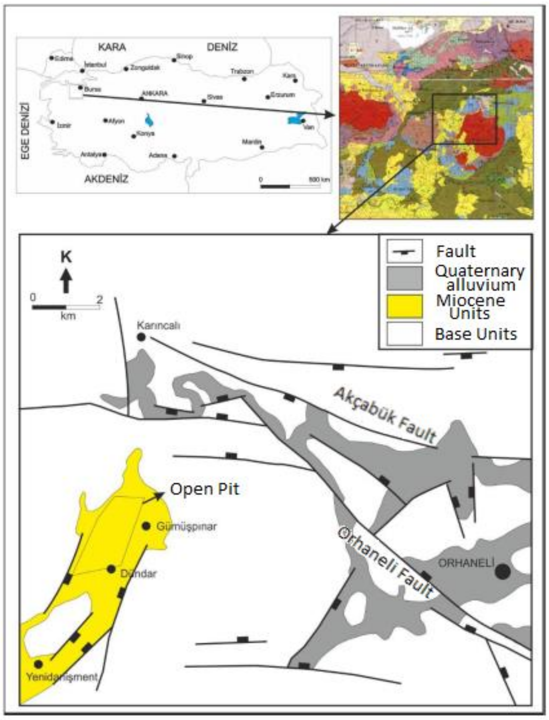


Fig. 2 Simplified geological map and location of the study area (Ö zçelik, Dirik *et al.* 2013, Karpuz, Koçyiğit *et al.* 2006)

Soil, Jointed Rock Mass (Aksoy, Oğul *et al.* 2012). Determination of true alternatives that will represent the rock mass is an important step in view of fidelity of the results. This choice requires a significance experience.

2. Geological and geotechnical properties of the study area

The study was conducted in Bursa/ Orhaneli Gümüspınar Open pit mine, Turkey. Geological and geotechnical information has been provided by Ozcelik, Dirik *et al*. (2013). The first detailed study of Gümüspınar open pit and its vicinity was carried out by Günalay (1974). This study indicated that, the age of the coal seam in this area is Lower-Middle Miocene. Memikoglu (1976) stated that there are NE-SW trending folds in the region. He also emphasized that the main faults are EW and NE-SW trending and they are covered with tuff. Gümüşpınar open-pit is in line with NNE-SSW direction and coal excavation continues on the eastern edge of the area. Basement rocks in the vicinity of the study area are Mesozoic ophiolitic units and Jurassic-Cretaceous aged marble typically observed in Izmir-Ankara-Erzincan zone. Miocene-aged coal cover these units. There are Miocene-aged rocks containing clay and coal in the lower level. In the upper level, these rocks are represented by tuff-sandstone-claystone. The youngest units in the region is the recent alluvial, mostly observed in the topographically lowest level (Fig. 2) (Karpuz, Koçyiğit *et al.* 2006). The strati graphical column of the region is given in Fig. 3.

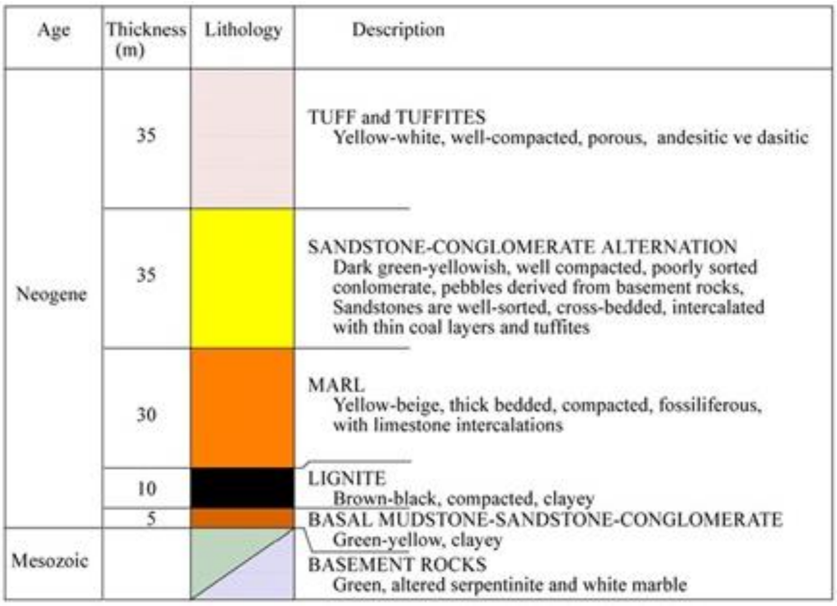


Fig. 3 Generalized stratigraphic column of Bursa Orhaneli Coal Area (Aldas 2002)

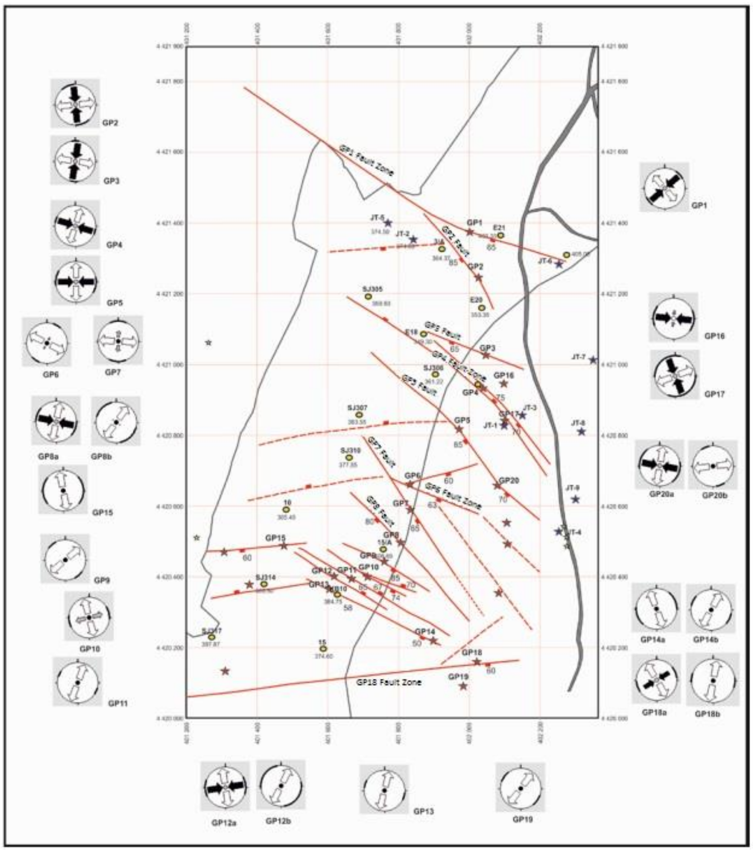


Fig. 4 Map showing the faults controlling Gümüspınar coal region (Özçelik, Dirik *et al.* 2013)

2.1 Formation mechanism of the guided waves discontinuity conditions of Gümüşpınar open pit mine

Gümüspınar surface coal mine deepened since there has been mining operation for many years. Mining has been done in a region having number of dicontinuities and weak rock mass. Özçelik, Dirik *et al*. (2013) determined the discontinuity structure of the latest state of the region (Fig. 4). The Neogene sedimentary units are overlying ophiolites in the western part of the region and crystalline limestones in the Eastern part. These units are cut by fault zones. The fault system controlling the basin was examined in detail and named from north to south as GP1fault zone, GP2 fault, GP3 fault zone, GP4 fault zone, GP5 fault, GP6 fault zone, GP7 fault, GP8 fault, GP9 fault, GP10 fault, GP15 fault, GP18. Field studies and kinematic analysis showed that two different tectonical regimes have affected the region.The first one is the opening regime developed depending on squeezing in EN-WSW direction.This regime led to formation of strike slip faults in the study area. Most affected and young tectonical regime, however, is expansion. Depending on NE-SE expansion, normal faults developed (Özçelik, Dirik *et al.* 2013).

