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Probabilistic Back Analysis of Landslides in Arakli-Tasonu Quarry, Northeast Turkey

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Abstract

This study evaluates the landslides that occurred on 3 October 2005, 20 March 2006, and 19 October 2006 in Tasonu Limestone Quarry, Trabzon, and Northeast Turkey. The failure plane of the three landslides occurred on the same clay layer. Back analyses using probabilistic technique were carried out in order to assess the failure mechanism, determine the range of shear strength mobilized in the failure plane, estimate the groundwater condition at the time of failure and evaluate the influence of blasting. The values of shear strength parameters c and ϕ obtained by direct shear tests were considered as prior information about the input parameters. It was determined that the results of the probabilistic back analysis could be exploited in evaluating the formation conditions of previous landslides and the potential for future landslides in that area.

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Keywords: rock slope; clay layer; direct shear tests; back analysis; probabilistic method.

1. Introduction

In the back analysis of a slope failure, if the general shape of a sliding surface and the volume of the failed mass are known, the shear strength parameters of a sliding surface can be estimated. Limit equilibrium techniques are commonly adopted methods due to their simplicity for structurally controlled slopes (Sharifzadeh et al. 2010). In this study, the probabilistic method suggested by Zhang et al. (2010a) was used in the back analysis of the slope failures in Araklı-Tasonü Limestone Quarry in Trabzon, NE Turkey (Figure 1). This limestone quarry provides approximately 80% of the raw material needed for the cement plant of Trabzon Cement Factory, which is the region's most important economic enterprise. The material in the quarry was obtained using an uncontrolled blasting technique for approximately 10 years.

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Three landslides occurred on 3 October 2005 (Landslide 1), 20 March 2006 (Landslide 2), and 19 October 2006 (Landslide 3) in the quarry following heavy rainfall. Kesimal et al. (2008) stated that the first plane failure had been strongly influenced by the acceleration of uncontrolled blasting operations, in addition to the heavy rainfall. Meteorological records for the last 30 years reveal that the area has received a mean monthly rainfall of 72 mm. The minimum value was 35 mm, measured in July, and the maximum value was 120 mm, recorded in October. The heaviest rainfall occurs between October and January, with a monthly average of 94 mm (Ceryan 2009).

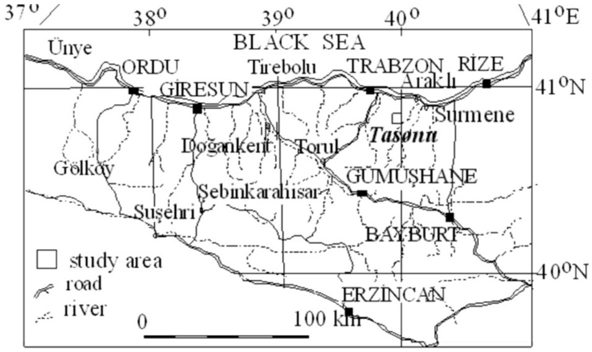


Figure 1. Location map of the study area.

After the landslides, the annual production has decreased significantly. In addition, new tension cracks on the slopes occurred producing the risk of new landslides. Therefore, the back analyses using probabilistic limit equilibrium technique were carried out to assess the failure mechanism, determine the range of the shear strength mobilized in the failure plane, estimate the groundwater condition at the time of failure, and evaluate the influence of blasting, back analysis using probabilistic limit equilibrium technique were carried out. Then probabilistic back-analysis methods were compared based on the results of the laboratory experiments and field measurements. Then, the applicability of the probabilistic back analysis methods of Zhang et al. (2010a) was investigated. In order to perform the back analysis, probabilistic stability analysis of the slope at the failure condition obtained by the probabilistic back analysis were performed.

2. Description of the landslides' area and mechanism of the landslides

The bench faces of slopes failure have clayey levels, which are inclined in the same direction as the quarry slopes (Figure 2a). In April 2005, these layers of weakness appeared on the slope excavation, and tensile cracks 38–91 meters away from the excavation faces developed on the northern side of the quarry (Figure 2b). After this date, following the results of Kesimal et al. (2008), the production at the quarry continued using blasting. However, excavation by blasting had the least impact in terms of slope stability. After the cracks were noticed, they were marked on the topographic map and groundwater level in the cracks and drillings was measured after heavy rains. The mentioned landslides occurred over the failure plane in the clay layer.

Figure 2 (a) Excavation slopes where landslides occurred and previously developed tension cracks (b).

