

Back analyses for slope failures in rock

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ABSTRACT

Some of the geotechnical parameters used in the analysis may not be accurately measured directly from laboratory tests due to effects of sample disturbance and errors of tests. The back analysis or the observational method are thus often applied to determine the representative and/or dominant strength parameters based on field observations in practice. Based on the Hoek-Brown failure criterion, it was known that the disturbance factor (D) should be determined with caution. The difficulty of measuring rock mass disturbance accurately has been shown. Three rock slope failures will be investigated in this paper using back-analysis technique. In addition, various rock mass strength parameters are taken into consideration.

Keywords: Hoek-Brown, landslide, limit equilibrium method

1 INTRODUCTION

Predicting the stability of rock slopes is often the problem for geotechnical engineers. There are a lot of researchers have focused on assessing the stability of rock slope but it is still a significant challenge to designer. However, back analysis is a common method used to assess strength parameters when there is a slope failure. It might also improve the knowledge in input parameters. In general, estimating the stability of the rock slope is always hard jobs for engineers because the nature of the variable which usually contains a jointed rock mass fracture, naturally occurring discontinuities and anisotropy.

Limit equilibrium method (LEM) is the most widely used approach to evaluate the slope stability. However it is known that the Mohr-Coulomb soil parameters are still required as inputs when using most of commercial software based on the limit equilibrium theory. The parameters for Mohr-Coulomb are cohesion (c') and friction angle (ϕ'). In fact, using the Mohr-Coulomb failure criterion will ignore completely the non-linear nature of the rock mass failure envelope. Fu and Liao (2010) indicated that non-linearity is operational at the low confining stresses, such as slope stability problems.

A non-linear empirical yield criterion was proposed by Hoek et al. (2002) which can estimate rock mass strength more accurately than the conventional Mohr-Coulomb failure criterion. Recently, the Hoek-Brown failure criterion (Hoek et al. 2002) has been applied to bearing capacity and slope stability by Merifield et al. (2006) and Li et al. (2008, 2009, 2011)

respectively. The latest Hoek-Brown failure criterion for rock masses is expressed as the following equations:

$$\sigma_1' = \sigma_3' + \sigma_{ci} \left(m_b \frac{\sigma_3'}{\sigma_{ci}} + s \right)^a \quad (1)$$

where

$$m_b = m_i \exp \left(\frac{GSI - 100}{28 - 14D} \right) \quad (2)$$

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

$$a = \frac{1}{2} + \frac{1}{6} \left(e^{-GSI/15} - e^{-20/3} \right) \quad (4)$$

The magnitudes of m_b , s and a rely on the geological strength index (GSI), which describes the rock mass quality. The range of GSI is between 5 and 100. GSI was introduced to estimate the rock mass strength for different geological conditions. σ_{ci} and m_i represent the intact uniaxial compressive strength and material constant respectively. The parameter D is a factor that depends on the degree of disturbance whose range is between 0 and 1. Greater details on how to estimate the Hoek-Brown strength parameters can be found in Wyllie and Mah (2004) and Marinos et al. (2005).

As highlighted by Burland (1989), some of the geotechnical parameters used in the analysis may not be accurately measured directly from laboratory tests due to effects of sample disturbance and errors of tests. The

back analysis or the observational method, as suggested by Peck (1969), is thus often applied to determine the representative and/or dominant strength parameters based on field observations in practice.

In this study, commercial software, SLIDE, are adopted as tool to perform back analyses for case studies. This software is suitable for analysing rock slopes as Hoek-Brown failure criterion has been written in it. The failed cases are obtained from presented papers. They are selected because there is no thorough investigation performing back calculation based on the latest version of the Hoek-Brown failure criterion (Hoek et al. 2002). Regarding these cases, detailed information is described in the discussion of each case.

2 CASE STUDIES

In this section, three failed slopes in rocks presented by Sonmez and Ulusay (1999) are examined. This study would be helpful for engineers to understand the application of the Hoek-Brown failure criterion.

