Probabilistic back analysis and their reliabilty of plane failure: A case study from trabzon-gumushane highway, NE Turkey

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PROBABILISTIC BACK ANALYSIS AND THEIR RELIABILTY OF PLANE FAILURE: A CASE STUDY FROM TRABZON-GUMUSHANE HIGHWAY, NE TURKEY

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ABSTRACT

This study discusses the prior information of input data from both laboratory tests and deterministic back calculation methods that are to be considered by probabilistic back analysis method. For this purpose, the failure based on discontinuity surface, plane failure, occurred at natural slope on the Trabzon highway, NE Turkey, in May 2008. In order to obtain the shear strength parameters of the sliding surface, the deterministic back analyses with a comparison with laboratory derived shear-box test and considering these value and water condition as input parameters, the probabilistic back analyses were performed. A measure of the reliability of the back calculated shear strength parameters were able to develop. For this, the probabilistic stability analysis for the failure conditions obtained by the deterministic and probabilistic back analysis were performed and standard deviation of the safety factor and slope failure probability were obtained the landslide investigated.. In this analysis, Monte Carlo method was applied. These analyses clearly reveal that the result of the probabilistic back analyses is a more representative according to the deterministic back analyses for at the failure condition of the landslide investigated. And the shear strength parameters obtained by the comparison laboratory derived shear-box test results and deterministic back analyses can be used as a input parameter for the probabilistic back analysis.

Keywords: Plane failure, back analysis, probabilistic method, reliability, NE Turkey

INTRODUCTION

Back-analysis approach has been widely applied to identify in-situ stress field rock mass deformation modulus and strength parameters, rock mass hydraulic properties, rock mass zoning, boundary conditions, loads acting on the tunnel linings, etc., through direct application of closed-form solutions or numerical methods [1], [2]. In the back analysis of slope failure, general shape of sliding surface and volume of failed mass are known, while the shear strength parameters of sliding surface are unknown. Based on this information, the knowledge on these parameters are updated which are unknown at the moment of slope failure [3],[4]. Limit equilibrium techniques are commonly adopted methods due to their simplicity for structurally controlled slopes [5]. Gioda [6] points out that a distinction of back-analysis methods can also be made considering deterministic methods and probabilistic approach. The deterministic back analysis methods for the slope failure under influence of an earthquake or a blasting methods can be classified under two headings, pseudo-static [7]-[10] and dynamic methods based on

Newmark's displacement-type analysis [8]. In a deterministic method, the slope stability model is usually believed or assumed accurate, and the purpose of back analysis is to find a set of parameters that would result in the slope failure [11],[12]. At a minimum, the back-calculated strength from the methods can be used to verify the strength values from the laboratory measured [10]. In a probabilistic back analysis, it is recognized that the slope stability model may not be perfectly accurate and numerous combinations of slope stability parameters may result in slope failure [4]. There can be a quantifiable degree of error in the measuring procedures or when an initial estimate of the descriptive statistics of the governing parameters can be made, then a probabilistic type of back-analysis is more appropriate [13]. Zhang et al. [3] proposed two efficient methods based on a system identification approach derived from Bayesian theory for probabilistic back analysis of slope failures.

This study discusses the prior information of input data from both laboratory tests and deterministic back calculation methods that are to be considered by probabilistic back analysis method as suggested by Zhang et al. [3]. For this purpose, three landslides which occurred at the Gumushane-Trabzon highway, NE Turkey, in May 2008.This type of landslides is plane failure. In this study, a reliability method was suggested. For this, the probabilistic analysis was performed at failure condition obtained by, the deterministic back analyses with a comparison with laboratory derived shear-box test and the probabilistic back analysis.

