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ABSTRACT

This paper focuses on the description, analysis and stabilization of a failed cut slope in schist. The slope is located near the top of a hill and was cut for the needs of a new industrial building. A few weeks after the excavation, a slide occurred along the schistosity plane of the slope. The slide was attributed to the effect of water which flooded the slope following an overflow of a water tank located a few meters above the slope crest. For the analysis both the deterministic and probabilistic approaches were carried out, with the input parameters determined from simple in-situ and laboratory tests and also from back-analysis. The results showed that the probabilistic approach offers significant advantages, providing a better feeling of the effect of the uncertainty and variability of the input parameters and in this case a more economical solution, given that a risk of failure equal to 2.25% is acceptable.

INTRODUCTION

The effect of water is one of the main parameters controlling the stability of rock slopes. Numerous rock slides attributed to the effect of hydrostatic forces have been reported in the literature. In this paper, the failure of a cut slope in schist is analyzed and the stabilization measures applied are presented. The slope is located in an area NE of the city of Thessaloniki, Greece, near the top of a hill, and was cut for the needs of a new building designed to accommodate the necessary equipment of a new gas metering and regulating station. The excavated slope had a mean inclination of 3:1 (vertical:horizontal), an average height of 12 m and a length of 70 m. A few weeks after the excavation a slide occurred along the plane of schistosity, in the middle part of the slope. The slide took place in mid August, the most dry period of the year, following a flooding of the slope caused by the overflow of a water tank existing a few meters above the crest. The tank had been built with the aim to cover the drinking water needs of a nearby village.

The study area is located within a zone of metamorphic formation, consisting mainly of dark grey fyllites and talc schists with quartz and carbonate veins. The major rock type in the area is a grayish black graphitic phyllite, characterized by an almost perfect foliaton. A large outcrop of light grayish brown talc schist with very well-defined schistosity planes exists in the central part of the slope, where the slide occurred.

The area has suffered significant damage due to strong earthquakes several times in the past, the most severe being in 1759 (M=6.5) when the majority of the inhabitants abandoned the city for about two years (Papazachos and Papazachou, 2003). The 1978 Thessaloniki earthquake (M=6.5) was the latest destructive one, causing the collapse of buildings and loss of lives in the city and nearby villages.

Due to the morphology of the area of interest, no groundwater flow is anticipated. Small quantities of rainfall infiltrated within the rock mass are not expected to lead to the development of serious hydrostatic pressures within the rock mass. However, since the slide was attributed to the water overflow from the adjacent water tank, a similar event in the future, can not be ignored and full action of water must be included in the analysis. However, the distance of the tank from the slope crest was large enough to avoid any loading of the slope, due to the weight of the tank and its content.

DATA COLLECTION AND INTERPRETATION

Field measurements including dip direction and dip angle of schistosity planes, faults and joints, were made in order to determine critical structural characteristics for the slope stability. The schistosity surfaces are smooth and planar, medium to close spaced. The statistical elaboration of the collected tectonic data was performed using DIPS and followed the procedure described by Hoek and Bray (1981). The analysis showed that sliding conditions are fulfilled along the plane of schistosity in the central part of the excavated slope (Fig. 1). The mean dip angle of the plane of schistosity (slide plane) is equal to 54° (range 45°- 60°) and the dip direction equal to 10° with a range between 0° and 15°. In the most critical section where the slide occurred, the orientation of the slope face was 005°/72° (dip direction/dip angle). The overall slope height at face H_1 was 12.0 m, the upper slope angle equal to 10°, and the resulted maximum slope height H_2 equal to 12.7 m (Fig. 2).

MECHANICAL PROPERTIES

Laboratory tests were carried out in order to determine the physical and mechanical properties of the rock material involved in the slide (talc schist). The dry unit weight was found to be 25.63 kN/m^3, the wet unit weight 26.19 kN/m^3, the Schmidt hammer number 30.3 and the unconfined compressive strength (estimated) equal to 30 MPa.