On 3 October 2005, a large rock—about 52 m high, 57 m wide, and with a length along the strike of 50 m—slid a distance of approximately 25 m along the red-white colour clayey layer dipping 14° into the quarry, following exposure of the sliding plane by blasting at the foot of the slope (Landslide 1, Figure 3a). Then, on 20 March 2006, a large rock—about 46 m high, 90 m wide, and with a length along the strike of 158.2 m—slid a distance of approximately 25 m along the clay layer dipping 14° into the quarry, following exposure of the sliding plane by bench blasting at the foot of the slope (Landslide 2, Figure 3c). Finally, on 19 October 2006, a large rock—about 48 m high, 77 m wide, and with a length along the strike of 138.3 m slid a distance of approximately 25 m along the clay layer dipping 14° into the quarry, following exposure of the sliding plane by blasting at the foot of the slope (Landslide 3, Figure 2b). These failures were developed on the same the clay layer (Figure 3a-c and Figure 4). The inclination of the clay layer, which is the failure plane, was measured at 14°. This red-white colour layer was formed by weathering of tuffit. The thickness of the clay level ranges from 30 to 115 cm. The surfaces of the limestone layers above and below the sliding surface were planar with a slight roughness. Since the thickness of the clay layer was greater than the amplitude of roughness of the bedding plane, the sliding plane passed through the layer of clay. Index properties, mineralogical characteristics, shear-strength properties of the clay layer and the failure plane of the landslides were determined in the laboratory using samples taken from the clay layer in the landslide area.

Figure 3. The landslides occurred on 3 October 2005 (a), 20 March 2006 (c) and 19 October 2006 (b) in Araklı- Tasonu Limestone Quarry.

3. Back analysis of the slope failures

One of the most important triggering mechanisms of the planar shear failures was the uncontrolled blasting operations (excessive explosive charge weight per delay for blast holes) (Ercikti et. al. 2004; Kesimal et al. 2005). Therefore, the effect of blast-induced acceleration was taken into consideration for the back analysis. Pre-failure geometry of the investigated slopes was estimated from photographs and the topographical map produced in 2004 before the landslides occurred (Ceryan 2009).

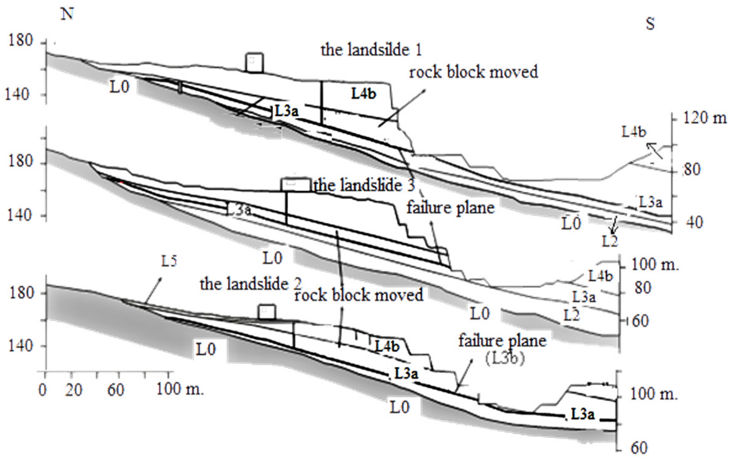


Figure 4. The cross section through the failed slopes (L0: basalt, andesite and their pyroclastic, L2: Red tuff alternates with white limestone, L3a: Common macro shelly karstic voided limestone, L4b: red sandy clayey limestone, L5: clayey limestone and marl alternates with sandy limestone (Ceryan 2009).

As the landslide occurred after a period of heavy rainfalls, the variation of water pressure is most likely to be one of the main causes of the landslides. Immediately after the development of the tension cracks behind the slopes, the excavations continued by blasting with minimal seismic effect. According to the blast-induced horizontal acceleration values obtained from 73 shots, the average value was considered as 0.013 g on the slopes (Kesimal et al. 2005). Groundwater levels measured in the tension cracks were approximately parallel to the layering and ranged from 135 to 138.5 m, 144.5 to 147 m, and 144.5 to 147 m, respectively, before Landslides 1, 2, and 3.