THE CHARACTERISTICS OF THE LANDSLIDE

The failure investigated was occurred October 2008 at the Trabzon-Gumushane road, NE Turkey (Figure 1). The slope on which the said landslides occurred has a height of 21 m. and an overall face angle of 380. The rocks masses outcropped in the evaluated slopes consist of mainly Turonian-Santonian dacite. The main discontinuities in the slightly weathered dacitic mass include three joint sets. Joint sets 1 and 2 are all sub-vertical, with strike of 280 °- 295 ° and 10° - 25°, dip direction of SW and NW, dip angle of about 88° - 89°, respectively. The type of the failure investigated is planer slide. The joint sets 3, the failure plane has strike of 287 ° and 28° dip direction. The surface of the joint sets 3 is slightly weathered and rough. The joint coefficient number (JRC) of the discolored surface is 16±4. The friction angle and cohesion of a discontinuity surface was determined in the laboratory using a direct shear box of the type. The laboratory tests were carried out at the Rock Mechanic Laboratories of the Department of Geological Engineering and Mining Engineering (KTU). The direct shear test was carried out on 16 block with discontinuity (Fig2a). In the Figure 2b, while the apparent friction angle above the normal stress of 300-400 kPa is found to be 23-39 degrees. Apparent cohesion at a normal stress level range of corresponds to 2-50 kPa. Mean and standard deviation of friction angle (ϕ) of the joint samples were, 31° and 4°, respectively, while the values of c were 26 and 13 kPa, respectively.

DETERMINISTIC BACK ANALYSIS

Back analyses of a failed slope deterministic method to find the shear strength parameters of the failure plane were performed by using equation 1. During back analysis by deterministic method, the safety factor (Fs) is accepted as equal to 1.0, and then equation 2 is derived from equation 1 for determination of cohesion during failure [7], [12], [13].

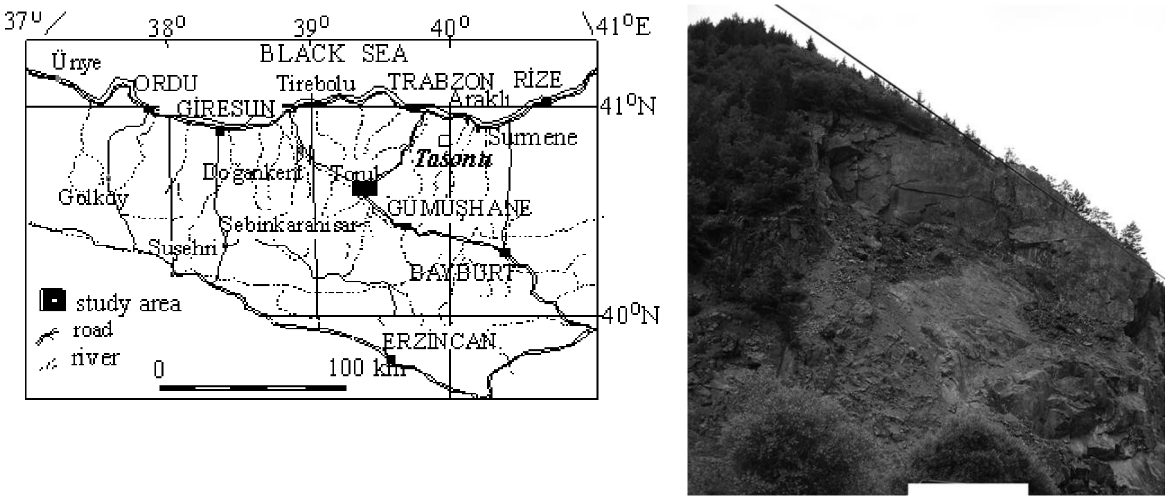


Figure.1. The location on which the investigated landslides occurred and the landslides