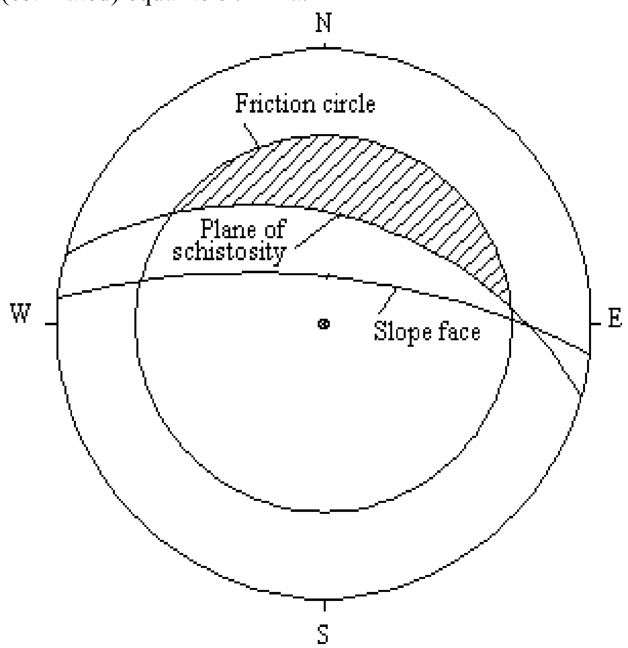


Fig. 1. Schmidt diagram

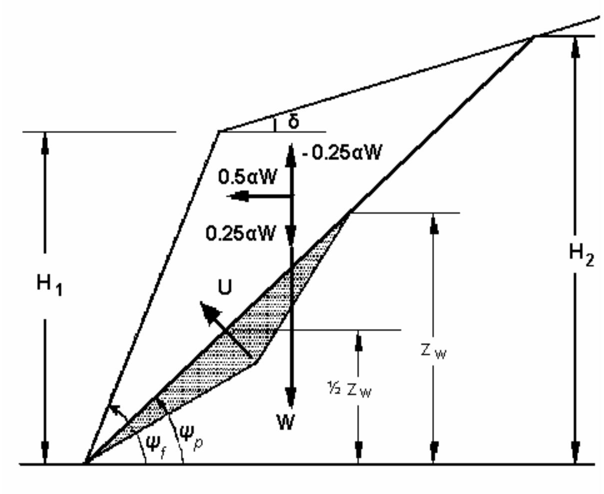


Fig. 2. Geometry and forces acting on slope

A series of direct shear tests was conducted on 100x100x60 mm jointed samples obtained from the plane of sliding in order to determine the shear strength parameters along the failure plane for both dry and wet conditions. The normal stress applied ranged between 100 and 500 kN/m^2.

The peak shear strength criterion used for the analysis of the experimental results is given by the following expression (Papaliangas *et al*, 1995):

$$\tau_p = \sigma_n \tan(\varphi_m + \psi) \qquad (1)$$

where

τ_p is the peak shear stress

σ_n is the normal stress

φ_p the peak friction angle of the rock joint

φ_m the friction angle of the rock wall material under high normal stress and

ψ the instantaneous dilation corresponding to the peak shear strength.

The friction angle φ_m is generally different from the "basic friction angle" (Barton and Choubey, 1971), and its relevance to the field shear strength of rock surfaces has been demonstrated elsewhere (Papaliangas *et al*, 1996, 1997). The non-dilational component of shear strength is for an effectively planar yet naturally textured surface and, for design, it can be used with a low shear strength factor of safety, as a lower bound (Hencher, 1995).

For each shear test the measured peak shear strength was analyzed in two components: a) The dilational (geometrical) component, which arises from overriding of asperities at an angle determined by the slope of the asperities. b) The non-dilational component, which arises from the shearing resistance of rock contacts. Strong experimental evidence suggests that the magnitude of the true stresses acting in these contacts, is of the same order as that existing in the intact rock material, under conditions of brittle-ductile transition, therefore causing plastic deformation of these contacts (Papaliangas *et al*, 1995). Measurements of true stresses in direct shear tests of rough joints (Power, 1996) support this theory. Continuous measurements of direct shear load and shear and normal displacements were recorded. Representative shear stress –shear displacement, normal displacement-shear displacement and shear stress-normal stress diagrams are shown in Fig. 3a-c. The resulted values of friction angle ranged between 21.5° and 23.7°, with the lower values corresponding to wet conditions.

These relatively low values of friction angle are due to the presence of talc in the mineralogical composition of schist and are consistent with published experimental results on similar rock types (e.g. Einstein and Dowding, 1989). The failure surface is smooth and planar, therefore a small nominal average value of the dilation angle (ψ) equal to 0.5° is taken into account.

The cohesion of the planes of schistocity is generally difficult to determine without carrying out laboratory or in-situ tests. In this case, it was determined by back-analysis, as a function of the mobilized friction angle φ_m. For a given value of φ_m, the value of cohesion resulting in a safety factor equal to unity was determined, for a loading case including the effect of water, but