3. Controlled blasting based numerical modeling studies carried out in Gümüspınar surface mine

Gümüşpınar open pit coal mine in Bursa-Orhaneli includes clay bands alongside the weak rock mass. In some parts, the depth of the mine exceeds 170 m. There is a residential area near the open pit (Fig. 5). Previously, the two major landslides occurred and some buildings in nearby settlements were damaged. In slope stability studies in the region, special blasting design techniques were aimed in order to minimize the effects of blasting to both slope stability and nearby settlements. After minimizing the blasting effects, slope stability works were carried out by numerical modeling. Another aim to minimize the blast effect is to take disturbance factor as D=1 in numerical modeling of rock mass parameters.

Fig. 5 View of Gümüşpınar surface coal mine

3.1 Controlled blast design studies

Blast induced vibrations are so dangerous to damage structure and slope. There are so many researches about the vibration damages and behavior (Men, Guo *et al.* 2015a, Men, Zhang *et al.* 2015b, Loh, Huang *et al.* 2015). In order to minimize the effects of blasting on slope stability, a new methodology developed by co-author (Aldaş and Ecevitoğlu 2007, Aldas and Ecevitoğlu 2008) was used. This methodology is quite different from conventional methods which do not take into account the mechanics of seismic waves. Contrary to conventional methods, the proposed methodology does not consider any blast-parameters such as types and amounts of explosive, blast-geometry, blast-hole design, hole-depth/diameter, etc., except time-delays. The methodology aims to employ the most suitable time-delays among blast-hole groupings to minimize destructive interference of the surface waves at the location of blast-induced vibrations. The crucial point of the proposed methodology is the use of a pilot blast signal which takes account of the seismic properties of all complex geology between the blast and the target locations. Therefore, it does not require any geological model or assumption. It is based on two seismic records related to: (1) Pilot-blast, (2) Group-blast. The group-blast is made up of pilot-blasts, i.e. a gathering of pilot-blasts representing the group-blast. The seismic records obtained from pilot and group blasts share the same blast-design properties, such as explosive types and amount, hole-diameter and depth, etc. It is assumed that seismic waves initiated from pilot and group blasts should travel along the same geological structures, such as lithology, stratigraphy and tectonics. Since the pilot-blast contains all the information related to the above stated factors, there is no need to take in account all the details of the complex geology. The effects of blasting on slope stability is not a deeply investigated topic in the literature. Generally, blasting effects to the nearby settlements are tried to be minimized. In this study, our aim was to develop blasting pattern design to minimize the vibration effects to both slopes and nearby Gümüşpınar Village. The reason of preferring this new methodology in this study is that there are many advantages compared to classical method: (1) Data evaluation is not solely based on peak particle velocities, as it is for conventional methods. seismic waveforms, their frequency content, and their time-duration are also taken into account, (2) The proposed methodology does not impose any restriction on the amount and type of explosives to be used, or on blast-design, (3) The new methodology requires fewer seismic stations than conventional methods to analyze blast-induced vibrations. Actually, one seismic station located at the target location is sufficient for data analysis, if the approximate surface wave velocity is known. Conversely, in the case of conventional methods based on empirical models, at least 30 seismic records are needed to make a reliable data analysis.

In the mine, prior to our study, a group blast having more than 20 drill holes was divided into 4 groups and every group having 5 drill holes were blasted individually in a couple of minutes. This is very dangerous in terms of the person connecting the initiation systems. Because after each blast, he was due to connect the rest of the group full of explosives. The other negative effect of this blasting method is for slopes. Because, more than 20 blast holes have not been blasted in the same group with certain milliseconds delay, instead, many small groups were blasted in every 5-6 minutes. Therefore, it may cause deformations in the slopes. This case was proven by the inclinometer results (Gökay, Özkan *et al.* 2013). To overcome this negative blasting results, blast groups were designed by using a new methodology. Groups were limited to 15 drill holes but blasted at the same blasting process with suitable delays determined by new methodology. Fig. 6 illustrates the blast vibration amplitudes recorded at the slope where landslides occurred previously. This blasting was carried out without our suggestions. This record was induced by

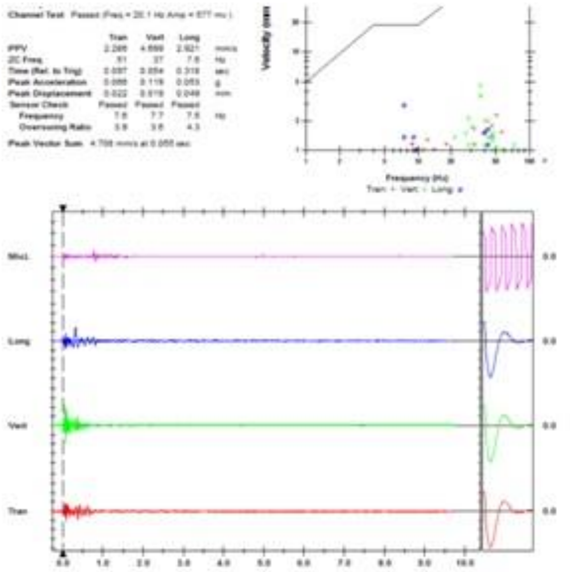


Fig. 6 Blasting record prior to this study

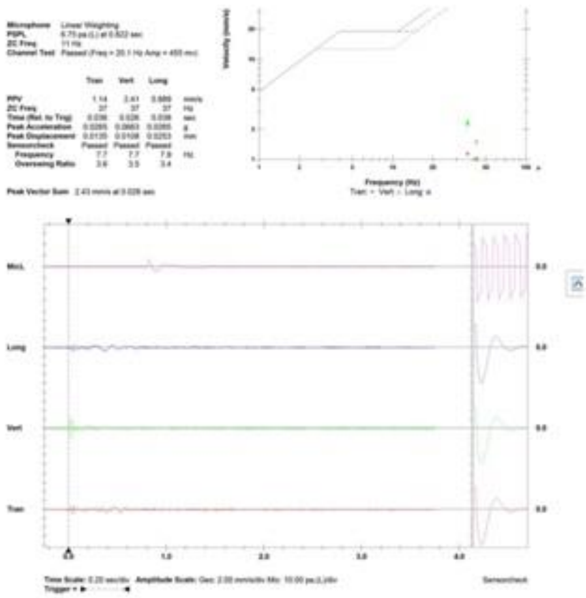


Fig. 7 Blasting record after using this suggestions

blasting of "5 drill holes" as small group of big blasting group having 20 blast-holes. Although it was a very small group (65 kg Anfo/delay was used), vibration amplitudes were recorded as around 4.7 mm/s, at a distance 450 meters from the blast location.