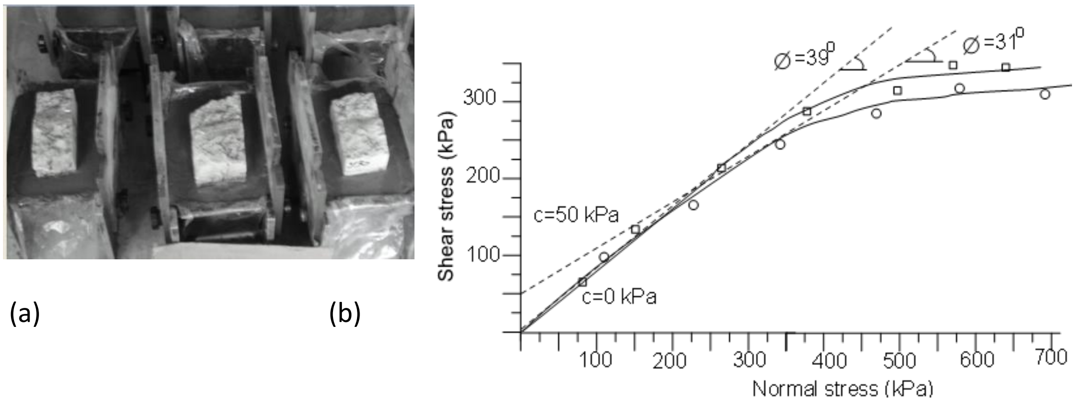


Figure 2. The block samples with discontinuity (a) and shear stress versus normal stress graph for joint 3

$$Fs = \frac{cL + [W\cos\alpha - U - V\sin\alpha]\tan\phi}{[W\sin\alpha + V\cos\alpha]} \tag{1}$$

$$c = \frac{[W\sin\alpha + V\cos\alpha] - [W\cos\alpha - U - V\sin\alpha]\tan\phi}{L} \tag{2}$$

$$W = A.\gamma \tag{3}$$

$$V = 0.5.\gamma_{w}.z_{w}^{2} \tag{4}$$

$$U = 0.5.L.\gamma_{w}.z_{w} \tag{5}$$

where α is dip angle of the failure plane (degree), c is cohesion (kPa), ϕ is friction angle (degree), γ is unit weight of the rock mass (kN/m^3), γw is unit weight of water (kN/m^3), L is the length of the failure plane (m), A is basal area of the block (m^2),W is the weights of sliding block (kN/m), k is the horizontal component of seismic coefficient caused by blasting, z_w is depth of the water in the tension crack (m), U is the water forces acting on the failure plane (kN/m) and V is the water forces acting in the tension crack (kN/m). The geometrical characteristics obtained from the cross section through the failed slopes, physical properties and weights of the rock mass blocks used in the analysis were given in Fig. 4a-b. The inclination of all failure planes were taken as 280^{o} as measured during the field studies. As the landslide occurred after a period of heavy rain falls, the variation of water pressure is most likely to be one of the main causes of the landslides. In Fig. 4b. the result of the deterministic back analysis and comparison of the analysis and the direct shear test were given. In these analyses, different value of percent filled tension crack was used.

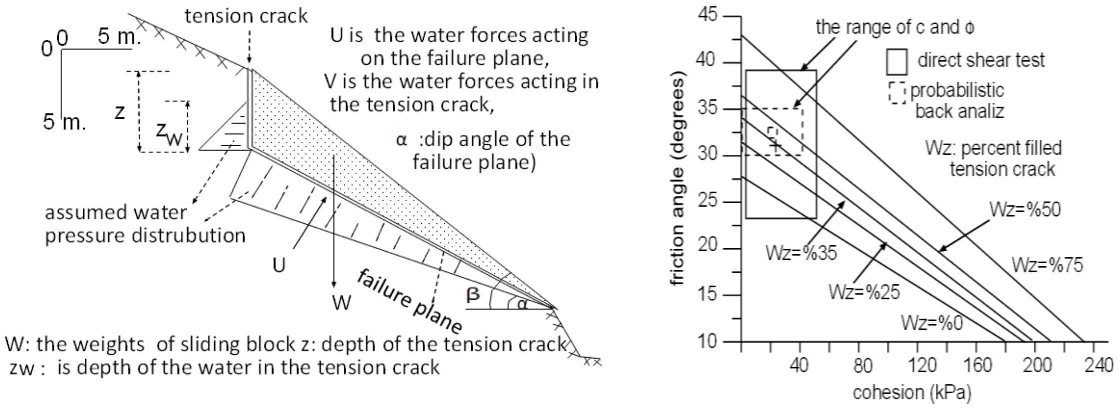


Fig. 4. The geometry of the failure slopes and limit equilibrium model deterministic back analysis and (a), the deterministic back analysis and comparison of the analysis and the direct shear test (b).