The probabilistic back analysis of landslides in Tasonu quarry was made by the method proposed from Zhang et al (2010a). In this method, let g (θ, r) denote a slope stability model (such as a model based on a limit equilibrium method), where θ is a 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 (Zhang et al 2010a). 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 the observed slope failure information. To quantify the effect of model imperfection, the uncertainty can be modelled as a random variable, which is defined as (Zhang et al 2010a).

$$\epsilon = y - g(\theta), \tag{1}$$

where y is an actual factor of safety and ε is a random variable characterizing the modelling uncertainty. For simplicity, assume ε follows the normal distribution with a mean of μ_ε and a standard deviation of $\boldsymbol{\sigma}_\varepsilon$.
Let $\mu_{\theta/d}$ and $C_{\theta/d}$ denote the improved mean and covariance matrix of θ, respectively. Since 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(θ), there is 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(θ) (Zhang et al 2010a)

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

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 (Zhang et al 2010a):

$$C_{\theta/d} = \left(\frac{G^T G}{\sigma_\varepsilon^2} + C_\theta^{-1} \right)^{-1} \tag{3}$$

$$G = \frac{\partial g(\theta)}{\partial \theta} \Big|_{\theta = \mu_{\theta/d}} \tag{4}$$

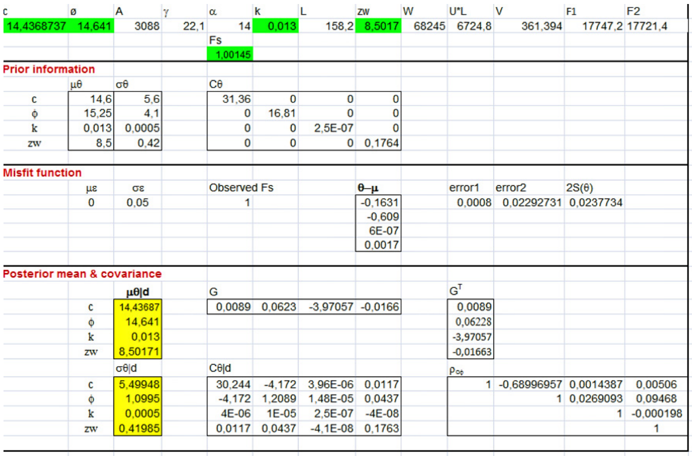
where **G** is row vector representing the sensitivity of g(θ) with respect to θ at $\mu_{\theta/d}$.
The back-analysis of the Tasonu landslides was implemented in three steps (Zhang et al. 2010a).
1. Select a stability model, g(θ) and identify the uncertain parameters θ;
2. Quantify the knowledge on θ prior to the back-analysis and on the model uncertainty of g**(**θ**)**. This step is in fact a process of determining μ_θ, $\mathbf{C}_\theta$, μ_ε and $\boldsymbol{\sigma}_\varepsilon$; and
3. 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 of the Arakli-Tasonu landslides based on the equations 1-4 was carried out in two way of using:

a) k, z_w, c and ϕ,

b) c and ϕ as input parameters, (Fig. 5).

In the analysis, the Excel worksheet given in Zhang et al (2010a) was used. In the header part (Fig.5); parameter F_1 is the resisting force and F_2 is the driving force (5)

$$F_1 = \left[W(\cos\alpha - k\sin\alpha) - U - V\sin\alpha\right]\tan\phi \text{ and } F_2 = \left[W(\sin\alpha + k\cos\alpha) + V\cos\alpha\right] \tag{5}$$



c	ø	A	γ	α	k	L	zw	W	U*L	V	F1	F2
14,4368737	14,641	3088	22,1	14	0,013	158,2	8,5017	68245	6724,8	361,394	17747,2	17721,4
				Fs								
				1,00145								
Prior information												
	μθ	σθ		Cθ								
c	14,6	5,6		31,36	0	0	0					
ϕ	15,25	4,1		0	16,81	0	0					
k	0,013	0,0005		0	0	2,5E-07	0					
zw	8,5	0,42		0	0	0	0,1764					
Misfit function												
	με	σε		Observed Fs			θ–μ		error1	error2	2S(θ)	
	0	0,05		1			-0,1631		0,0008	0,02292731	0,0237734	
							-0,609					
							6E-07					
							0,0017					
Posterior mean & covariance												
		μθ|d		G					G^T			
	c	14,43687		0,0089	0,0623	-3,97057	-0,0166		0,0089			
	ϕ	14,641							0,06228			
	k	0,013							-3,97057			
	zw	8,50171							-0,01663			
		σθ|d		Cθ|d					$\rho_{c\phi}$			
	c	5,49948		30,244	-4,172	3,96E-06	0,0117		1	-0,68996957	0,0014387	0,00506
	ϕ	1,0995		-4,172	1,2089	1,48E-05	0,0437			1	0,0269093	0,09468
	k	0,0005		4E-06	1E-05	2,5E-07	-4E-08				1	-0,000198
	zw	0,41985		0,0117	0,0437	-4,1E-08	0,1763					1

Figure 5. Input parameters.