Fig. 7 shows the blast vibration amplitudes recorded at the same slope. The amount of explosive per delay was the same. In this blast, in line with our suggestion, 14 blast holes were blasted at the same blasting operation but using certain delays determined by our new method. As the figure illustrates this blast induced approximately half vibration amplitudes comparing to previous blast, although it has more drill holes from the previous blast.

With controlled blasting techniques developed by using our new methodology, it was possible to minimize the effects of blasting on the slopes. While doing this, nearby settlement, Gümüşpınar Village, was also taken into consideration and blasting pattern design was suggested to minimize the blasting effects both for the slopes and for village. Optimal solutions for slopes and villages were developed to minimize the blasting effects Aldaş and Kaypak (2013). As a result of all those studies, it can be safely said that disturbance factor D can be taken as 1 in the calculation of rock mass parameters in numerical modeling studies. Because, newly suggested blasting pattern was able to minimize the blast-induced vibration effects for the slopes.

3.2 Numerical modeling studies

Today's technology enables that many complex problems can be analyzed on the same model by numerical modeling. Crack are so common in rock mass and the crack analyses via numerical analysis are very complex (Yaylac 2016). On the other hand, rock mass with clay is behave differently for every different field. Analyses of these type of rock mass are also veru complex (Yıldız and Uysal 2015, Chen, Xu *et al.* 2014). In this research, in the Gümüşpınar open pit, slope stability studies were examined by numerical modeling method. One of the most important issues in numerical modeling is the correct determination of rock mass deformation behavior and rock mass failure criteria. This requires considerable experience, as stated previously. Due to weak rock mass, clay band and important discontinuities in Gümüşpınar open pit mine, it was intended to use both Hoek-Brown and Mohr-Coulomb failure criteria. Rock mass classification systems were used while preparing data to numerical models. Discontinuities in the region were evaluated in the rock mass classification systems and used as parameters in preparing rock mass parameters to the models.

3.2.1 Mohr-Coulomb and Hoek-Brown failure criteria

The Mohr-Coulomb failure criterion is a set of linear equations in principal stress space describing the isotropic material conditions failing with any effect from the intermediate principal stress σ_2, being neglected. Mohr-Coulomb failure criterion can be illustrated as a function of major σ_1 and minor σ_3 principal stresses or normal stress σ and shear stress τ on the failure plane (Jaeger and Cook 1979). Mohr-Coulomb suggested the relationship as;

$$\tau = c + \sigma_i tan\varphi \tag{1}$$

where c is the inherent shear strength, also known as cohesion and φ is the angle of internal friction, with the coefficient of internal friction μ=tan φ. The criterion contains two material constants, c and φ, as opposed to one material constant for the Tresca criterion (Nadai 1950).

Mohr-Coulomb failure criteria consists of 6 failure functions in terms of the principal stress.

$$f_{1a} = \frac{1}{2}(\sigma'_2 - \sigma'_3) + \frac{1}{2}(\sigma'_2 + \sigma'_3)Sin\varphi - c\cos\varphi \le 0 \tag{2}$$

$$f_{1b} = \frac{1}{2}\left(\sigma'_3 - \sigma'_2\right) + \frac{1}{2}\left(\sigma_3 + \sigma'_2\right)Sin\varphi - c\cos\varphi \le 0 \tag{3}$$

$$f_{2a} = \frac{1}{2}\left(\sigma'_3 - \sigma'_1\right) + \frac{1}{2}\left(\sigma'_3 + \sigma'_1\right)Sin\varphi - c\cos\varphi \le 0 \tag{4}$$

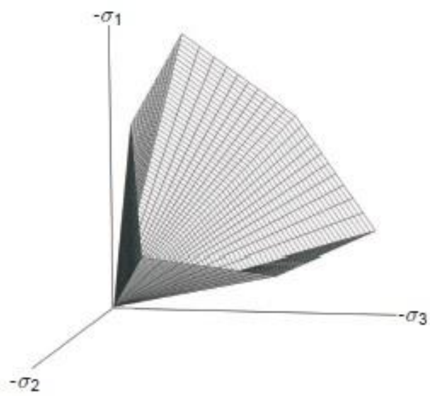


Fig. 8 Mohr-Coulomb failure surface in principal stress space (*c*=0)

$$f_{2b} = \frac{1}{2}(\sigma'_1 - \sigma'_3) + \frac{1}{2}(\sigma'_1 + \sigma'_3)Sin\varphi - c\cos\varphi \le 0 \tag{5}$$

$$f_{3a} = \frac{1}{2}(\sigma'_1 - \sigma'_2) + \frac{1}{2}(\sigma'_2 + \sigma'_1)Sin\varphi - c\cos\varphi \le 0 \tag{6}$$

$$f_{3b} = \frac{1}{2}(\sigma'_2 - \sigma'_1) + \frac{1}{2}(\sigma'_2 + \sigma'_1)Sin\varphi - c\cos\varphi \le 0 \tag{7}$$

Two prominent parameters in failure function are cohesion (*c*) and internal friction (ϕ). When f_i= 0, it takes the form of a hexagonal cone in the principal stress space (Fig. 8).

In addition to the failure condition, 6 plastic functions are defined in the Mohr-Coulomb model. Expansion angle (ψ) is the third parameter added to the failure criteria in addition to plastic functions.

$$g_{1a} = \frac{1}{2}(\sigma'_2 - \sigma'_3) + \frac{1}{2}(\sigma'_2 + \sigma'_1)Sin\varphi \tag{8}$$

$$g_{1b} = \frac{1}{2}(\sigma'_3 - \sigma'_2) + \frac{1}{2}(\sigma'_2 + \sigma'_3)Sin\varphi \tag{9}$$

$$g_{2a} = \frac{1}{2}(\sigma'_3 - \sigma'_1) + \frac{1}{2}(\sigma'_3 + \sigma'_1)Sin\varphi \tag{10}$$

$$g_{2b} = \frac{1}{2}(\sigma'_1 - \sigma'_3) + \frac{1}{2}(\sigma'_1 + \sigma'_3)Sin\varphi \tag{11}$$

$$g_{3a} = \frac{1}{2}(\sigma'_1 - \sigma'_2) + \frac{1}{2}(\sigma'_2 + \sigma'_1)Sin\varphi$$

$$g_{3b} = \frac{1}{2}(\sigma'_2 - \sigma'_1) + \frac{1}{2}(\sigma'_2 + \sigma'_1)Sin\varphi \tag{12}$$

3.2.2 Hoek-Brown failure criteria

The Hoek-Brown failure criterion is an empirically derived relationship used to describe a non-linear increase in peak strength of isotropic rock with increasing confining stress. Following the parabolic, non-linear form of Hoek-Brown discriminate it from the linear Mohr-Coulomb failure criterion. The criterion includes companion procedures established to provide a practical means to estimate rock mass strength from laboratory test values and field observations. Hoek-Brown assumes independence of the intermediate principal stress (Eberhart 2012).