PROBABILISTIC BACK ANALYSIS

In the method proposed from Zhang et al. [3], let g(θ,r) denote a slope stability model (such as a model based on a limit equilibrium method), where θ is vector denoting uncertain input parameters and *r* is vector denoting input parameters without uncertainty. In other words *r* is dropped from the slope stability model for simplicity. [3]. The uncertain input parameters θ may include both soil strength parameters and pore-water pressure parameters. For simplicity, assume that the prior knowledge on θ can be described by a multivariate normal distribution with a mean of μ_θ and a covariance matrix of C_θ. The objective of probabilistic back-analysis is then to improve the probability distribution of θ based on observed slope failure information. To quantify the effect of model imperfection, the model uncertainty can be modeled as a random variable, which is defined as [3];

$$\varepsilon = y - g(\theta) \tag{6}$$

where *y is* actual factor of safety and ε is random variable characterizing the modeling uncertainty.

For simplicity, assume ε follows the normal distribution with a mean of μ_ε and a standard deviation of σ_ε. Let $\mu_{\theta/d}$ and $C_{\theta/d}$ denote the improved mean and covariance matrix of θ, respectively. As a multivariate normal distribution can be fully determined by its mean and covariance matrix, the task in the probabilistic back-analysis is then reduced to determining $\mu_{\theta/d}$ and $C_{\theta/d}$. For a general slope stability model $g(\theta)$, s a point that maximizes the chance to observe the slope failure event, and it denotes the most probable combination of parameters that had led to the slope failure event. μ_θ/d can be obtained by minimizing the following misfit function $2S(\theta)$ [3],

$$2S(\theta) = \frac{[g\ (\theta) + \mu_\varepsilon - 1]^T [g\ (\theta) + \mu_\varepsilon - 1]}{\sigma_\varepsilon^2} + (\theta - \mu_\theta)^T C_\theta^{-1} (\theta - \mu_\theta) \tag{7}$$

The improved covariance matrix of θ, $C_{\theta/d}$, which describes the magnitude of uncertainty in each component of θ as well as the dependence relationships among various components of θ, can be determined as follows [3];

$$C_{\theta/d} = \left(\frac{G^T G}{\sigma_\varepsilon^2} + C_\theta^{-1} \right)^{-1} \tag{8}$$

$$G = \left. \frac{\partial g(\theta)}{\partial \theta} \right|_{\theta = \mu_{\theta/d}} \tag{9}$$

where **G** is row vector representing the sensitivity of $g(\theta)$ with respect to θ at $\mu_{\theta/d}$.

When $g(\theta)$ is approximately linear, $\mu_{\theta/d}$ and $C_{\theta/d}$ can be determined analytically with the following equations without resorting to the minimization procedure [3] ;

$$\mu_{\theta/d} = \mu_\theta + C_\theta H^T (H C_\theta H^T + \sigma^2)^{-1} [1 - g(\mu_\theta) - \mu_\theta] \tag{10}$$

$$C_{\theta/d} = \left(\frac{H^T H}{\sigma_\varepsilon^2} + C_\theta^{-1} \right)^{-1} \tag{11}$$

$$H = \left. \frac{\partial g(\theta)}{\partial \theta} \right|_{\theta = \mu_\theta} \tag{12}$$

where $g(\mu_\theta)$ is predicted factor of safety calculated at point μ_θ and **H** isrow vector representing the sensitivity of $g(\theta)$ with respect to θ point μ_θ