On Fig. 5 is the probabilistic back analysis of landslides for Araklı-Tasonu with input parameters k, z_w, c and input parameters ϕ; c. In equation (5) parameters are:

- c -cohesion in (kPa),
- ø - friction angle (degrees),
- γ - unit weight of the rock mass in (kN/m^3).
- A - is basal area of the block (m^2),
- α - is dip angle of the failure plane,
- k - is the horizontal component of seismic coefficient caused by blasting,
- W- is the weights of sliding block (kN/m),
- z - depth of the tension crack (m),
- z_w - is depth of the water in the tension crack,
- U - is the water forces acting on the failure plane (kN/m^2),
- V - is the water forces acting in the tension crack (kN/m).

Two probabilistic back analyses were carried out in the present study. In the first analysis, c and ϕ were taken as input parameters when the mean values of k and z_w were considered (Table 1). In the second probabilistic back analysis, k, z_w c and ϕ, and (b) c and ϕ were considered as input parameters (Table 2). The prior information which are mean (μ_θ), standard deviation (σ_θ) and variation coefficient defined as (σ_θ/μ_θ)x100 and posterior value of these parameters, $\mu_{\theta|d}$, $\sigma_{\theta|d}$, and v_θ for uncertain input parameters shown as θ were compared in the Table 1 and Table 2.

Table 1. The result of probabilistic back-analysis (input parameters are c and ϕ)

RI*				RII						RIII		
				Landslide 1		Landslide 2		Landslide 3				
θ	μ_θ	σ_θ	v_θ	$\mu_{\theta\|d}$	$\sigma_{\theta\ d}$	$\mu_{\theta\|d}$	$\sigma_{\theta\|}$	$\mu_{\theta\|}$	$\sigma_{\theta\|d}$	$\mu_{\theta\|d}$	$\sigma_{\theta\|d}$	$v_{\theta/d}$
c	14.6	5.6	38.3	14.49	5.572	14.437	5.499	14.73	5.544	14.47	5.61	38.8
ϕ	15.25	4.1	26.9	14.45	0.837	14.64	1.094	14.604	0.947	14.57	1.024	7.0

(RI*:the values were obtained with the results of shear strength test for the clay layer, RII: the values were obtained with the probabilistic back-analysis of the each landside, RIII: the values were obtained by RII for the clay layer formed the failure plane).

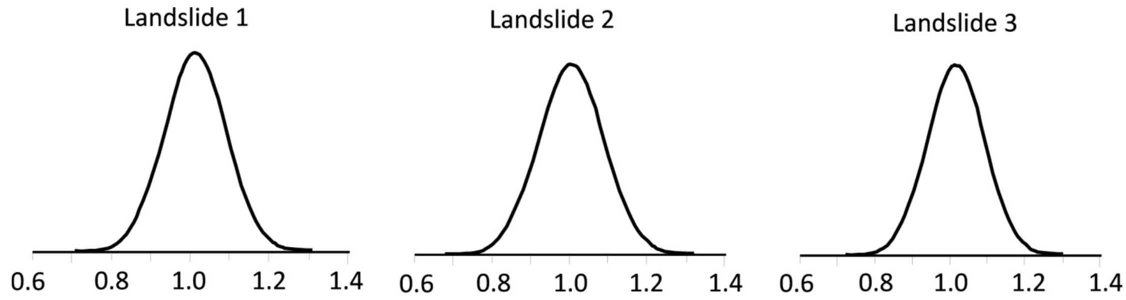
Table 2. The result of probabilistic back-analysis (input parameters are c, ϕ, z_w and k).