The main purpose of Hoek-Brown criterion is to estimate rock mass strength by scaling the relationship derived according to the present geological conditions. The criterion was conceived based on Hoek's (1968) experiences with brittle rock failure and his use of a parabolic Mohr envelope derived from Griffith's crack theory (Griffith 1920, 1924) to define the relationship between shear and normal stress at fracture initiation. Accordingly, the Hoek-Brown criterion is empirical with no fundamental relationship between the constants included in the criterion and any physical characteristics of the rock (Hoek 1983).

The original non-linear Hoek-Brown failure criterion for intact rock (Hoek and Brown 1980) was introduced as

$$\sigma_1 = \sigma_3 + \sqrt{m\sigma_i\sigma_3 + s\sigma_i^2} \tag{14}$$

where σ_1 and σ_3 are the major and minor principal stresses at failure, σ_i is the uniaxial compressive strength of the intact rock, and *m* and *s* are dimensionless empirical constants.

As an empirical criterion, the Hoek-Brown criterion has been restructured several times in response to experience gained with its usage. Moreover, certain practical limitations were addressed in these updates (Hoek and Brown 1988, Hoek *et al*. 1992, 1995, 2002). These updates primarily involve adjustments to improve the estimate of rock mass strength. One of the most important one is the reporting of the 'generalized' form of the criterion (Hoek *et al.* 1995)

$$\sigma_1' = \sigma_3' + \sigma_{ci}\left(m\frac{\sigma_3'}{\sigma_{ci}} + s\right)^{a} \tag{15}$$

Where m_b is a reduced value of the material constant m_i and is given by

$$m_b = m_i exp\left(\frac{GSI-100}{28-14D}\right) \tag{16}$$

s and a are constants for the rock mass given by the following relationships

$$s = exp\left(\frac{GSI-100}{9-3D}\right) \tag{17}$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-\frac{GSI}{15}} - e^{-\frac{20}{3}}\right) \tag{18}$$

D is a factor which depends upon the degree of disturbance (Hoek *et al*. 2002).

The Hoek-Brown failure criterion is an empirically derived relationship used to describe a non-linear increase in peak strength of isotropic rock with increasing confining stress. Hoek-Brown follow a non-linear, parabolic form that distinguishes it from the linear Mohr-Coulomb failure criterion. The criterion includes companion procedures developed to provide a practical means to estimate rock mass strength from laboratory test values and field observations. Hoek-Brown assumes independence of the intermediate principal stress (Eberhart 2012).

The Hoek-Brown criterion was developed as a means to estimate rock mass strength by scaling the relationship derived according to the geological conditions present. The criterion was

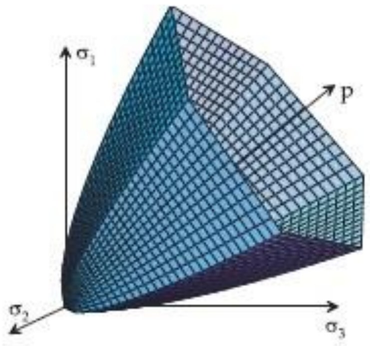


Fig. 9 Failure surfaces of the Hoek-Brown failure condition in the principal stress space

conceived based on Hoek's (1968) experiences with brittle rock failure and his use of a parabolic Mohr envelope derived from Griffith's crack theory (Griffith 1920, 1924) to define the relationship between shear and normal stress at fracture initiation. Accordingly, the Hoek-Brown criterion is empirical with no fundamental relationship between the constants included in the criterion and any physical characteristics of the rock (Hoek 1983).

Hoek-Brown failure criteria are defined by two failure functions in 3D stress condition.

$$f_{HB,13} = \sigma'_1 - \sigma'_3 + \bar{f}(\sigma'_3), \Rightarrow \bar{f}(\sigma'_3) = \sigma_{ci}(m_b \frac{-\sigma_3}{\sigma_{ci}} + s)^a \tag{19}$$

$$f_{HB,12} = \sigma'_1 - \sigma'_2 + \bar{f}(\sigma'_2), \Rightarrow \bar{f}(\sigma'_2) = \sigma_{ci}(m_b \frac{-\sigma_2}{\sigma_{ci}} + s)^a \tag{20}$$

In addition to two failure functions, two plastic functions are defined for Hoek-Brown Model.

$$g_{HB,13} = S_1 - (\frac{1+\sin\psi_{mob}}{1-\sin\psi_{mob}})S_3 \tag{21}$$

$$g_{HB,12} = S_1 - (\frac{1+\sin\psi_{mob}}{1-\sin\psi_{mob}})S_2 \tag{22}$$

Failure surfaces of the Hoek-Brown failure condition in the principal stress space are given in Fig. 9.

3.3 Production planning

While conducting open pit mine planning, the top of the coal will be stripped as 100 meters in the first instance and then the top of coal was designed to be exposed 50 meters at most. The production studies were planned so that after the stripping of coal, the coal will be extracted 50 meters in the advance direction and the extracted part will be backfilled and current production practices have been going on according to this principle. The bench heights are 8 meters, slope widths are 20 meters and the horizontal distance between top of slope and slope tip is taken as 3 meters. During the production, as these principles are adopted, the region where the coal is extracted will be filled until the topography elevation. Under the guidance of these principles, numerical modeling studies were carried out. The recent state of the model created according to

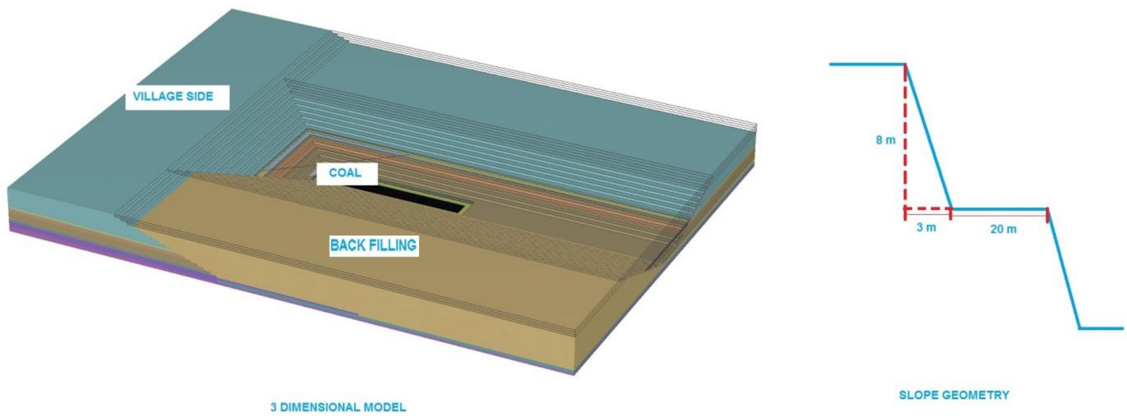


Fig. 10 View of the model and slope geometry

those principles is seen in Fig. 10.