The probabilistic back-analysis above was implemented in three steps as given by a) Select a stability model, $g(\theta)$ and identify the uncertain parameters θ; b) Quantify the knowledge on θ prior to the back-analysis and on the model uncertainty of $g(\theta)$ [3]. This step is in fact a process of determining μ_θ, $\mathbf{C}_\theta$, μ_ε and $\boldsymbol{\sigma}_\varepsilon$; and c) Improve the probability distribution of θ considering the slope failure event. In this step, $\mu_{\theta/d}$ and $C_{\theta/d}$, which contain the improved knowledge on θ, are calculated using the formulas presented in the previous section. The back-analysis was carried out in two ways: using c and ϕ as input parameters (Table 1). In the analysis, the Excel worksheet given in [3] was used.

Table 1. Probabilistic back analysis of the landslides investigated

c	ø	γ	α	zw	F1	F2				
20	33	27	28	1,75	72831	72910				
			Fs=	0,999						
Prior	information									
	μ_θ	σ_θ		C_θ						
c	26	13		169	0					
ø	31	4		0	16					
Misit function										
			Observed	Fs		$\theta-\mu$		error1	error2	2S(θ)
μ_ε	σ_ε		1			-6		0,0005	0,463	0,46
0	0,05					2				
Posterior	mean	and	covartance							
	$\mu_{\theta/d}$			G			G^T			
c	20			0,005	0,062		0,005			
ø	33						0,062			
	$\sigma_{\theta/d}$			$C_{\theta/d}$			$\rho_{c\phi}$			
c	12,59			158,5	-12,3		-0,78			
ø	1,255			-12,3	1,575					

the resisting force:

$$F1=\left[W\cos\alpha-U-V\sin\alpha\right]\tan\phi$$

driving force:

$$F2=\left[W\sin\alpha+V\cos\alpha\right]$$

In order to determine of the probabilistic back analysis, the probabilistic analysis is performed for the slopes under interest at the failure condition obtained by deterministic back analysis and the probabilistic back analysis. Monte Carlo simulation method was used to probability distribution of the safety factor and probability of failure (Fig. 6,7). The facts that the standard deviation of safety factor (σ_F) is small, and probably of failure (Pf) is close to 50% show that there is small uncertainty in the limit equilibrium conditions.

CONCLUSION AND RESULT

In this study, probabilistic back analysis was performed for the landslide occurred October 2008 at the Trabzon-Gumushane road, NE Turkey. The type of the failure investigated is planer slide. The main discontinuities in the slightly weathered dacitic mass in which landslides developed include three joint sets. According to the result of direct shear tests, the values of the friction angle of the failure plane vary between 23° and 37°, while the values of cohesion (c) of the samples vary between 2 and 50 kPa. The mean value of these c and ø obtained from direct shear tests is 26 kPa and 31 degree, respectively. Assuming that these c and ø values display normal distribution, standard derivation values are determined as 13 kPa and 4 by means of the Monte Carlo method (prior information). The back analyses with deterministic method for the landslides investigated resulted in c and ø values satisfying Fs=1. T In these analyses, the range per cent filled tension cracks was taken as 0-75%. Considering the mean values of the c and ø, the per cent filled tension cracks value was in range 25-50% at the failure. The mean value of the per cent filled tension cracks was found as 35%, in the deterministic back analyses (prior information). Considering the prior information obtained from the deterministic back analyses and direct shear tests, the probabilistic back analyses for the investigated landslide. In the back analysis carried out using the prior information obtained from the deterministic back analyses and direct shear tests , the posterior mean value and standard derivation value of c were 20 kPa and 12.6 kPa, respectively