	RII			RII						RIII		
				Landslide 1		Landslide 2		Landslide 3				
θ	μ_θ	σ_θ	V_θ	$\mu_{\theta\|d}$	$\sigma_{\theta\|d}$	$\mu_{\theta\|d}$	$\sigma_{\theta\|d}$	$\mu_{\theta\|d}$	$\sigma_{\theta\|d}$	$\mu_{\theta\|d}$	$\sigma_{\theta\|d}$	$V_{\theta/d}$
c	14.6	5.6	38.4	14.488	5.572	14.437	5.499	14.473	5.545	14.49	5.608	39
ϕ	15.25	4.1	26.9	14.481	0.840	14.641	1.099	14.604	0.951	14.72	1.042	7.1
k	0,013	0.0005	3.9	0.013	0.0005	0.013	0.0005	0.013	0.0005			
z_w	6.0	0.42	7.0	6.013	0.4199							
	8.5	0.42	4.9			8.501	0.4199					
	8.0	0.42	5.3					8.04	0.4199			

The probabilistic analysis is performed for the slopes under interest at the failure condition obtained by the probabilistic back analysis. Monte Carlo simulation method was used for the probability distribution of the safety factor and the probability of failure (Figure 6). The shear strength parameters in these analyses were obtained from the results of the probabilistic back analysis.

4. Result and conclusion

In this study, the back analysis was performed for the three landsides occurred on 3 October 2005 (Landslide 1), 20 March 2006 (Landslide 2), and 19 October 2006 (Landslide 3) at the Tasonu quary in Trabzon, NE Turkey. The sliding plane of the landslides passed through the same clay layer. The samples from the clay layer were investigated and determined to be the CH group: high-plasticity sandy clay – high-plasticity fat clay. They consisted of 77% smectite and 9% illite (Ceryan 2009). The values of the internal friction angle (ϕ) of the samples obtained by the consolidated-drained direct shear tests varied between 9° and 20°, while the values of cohesion (c) of the samples varied between 9 and 24 kPa. The mean and the standard deviation of ϕ of the clay were, respectively, 15.3° and 4.1°, while the values of c were 14.6 and 5.6 kPa, respectively. The coefficient of variation ϕ was 26.9, while the value of c was 38.4. Two probabilistic back analyses were carried out in the present study. In the first analysis, c and ϕ were taken as input parameters when the mean values of k and z_w were considered. In the second probabilistic back analysis, k, z_w c and ϕ, and (b) c and ϕ were considered as input parameters. In these analyses, the depth of water in the tension crack values (z_w) for Landslides 1, 2, and 3 was considered as 6, 8, and 8.5 m, respectively, and the horizontal component of the seismic coefficient caused by blasting (k) was 0.013.There were minor differences between the results using the two probabilistic back analyses (Tables 1 and 2). In the second probabilistic back analysis, there were no differences between posterior and prior information about k and z_w values.



1:μ_F=1.01, σ_F=0.078, Pf %=44 μ_F=1.006, σ_F=0.081, Pf %=43 μ_F=1.049, σ_F=0.077, Pf %=32

Figure 6. Results of the probabilistic stability analysis performed for the failure conditions obtained in the deterministic back analysis.

In the second probabilistic back analysis carried out for landslide 2, the posterior mean value and standard derivation value of c of the failure plane are 14.437 kPa and 5.499 kPa, respectively. In this analysis, the posterior mean value and the standard deviation value of ϕ for the failure plane are 14.72˚ and 1.04˚. According to the result of the second probabilistic back analysis, the variation coefficient of c of the clay layer, failure plane, found using probabilistic back analysis was approximately equal to the value of c obtained using the shear test. The difference between the mean value of ϕ obtained by the direct shear test and the mean value of ϕ found by probabilistic back analysis is approximately equal to 1°. However, the variation coefficient of ϕ obtained in the laboratory tests was 26.9% while the value of ϕ was 7.1% using probabilistic back analysis. The probabilistic analyses were performed at the failure condition obtained by the probabilistic back analysis for the said failure slopes (confusing).

By these analyses mean of the safety factor values were obtained as 1.01, 1.006 and 1.049 for the Landside 1-3 respectively, while standard deviations of the safety factor were obtained as 0.078, 0.081 and 0.077. The probability (correct it in the article) of failure (Pf) values for Landslides1-3 were obtained 0.44, 0.43 and 0.32. The Mean of the safety factor approaches *(*μ_F) unity and the reliability index gets closer to zero in the probabilistic back analysis optimization. The fact that the standard deviation of the *safety factor (*σ_F*)* is small and probably of failure (Pf) close to 50% show that there is a small uncertainty in the limit equilibrium conditions. Considering these conditions and the results of the probabilistic analysis for the failure conditions of the mentioned failure slopes, it can be said that the values of cohesion and friction angle obtained by the probabilistic back analysis express the mobilized shear strength.

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