2.1 Case 1: Slope failure in closely jointed rock mass in barite open pit mine

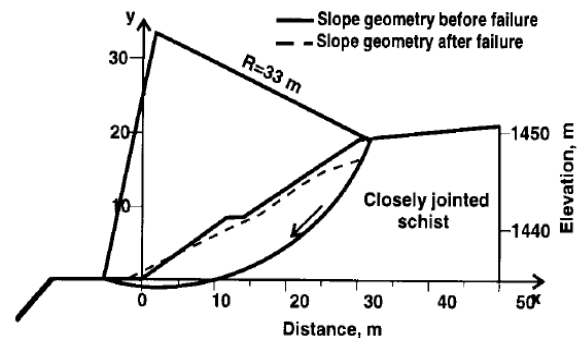
The rock slope was located at Baskoyak barite open pit mine, in western Anatolia. Due to the heavily jointed nature of the schist, the rock mass was assumed as homogeneous and isotropic. The mean unit weight (γ) and uniaxial compressive strength (σ_{ci}) of the heavily broken part of the schist are 22.2kN/m^3 and 5.2MPa , respectively. Other parameters required can be obtained in Sonmez and Ulusay (1999) and Sonmez et al. (2003) where $m_i = 7$ and $GSI = 16$. As indicated by Sonmez and Ulusay (1999), no sign of groundwater was encountered. Thus, the pit slopes were treated as dry for stability assessments.

Due to the fact that Sonmez and Ulusay (1999) used different measurement to define rock mass disturbance, this study proposes to back calculate disturbance factor (D). Based on Bishop's simplified method (Bishop 1955), the obtained factor of safety (F) is 1.007 with the disturbance factor (D) of 0.68. $D = 0.68$ is very close to the result investigated in the study of Li et al. (2011) where $D = 0.7$. Since the overburden material and the ore are removed by excavators without any blasting, the disturbance factor $D = 0.7$ can be adopted. The result obtain for this case agrees well with the suggestion of Hoek et al. (2002). Fig. 1 shows the failure surface obtained from SLIDE which is similar to that presented by Sonmez and Ulusay (1999).

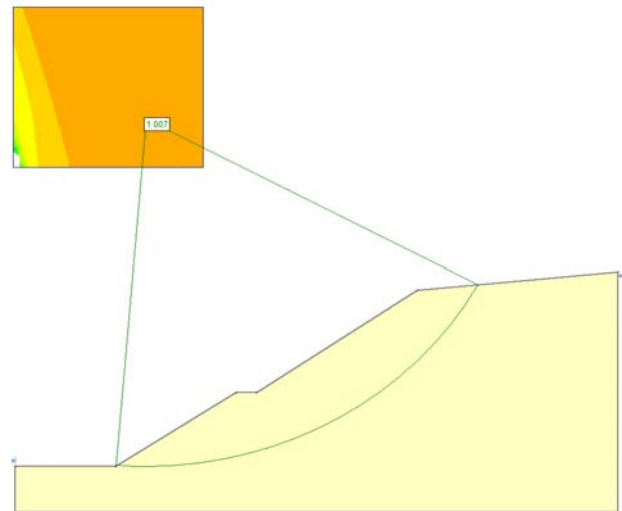
2.2 Case 2: Slope instability in coal mine in western Turkey

This example of rock slope instability originates from the Kistrakdere open pit mine located at Soma lignite basin, western Turkey. The necessary data collected by Sonmez and Ulusay (1999) shows the geometry of the failed slope in which a single thin coal seam with a thickness of 4.5m is overlain by a sequence

of compact marl and soft clay beds about 10m of thickness. The observations of slope surfaces and available records indicated that the groundwater was below the failed marly rock mass, and the coal seam acted as an aquifer. The marly rock with a uniaxial compressive strength of 40MPa and $m_i = 9.04$ has a carbonate content more than its clay content. In addition, $GSI = 16$ and $\gamma = 21\text{kN/m}^3$ are known. The observed actual slip surface was of circular shape and passed through the compact marl rock mass and along the clay bed, above the coal seam.



(a) Failure mechanism adapted from Sonmez and Ulusay (1999)