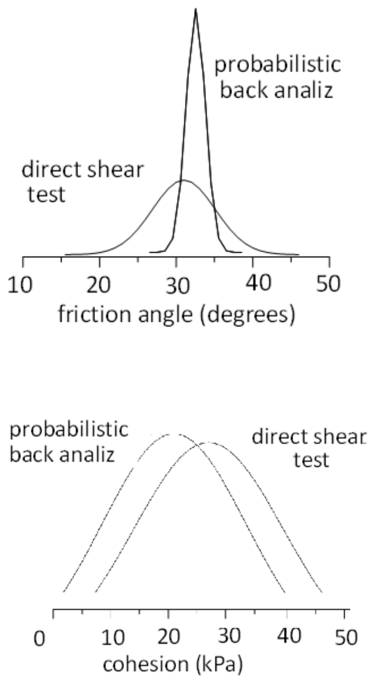
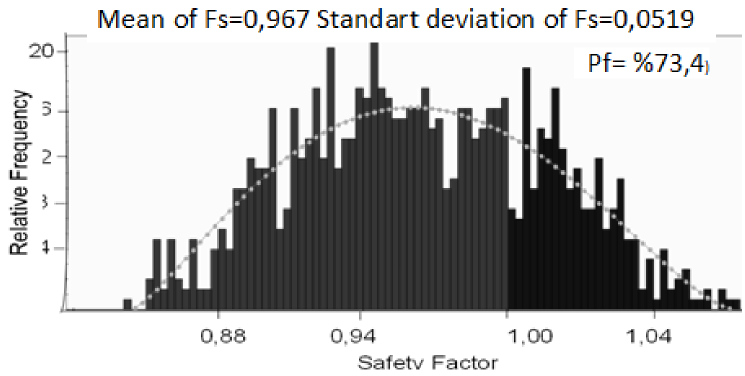
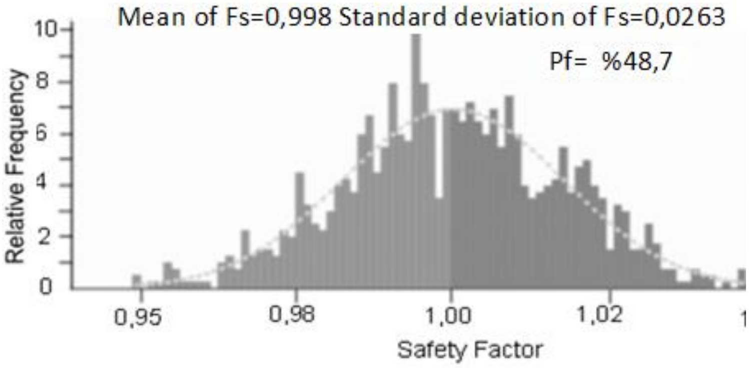


Figure 5. Comparison of prior information and posterior value in the probabilistic back analysis



The value of strength parameter from direct shear test



The value of strength parameter from probabilistic back analysis

Figure 6. Results of the probabilistic stability analysis for the failure conditions

In these analyses, the posterior mean value and standard deviation value of ø for the said prior information are 33˚ and 1.25 ˚. Two probabilistic analysis is performed for the slopes under interest at the failure condition. c, ø and the percent filled tension cracks value using in the first analysis, were obtained by the deterministic back analyses and direct shear test. In this analysis, mean and standard deviation of safety factor and probability failure value were found 0,967, 0,0519 and 73,4%, respectively. In the second probabilistic analysis at the failure condition, c, ø and the per cent filled tension cracks value using as input parameters was obtained from probabilistic back analyses. In the second analysis, mean, standard deviation of safety factor and probability failure value were found 0,998, 0,0263 and 48.7%, respectively. The standard deviation of safety facto) is small, and probably of failure is close to 50% show that there is small uncertainty in the limit equilibrium conditions. Taking into consideration this condition, it can be said that, the result of the probabilistic back analyses is a more representative according to the deterministic back analyses for at the failure condition of the landslide investigated. In addition, the values of cohesion and friction angle from the comparison of direct shear tests and deterministic back analysis as given in this study can be considered as input parameters in the probabilistic back analysis.

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