3.4 Numerical modeling studies

Geotechnical data obtained from the drilling studies were integrated into the database and preliminary models were created representing the field. Evaluation of the obtained feedback results, stress-strain characteristics of the field were determined. In addition, due to these studies and feedbacks obtained, restrictions of the field were seen and pre-models for slope stability were created. In the studies conducted, data from Özcelik, Dirik *et al.* (2013) were used. PLAXIS 3D software was used in numerical modeling studies. The width of the model in *x* and *y* directions are 2000 m and 1500 m, respectively. The depth of the model in *z*-direction is 200 m.

In numerical modeling studies, 67 steps are described as given below. Excavation and filling steps are also given as follows.

Starting: This is the stage where the initial conditions are created by applying the K_0 procedure.

1. Stage: This is the stage that takes the form of the current situation of mine model.
2. Stage: It is the case where the first 8 m excavation is done in the model (top to down).
3. Stage: It is the case where the second 8 m excavation is done in the model.
4. Stage-14. Stage: They are the case where excavation between the third and thirteenth 8 m is done in the model (until bottom of the mine)
15. Stage: First bench excavation (8m) towards progressing direction (top of mine)
16. Stage: Second bench excavation (8m) towards progressing direction
17. Stage-27. Stage: Bench excavations between third bench and thirteenth bench (every benchs are 8 m) towards progressing direction
28. Stage: First bench backfill (8m) at the back (from down to top of back of produced area)
29. Stage: Second bench backfill (8m) at the back
30. Stage-40. Stage: Backfilling from third bench to thirteenth bench (every backfilling is 8m) at the back of the produced area
41. Stage-53. Stage: Repeating from 15. Stage to 27. Stage progressing direction

Table 1 Rock mass parameters used in numerical modeling analysis

Identification	γ_{unsat} kN/m^3	γ_{sat} kN/m^3	E_{mass} knew/m^3	v	σ_{ci} knew/m^3	c_{rdf} knew/m^2	φ (*phi*)	D	K_{0x}	K_{0y}
Sandstone-1 with Claystone interband	18.76	18.76	1.65E+05	0.37	4600	264	34	1	0.6744	0.6744
Claystone-1	18346	18.46	7.60E+04	0.37	6800	211	21.5	1	0.658	0.658
Sandstone-2 with Claystone interband	17.65	17.65	8.78E+05	0.38	9700	482	28.5	1	0.658	0.658
Sandy Claystone	20.75	20.75	1.40E+05	0.39	4000	176	27.3	1	0.6416	0.6416
Gravelly Sandstone	17.98	17.98	3.35E+05	0.36	1.21E+04	667	33	1	0.6744	0.6744
Sandstone-3 with Claystone interband	18.37	18.37	3.46E 05	0.37	4400	240	31.2	1	0.658	0.658
Claystone-2	17.26	17.26	3.94E+05	0.37	1.10E+04	582	31.9	1	0.6744	0.6744
Clay	17.45	17.45	5.05E+04	0.35	8900	277	21.5	1	0.691	0.691
Claystone-3	18.19	18.19	1.10E+05	0.39	9900	385	25.3	1	0.658	0.658
White Mudstone	17.51	17.51	3.39E+05	0.38	1.09E+04	501	27.4	1	0.6416	0.6416
Claystone-4	16.76	16.76	7.04E+04	0.4	9900	314	21.8	1	0.691	0.691
Lignite	12.46	12.46	1.50E+05	0.37	2200	324	21.4	1	0.6093	0.6093
Claystone-5	19.21	19.21	2.83E+05	0.36	3800	231	32	1	0.546	0.546
Sandstone	15	15.	1.95E+05	0.34	9800	528	32.8	1	0.5616	0.5616
Congtomerate	20.12	20.12	5.64E+04	0.35	6400	230	24.9	1	0.4701	0.4701
Tuffite	19	19	8.37E+04	0.35	1.04E+04	350	26.2	1	0.5585	0.5585
Limestone	20	20	1.04E+06	0.28	4.51E+04	500	38	1	0.3843	0.3843
Backfill	18	18	3501.96	0.35	400	3	5.37	1	0.5774	0.5774

54. 54. Stage-66. Stage: Repeating from 28. Stage to 40. Stage to back of produced area
67. After all excavation steps in the model, determination of safety factor for slope stability by Phi-c reduction method.

Same rock mass parameters for numerical modeling were selected for Hoek-Brown and Mohr-Coulomb failure criteria. Those parameters are given in Table 1. Numerical modeling studies using Hoek-Brown failure condition failed in stage 10 given above. Numerical model using Mohr-Coulomb failure criteria, however, did not failed even after coal production stages. Considering the structure and geological state of the region, failure criteria representing shear strength can give better results.

In numerical modeling studies, primarily, the present situation was simulated and after this stage, production was carried out until coal layer. After 100m excavation from the coal, excavation continues towards the advance direction and excavated area is used as backfill area in the model. Excavation and backfill stages were integrated into the model totally in 66 stages. Last stage is determination of safety factor of slope by Phi-c Reduction method.

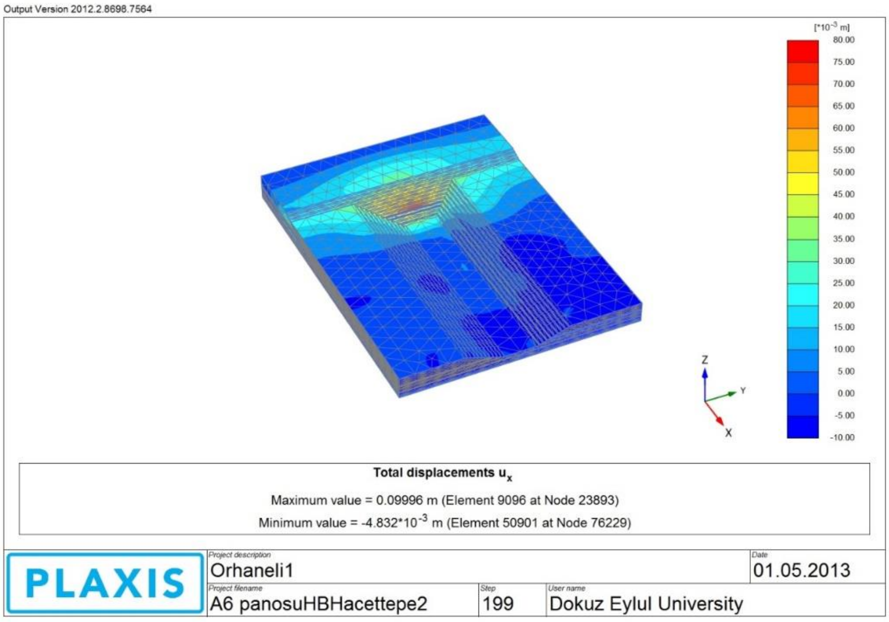


Fig. 11 Horizontal deformation occurring at stage 10 of the model

3.4.1 Results of numerical modeling studies

The numerical modeling studies within the scope of studies related to whether the stability of the slope geometry for coal production was carried out by taking into consideration two different failure conditions: Hoek-Brown Failure Criteria and Mohr-Coulomb Failure Criteria. In both applications, all conditions belonging to two models (initial and boundary conditions) were taken the same. Groundwater level was selected as -12 m. As mentioned before, analysis done by using Hoek-Brown failure criteria failed in the stage 10 (72 m deep from the first excavation stage). Therefore, only results of this model are given in this part. There is no failure as a result of the modeling by using Mohr-Coulomb failure criteria and it is determined that mine will be working under stable conditions.