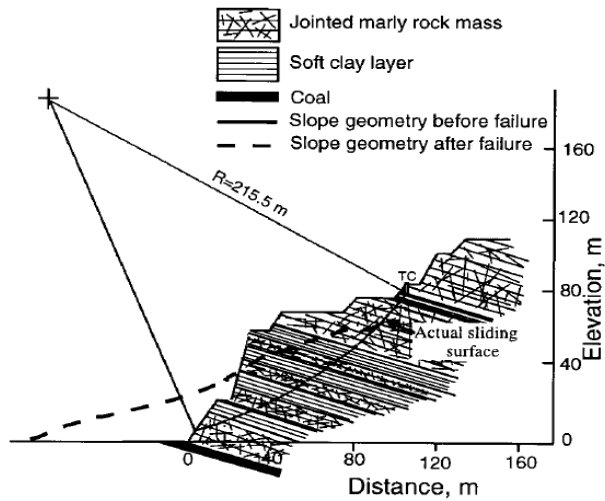


(b) Failure mechanism obtained from SLIDE

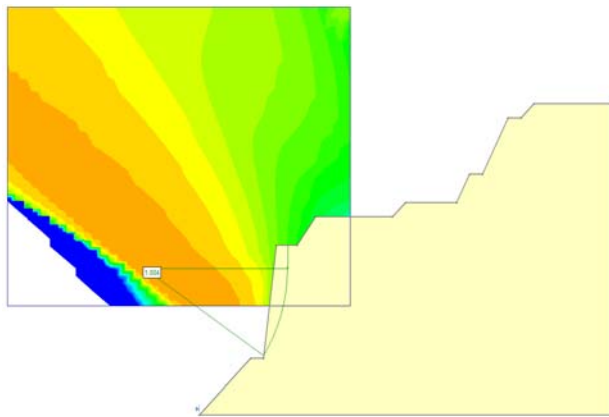
Fig. 1. Failure mechanisms for Case 1.

Based on the back calculation approach, the obtained factor of safety (F) is 1.004 with the disturbance factor (D) of 0.9. Based on the suggestion of Hoek et al. (2002), $D = 0.9$ could be classified as large scale overburden removal. In fact, the total slope height for this case is around 110m . It implied that the obtained result is reasonable. Figure 2 shows the failure surfaces presented in Sonmez and Ulusay (1999) and this study. In fact, two different failure mechanism can be seen. Li et al. (2011) also investigated this case

using numerical upper and lower bound limit analysis methods (Lyamin and Sloan 2002a and 2002b) and indicated that this slope is very close to instability. One more reason for the discrepancies between analysis and observation for this case would be the fact that the slope is strongly heterogeneous along its height.



(a) Failure mechanism adapted from Sonmez and Ulusay (1999)



(b) Failure mechanism obtained from SLIDE

Fig. 2. Failure mechanisms for Case 2.

2.3 Case 3: A bench failure in a coal mine

Turkish Coal Enterprises (TKI) operated an open pit coal mine namely Himmetoglu where located in north-west Anatolia and produce low calorific value of coal. This bench failure happens in 1998 in the eastern slope, excavated in heavily jointed marly rock mass. From Sonmez and Ulusay (1999) and Sonmez et al. (2003), $\sigma_{ci} = 4.8\text{MPa}$ and $m_i = 10$, $GSI = 27$ and $\gamma = 18.5\text{kN/m}^3$. Based on the study of Sonmez and Ulusay (1999), it was indicated that the residual shear strength parameters of the weak and slickensided bedding planes were $c_r = 1.4\text{kPa}$ and $\phi_r = 12^\circ$. Due to the fact that the

failure mechanism for this case is not circular (Fig. 3), Janbu's method (Janbu et al. 1956) has been used.

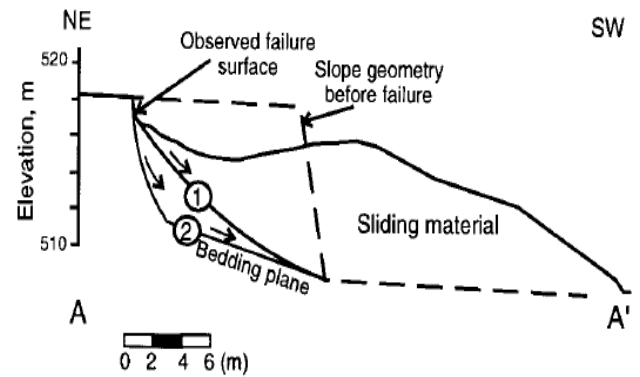


Fig. 3. Failure mechanisms for Case 3 adapted from Sonmez and Ulusay (1999).

Based on above information, back calculation was undertaken in order to find D . However, various magnitudes of D are used as input. The obtained F is always less than 1, even if $D = 0$ is employed. It should be noted Sonmez et al. (2003) estimates the degree of the rock mass disturbance is similar to Case 1 and thus D should be around 0.7. Therefore more detailed analyses should be done. For this case, several failure mechanisms are investigated firstly, as shown in Fig. 4. In fact, these three failure mechanisms are quite similar.