The analysis done by using Hoek-Brown failure criteria to determine whether slopes created for production are stable or not, failed in the stage10 (72 m deep from the first excavation stage). When drilling results were examined, at the depth where failure occurred, clay bands were seen.

Deformation condition after stage 10 is given below in Fig. 11. In this analysis, 9.99 cm horizontal deformation was determined in the final slopes towards the village. It was determined that clay level could not respond to this deformation and eventually, failure occurred.

Numerical model using Mohr-Coulomb failure criteria did not fail even until coal production stage started. The maximum horizontal deformation amount in the village side was found to be 7.50 cm. Deformation situation that occurred in the last excavation stage is given in detail in Fig. 12.

Discussing the numerical modeling results of a research, the analysis conducted by Mohr-Coulomb failure criteria seems to provide more immediate results to the field studies.

The analysis made by Hoek-Brown Failure Criteria presents failure in stage 10, when 9.99 cm horizontal deformation occurs. In the analysis by Mohr-Coulomb Failure Criterion, however, all of

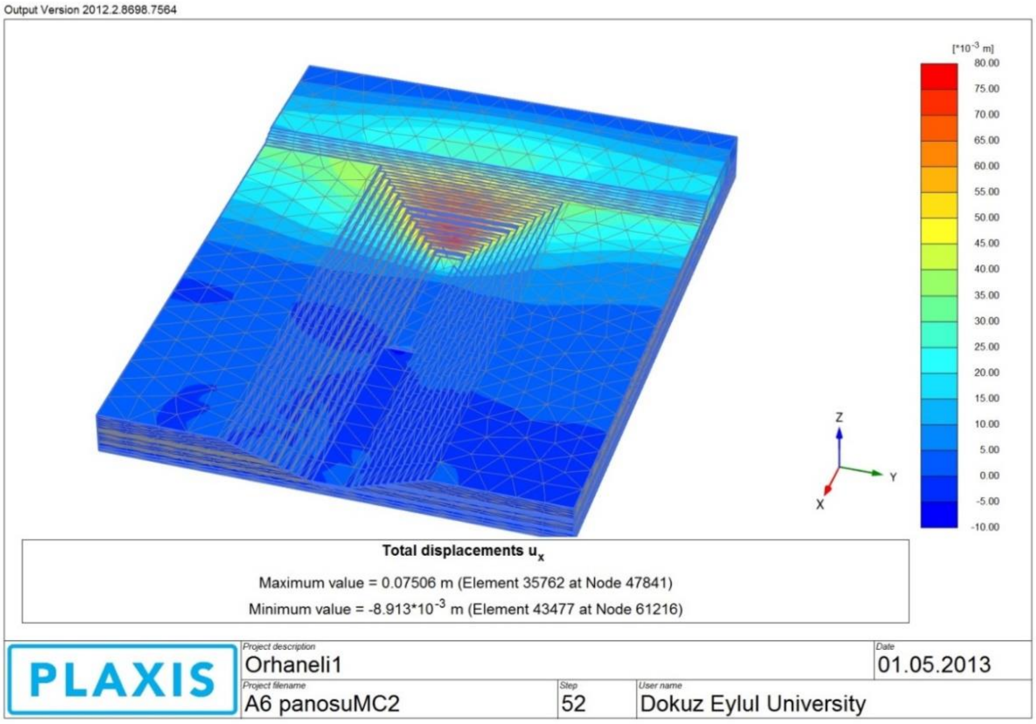
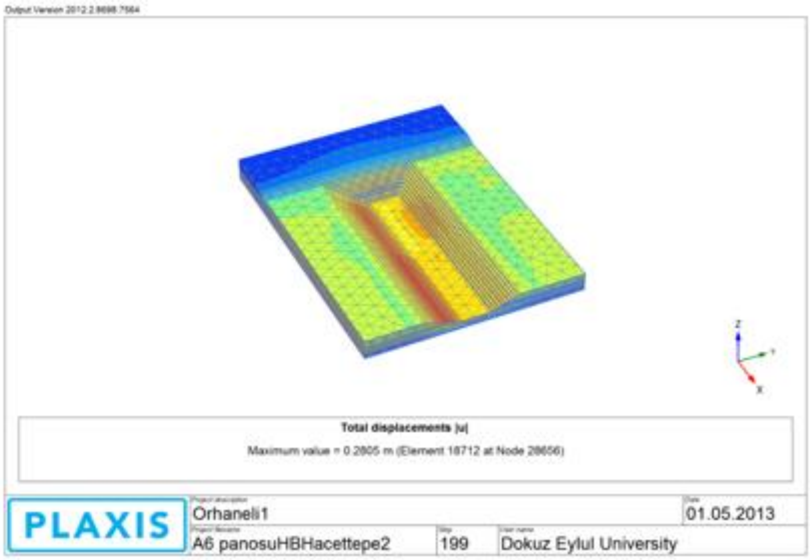


Fig. 12 Horizontal deformations occurring in stage 66 of the model.

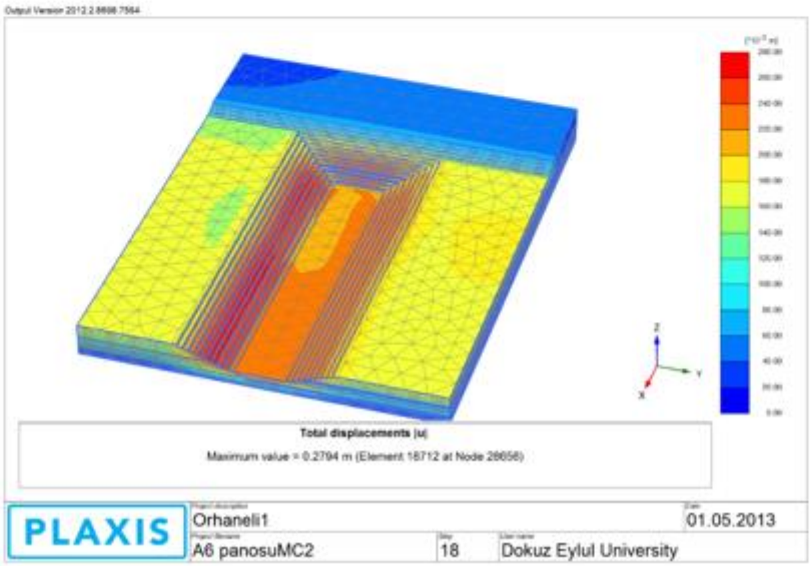
the stages were solved without failure and the total amount of horizontal deformation was 7:50 cm (Fig. 12). The main reason of this case is that; Hoek and Brown Failure Criteria has been developed for rock masses. Therefore, there is a difficulty in defining the clay bands, expressed as ground, in the formulation. In fact, rock parameters (s, m_b) in the definition of Hoek and Brown Failure Criteria cannot show sufficient accuracy in the definition of ground such as clay. At this point, it is useful to examine the total deformation results in the analysis stage 10 for both models. Fig. 13(a) and 13(b) illustrate the total deformation results at stage 10 according to Hoek-Brown Failure Criteria (failure occurred at this stage) and Mohr-Coulomb Failure Criteria (no failure at this stage), respectively. The total amount of deformation in both models stands out at this stage is quite close. While the maximum total amount of deformation according to the results of analysis performed by Hoek-Brown Failure Criteria was 28.05 cm that of analysis performed by Mohr-Coulomb Failure Criterion was identified by 27.94 cm. Fig. 13 (a) and (b) illustrate the deformation amounts at the same and different points at stage 10 for both models. Although the total deformation amounts are so close, the horizontal deformation difference is 2.5 cm between two failure criteria leading to the failure in numerical model performed by Hoek-Brown.

4. Investigations and measurements conducted in the field

For the production of open pit mine, panel A6, belonging to Turkish Coal Enterprises, slope stability studies (Aksoy, Onargan *et al*. 2013) were conducted along with some inclinometer



(a)



(b)

Fig. 13 Comparison of the 10th stage of Hoek-Brown and Mohr-Coulomb Failure Criterion (a) Hoek Brown; (b) Mohr Coulomb

measurements at the critical points (Gökay, Özkan *et al.* 2013). Long-term reading was made from these inclinometers. Basic result of these measurements is that deformation rate in A6 panel after the production which was carried out according to Aksoy *et al.*'s suggestions (Aksoy, Onargan *et al.* 2013) decrease comparing to the case previous to this modeling studies Gökay, Özkan *et al.* 2013). Inclinometer measurements and analysis were available from the report prepared by four universities (Özçelik, Özkan *et al.* 2013). Moreover, blasting effects to the slope stability was

Table 2 Picture showing the situation of the mine at different times (long term observations)

		
A view from the open pit on 08/07/2013	A view from the open pit on 15/08/2013	A view from the open pit on 22/12/2013
		
A view from the open pit on 25/06/2014	A view from the open pit on 08/07/2014	A view from the open pit on 21/08/2014
		
A view from the open pit on 19/09/2014	A view from the open pit on 09/10/2014	A view from the open pit on 13/11/2014

investigated by the researchers from Ankara University Geophysical Department and blasting patterns were suggested in order to minimize those effects both for the slopes and for the nearby village (Aldaş and Kaypak 2013). While the production continued in panel A6, at some period, suggested blasting pattern was not applied and increase in the deformation rate was observed. This data is not given here because this study was conducted under the supervision of the other project partner. This paper is only related with evaluation of the performances of Hoek-Brown and Mohr-Coulomb failure criteria.

Monitoring studies continued in Gümüspınar surface mine after finishing the study by monthly technical visits. Mining facilities were monitored time-dependent. Table 2 illustrates the picture taken from the mine at different times.

5. Results and discussions

In order to monitor the stability of the slopes due to mining production facilities in Gümüşpınar surface mine, numerical modeling analysis was carried out by using Hoek-Brown and Mohr-Coulomb failure criteria. Prior to modeling studies, a blasting pattern design was suggested by using newly developed method of co-Author (Aldaş and Ecevitoğlu 2007) to minimize blasting effects on the slopes. Determination of the fact that using new blasting pattern minimizes the effects of vibrations to both slopes and nearby settlements, disturbance factor, *D* was selected as 1 while determining rock mass parameters in modeling studies. Same input data were selected in the numerical analysis using both Hoek-Brown and Mohr-Coulomb criteria. Moreover, same boundary and initial conditions were used in all numerical models. In both models, K_0 procedure was used as initial condition. As it is known, K_0 is expressed as the ratio of horizontal stresses to vertical stresses. After analyzing by Hoek-Brown Failure Criteria, model happened to fail in Stage 10 (after 72 m excavation from the present case). However, the model using Mohr-Coulomb Failure Criteria with the same rock mass parameters and conditions, did not fail even until coal production started.

Production facilities in the open pit were applied within a certain discipline. After excavation of top coal, both production and overburden excavations continued in progression direction. The region where coal was produced was backfilled immediately. Long-term monitoring and observations stated that no stability problems arose in the mine recently. Only, local small rock falls were observed but those are very limited in size that will not affect coal productions.

6. Conclusions

Gümüşpınar Surface Mine has been engaged in production for many years and the mine at the moment has rather deepened.With the deepening of the mine, thick and weak clay bands become active at various depths. Therefore, there have been significant landslides, approximately 2.5 million metercube in volume, in time. In order to prevent these problems, extensive studies on slope stability again were realized. To monitor the performance of slope stability, numerical modeling methods were used. The most important elements of the numerical model is to reflect the engineering experience to the model. For this purpose, after evaluation of the measurements and experiments in the field, the solutions were carried out with the two failure models (Hoek - Brown and Mohr-Coulomb Failure Criteria) which were considered as the best representative of the field. During the analyses, the model formed by Hoek - Brown Failure Criterion was failed at the stage 10 corresponding to the depth of the clay band. However, the model formed by Mohr-Coulomb Failure Criteria with the same conditions and same stages was not failed.

The work during the excavation in the field has been monitored over a long period. The results obtained from the field were compatible with the results demonstrated by Mohr-Coulomb Failure model. No landslides were faced with. Considering these results, it is clear that Hoek -Brown failure criteria shows limited success in the formation such as clays illustrating soil properties. The main reason of this is that Hoek-Brown Failure Criteria solutions are provided according to the rock mechanics parameters rather than soil parameters.

As a result of this study, it is seen that, Hoek-Brown Failure Criteria have great difficulties in representing the region for the deep surface mine having clay content. As for the Mohr-Coulomb Failure Criteria, it is more realistic that weak rock should be defined in the model. Problems due to definition of clay intermediate bands having soil properties by Hoek-Brown parameters can be overcome by using Mohr-Coulomb Failure Criteria in these types of field.

Landslides are time-dependent mechanisms in this type of deep mines having weakly rock masses. For these reasons, studies consist of determining the time-dependent deformation characteristics and deformation amounts and predictions of the risks due to deformations have still continued. Those studies will be the subject of another paper. However, it is better now to define that, Mohr-Coulomb Failure Criteria and material models have been used in newly time-dependent analysis.