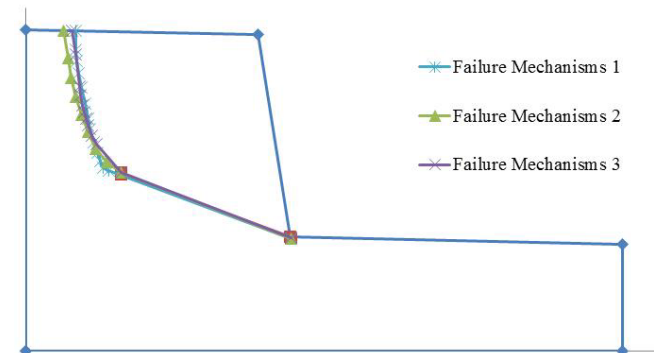


Fig. 4. Various failure mechanisms for Case 3 investigated in this study.

The back calculated F for different failure mechanisms and D values are shown in Table 1. It can be seen that the difference in F between Failure mechanisms 2 and 3 is the most significant. Although the difference in F can achieve by up to 17%, none of them is greater than 1. It means that the difference in failure surface is not the only reason to cause $F < 1$.

The authors believed that the uncertainties from other input parameters would also influence the back analysis result. Based on the studies of Hoek (1998) and Li et al. (2012), σ_{ci} and m_i , can distribute normally with coefficient of variation (COV) values of 0.25, 0.125 respectively. GSI also distributed normally with standard deviation ($Stdev$) of 2.5. Using the COV and

Stdev can consider dispersion of each parameter.

Table 1. Back calculated F .

Failure Mechanism 1				
σ_{ci} (MPa)	m_i	GSI	D	F
4.8	10	27	0.7	0.793
4.8	10	27	0.5	0.823
4.8	10	27	0	0.879
Failure Mechanism 2				
σ_{ci} (MPa)	m_i	GSI	D	F
4.8	10	27	0.7	0.819
4.8	10	27	0.5	0.872
4.8	10	27	0	0.972
Failure Mechanism 3				
σ_{ci} (MPa)	m_i	GSI	D	F
4.8	10	27	0.7	0.677
4.8	10	27	0.5	0.709
4.8	10	27	0	0.768

It is known that 99.7% observations fall within $\pm 3Stdev$ of the average value for a parameter in the normal distribution. This study considered that the original $\sigma_{ci} = 4.8\text{MPa}$, $m_i = 10$ and $GSI = 27$ are as the average values. Trial and error are employed by increasing σ_{ci} , m_i and GSI values which must be within $3Stdev$. This approach would be helpful for back calculation until $F = 1$.

The above three failure mechanisms are still adopted for this part of study. Based on a large numbers of trials, $F = 0.966$ can be obtained for Failure mechanism 2 when $\sigma_{ci} = 8.4\text{MPa}$, $m_i = 13.75$, $GSI = 34.5$ and $D = 0.7$. It should be noted that F for other failure mechanisms are smaller than that for Failure mechanism 2. In addition, the magnitudes of σ_{ci} , m_i and GSI all are the average adding $3Stdev$. This should be considered as an extreme case. In fact, the above F is still less than 1, and therefore D was adjusted. Finally, $F = 1.002$ was carried out when $\sigma_{ci} = 8.4\text{MPa}$, $m_i = 13.75$, $GSI = 34.5$ and $D = 0.6$ for Failure mechanism 2.

It is interesting that using original presented σ_{ci} , m_i and GSI values and similar failure mechanisms is difficult to achieve $F = 1$. It would be due to the fact that rock mass disturbance for Hoek-Brown yield criterion (Hoek et al. 2002) is presented differently from that used by Sonmez and Ulusay (1999). However, as discussed previously, D should be around 0.7 because Sonmez and Ulusay (1999) gave a same magnitude as Case 1. Due to the fact that limited information for this case is available, it is suggested that more investigations are needed.

3 CONCLUSIONS

A better understanding of mechanics of jointed rock mass behaviour always is the major problem for the geotechnical engineering. In fact, the Hoek-Brown failure criterion has gained an increasing popularity in stability analysis made in conjunction with rock mass classification system. It can provide a good estimate for

the shear strength of closely jointed rock masses. Based on Hoek-Brown failure criterion, this study used conventional limit equilibrium method to perform back analyses for three failed rock slopes. However, the results obtained are not exactly agreed with those presented by Sonmez and Ulusay (1999). It was recommended that more detailed investigations are required.

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