Acknowledgements

Authors thank to Turkish Coal Enterprises (TKİ) which gives financial supports. The great thanks go to Mustafa AKTAS, Mustafa OZDINGIS, Ali SAGIR and Kenan EMIRALIOGLU from TKI.

References

Aksoy, C.O., Oğul, K., Topal, İ., Ozer, S.C., Özacar, V. and Posluk, E. (2012), "Numerical modeling of non-deformable support in swelling and squeezing rock", *Int. J. Rock Mech. Min*., 5261-70.

Aksoy, C.O., Onargan, T., Pamukçu, Ç., Küçük, K. and Özacar, V. (2013), "The Determination of optimal evaluation condition of gümüşpinar, sağirlar and civili coal field connected to bursa lignite management of TKİ-GLİ and research of the solution of landslides on gümüşpinar section, revised project", The Prepare The Primary Project of Sağirlar and Çivili Underground Reserves for Production, Appendix-2. (In Turkish)

Aldaş, G.G. and Kaypak, B. (2013), "The determination of optimal evaluation condition of gümüşpinar, sağirlar and civili coal field connected to bursa lignite management of TKİ-GLİ and research of the solution of landslides on gümüşpinar section", Revised Project, The Prepare The Primary Project of Sağirlar and Çivili Underground Reserves for Production, Appendix-4. (In Turkish)

Aldaş, G.G.U. (2002), "Effect of some rock mass properties on blasting induced ground vibration wave characteristics at Orhaneli surface coal mine", Ph.D. thesis, Middle East Technical University, Department of Mining Engineering, Ankara, Turkey.

Aldaş, G.G.U. and Ecevitoğlu B. (2008), "Waveform analysis in mitigation of blast-vibration", *J. Appl. Geophys.*, **66**, 25-30.

Aldaş, G.G.U. and Ecevitoğlu, B. (2007), "Methodology for minimising blast-induced ground vibration", TPE. 2007/03459.

Casciati, S., Chassiakos, A.G. and Masri, S.F. (2014), "Toward a paradigm for civil structural control", *Smart Struct. Syst.*, **14**(5), 981-1004.

Chen, B., Xu, Q. and Sun, D. (2014), "An elastoplastic model for structured clays", *Geomech. Eng.*, **7(**2), 213-231.

Chen, C.H., Zhu, Y.F., Yao, Y. and Huang, Y. (2016), "The finite element model research of the pre-twisted thin-walled beam", *Struct. Eng. Mech*., **57**(3), 389-402.

Eberhardt, E. (2012), "The Hoek-Brown failure criterion", *Rock Mech. Rock Eng.* **45**(6), 981-988.

Feng, X.T. and Hudson, J.A. (2008), "Specifying the required information for rock engineering modeling and design", *Int. J. Rock Mech. Min. Sci.*, **47**, 179-94.

Gökay, M.K., Özkan, İ., Özşen, H., Doğan, K. and Mesutoğlu, M. (2013), "The determination of optimal evaluation condition of gümüşpinar, sağirlar and civili coal field connected to bursa lignite management of TKİ-GLİ and research of the solution of landslides on gümüşpinar section", Revised Project, The Prepare The Primary Project of Sağirlar and Çivili Underground Reserves for Production, Ek-3. (In Turkish)

Griffith, A.A. (1920), "The phenomena of rupture and flow in solids", *Philos. Trans. R. Soc. Lond. Ser. A. Math. Phys. Sci.*, **221**(587), 163-198.

Griffith, A.A. (1924), "The theory of rupture", *Proceedings of the First International Congress for Applied Mechanics*, Delft. J. Waltman Jr, Delft.

Günalay, M.E. (1974), "Bursa ili-Orhaneli ilçesi-Yenidanişment Köyü Civarında Bulunan Maden Kömürü sahası İşletme Projesi", MTA Report No: 5271, p:43. (in Turkish)

Hoek, E. (1968), *Brittle Failure of Rock*, Eds. Stagg, K.G., Zienkiewicz, O.C., Rock mechanics in engineering practice, Wiley, New York.

Hoek, E. (1983), "Strength of jointed rock masses", 23rd Rankine Lecture, *Géotechnique*, **33**(3), 187-223.

Hoek, E. and Bray, J.W. (1981), “Rock slope engineering”, Institution of Mining and Metallurgy, London.

Hoek, E. and Brown, E.T. (1980), “Underground excavations in rock”, The Institution of Mining and Metallurgy, London.

Hoek, E. and Brown, E.T. (1988), “The Hoek-Brown failure criterion-a 1988 update”, *Proceedings of the 15th Canadian Rock Mechanics Symposium*, University of Toronto, Toronto.

Jaeger, J.C. and Cook, N.G.W. (1979), *Fundamentals of Rock Mechanics*, 3rd Edition, Chapman & Hall, London.

John, K.W. (1968), “Graphical stability analysis of slopes in jointed rock mass”, *J. Soil Found. Div.*, ASCE, **94**(SM2), 497-523.

Karpuz, C., Koçyiğit, A., Tutluoğlu, L., Düzgün, Ş., Koçal, A., Erdem, E. and Alkılıçgil, Ç. (2006), “Türkiye Kömür İşletmeleri Kurumu TKİ, Orhaneli İşletmesi Açık Ocakları Panolarında Şev Tasarımı ve Dragline Çalışma sisteminin Belirlenmesi Projesi”, Proje kod no: 05-03-05-2-00-13. (in Turkish)

Loh, C.H., Huang, Y.T., Hsiung, W.H., Yang, Y. and Loh, K.J. (2015), “Vibration-based identification of rotating blades using Rodrigues”, *Wind Struct.*, **21**(6), 677-691.

Londe, P., Vigier, G. and Vormeringer, R. (1969), “Stability of rock slopes-a three dimensional study”, *J. Soil and Found. Div.*, ASCE, **95**(SM1), 235-262.

Memikoğlu, O. (1976), “Bursa-Orhaneli Kömür Yatağı Fizibilite Araştırması”, Cilt 1-Jeoloji. MTA Report no: 7112. (in Turkish)

Men, J., Guo, Z. and Shi, Q. (2015a), “Experimental research on seismic behaviour of novel composite RCS joints”, *Steel Compos. Struct.*, **19**(1), 209-221.

Men, J., Zhang, Y., Guo, Z. and Shi, Q. (2015b), “Experimental research on seismic behavior of a composite RCS frame”, *Steel Compos. Struct.*, **18**(4), 971-983.

Nadai, A. (1950), *Theory of Flow and Fracture of Solids*, McGraw Hill, New York.

Özçelik, Y., Dirik, K., Cengiz, K., Özsayın, E. and Yılmazkaya, E. (2013), “The determination of optimal evaluation condition of gümüşpinar, sağirlar and civili coal field connected to bursa lignite management of TKİ-GLİ and research of the solution of landslides on gümüşpinar section”, Revised Project, The Prepare The Primary Project of Sağirlar and Çivili Underground Reserves for Production, Appendix-1.

Yaylac, M. (2016), “The investigation crack problem through numerical analysis”, *Struct. Eng. Mech.*, **57**(6), 1141-1156.

CC