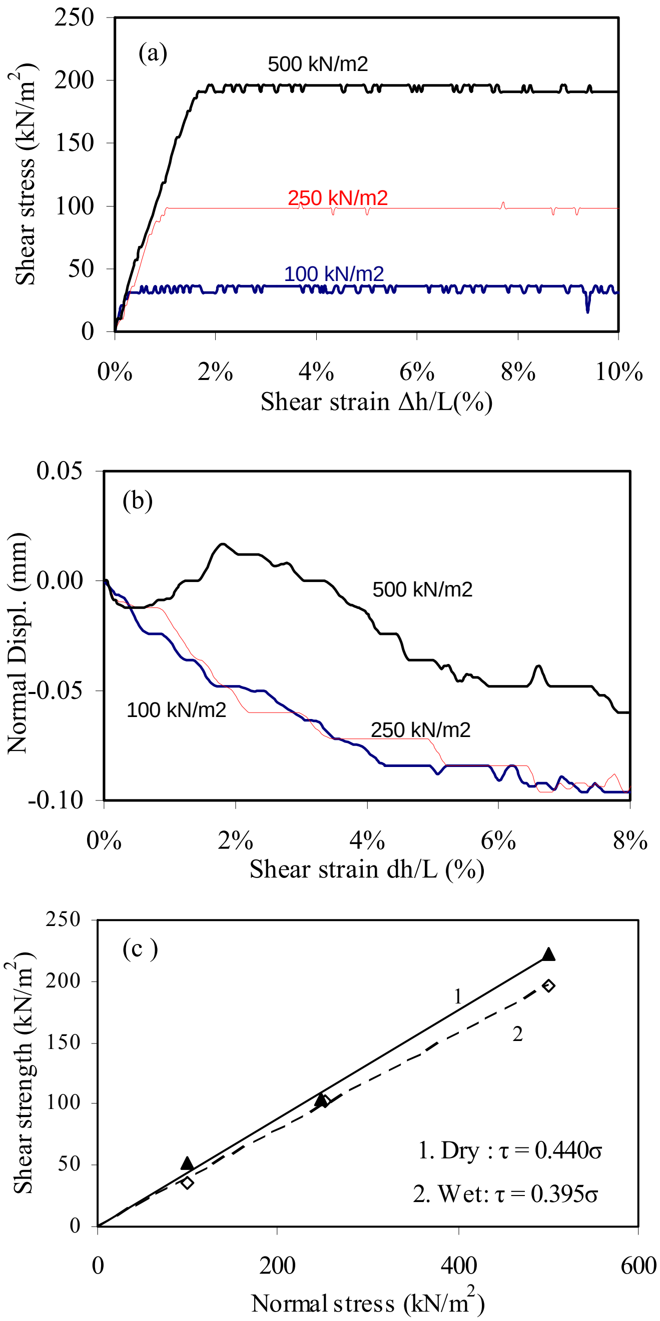


Fig. 3. Direct shear test results on planes of schistosity.

not that of the earthquake. It is estimated that this loading case corresponds to the existing conditions when the slide occurred. The normal stress acting on the failure plane was considered to be constant for all loading cases. For the geometry of Fig. 2, the value of cohesion c is given by the equation:

$$c = \frac{W \sin\psi_p - (W\cos\psi_p - U)\tan(\phi_m + \psi)}{A} \tag{2}$$

The water pressure acting on the plane of failure is assumed to have the idealized triangular distribution shown in Fig. 2 (Hoek and Bray, 2001), with a maximum pressure corresponding to the hydrostatic at a height equal to $0.50H_2$. This situation represents water that enters freely at the top of the slope and fully drains at the crest. For any height of water z_w, the water pressure is assumed to be equal to

$$u = \frac{1}{2}\gamma_w z_w \tag{3}$$

and the resultant water force

$$U = \frac{1}{2}\gamma_w z_w^2 / \sin\psi_p \tag{4}$$

In Fig. 4 the range of cohesion resulting from back analysis for a friction angle between 18 and 24 degrees is shown, using the following geometrical data:

Overall slope height	$H_1 = 12$ m
Overall slope angle	$\psi_f = 72°$
Failure plane dip angle	$\psi_p = 54°$
Upper slope inclination	$\delta = 10°$
Slope height	$H_2 = 12.7$ m
Unit weight of rock	$\gamma_r = 26$ kN/m^3
Unit weight of water	$\gamma_w = 9.81$ kN/m^3
Dilation angle	$\psi=0.5°$
Depth of water	$z_w/H_1=0\text{-}1.0$

For a wet slope the friction angle φ_m is taken equal to 21°, whereas the resulting cohesion c for a saturated slope with $z/H_1=1.0$, is equal to 41.5 kN/m^2, and without water pressure ($z/H_1=0.0$) equal to 29.3 kN/m^2. In the latter case the water force acting on the plane of sliding becomes approximately equal to the normal component of the weight of the slope, and the sliding force equal to the cohesive force cA.

STABILITY ANALYSIS

Deterministic analysis

The deterministic analysis was carried out using the geometry and the acting forces shown in Fig. 2. The average values selected for the input parameters at sliding conditions are: cohesion c=35 kPa, friction angle φ_m=21°, which correspond to the approximate centre of the assumed range of shear strength parameters illustrated in Fig. 4, dilation angle ψ=0.5° and depth of water z_w/H_1=0.40. The water force acting on the plane of failure is given by equation (4).

According to the Greek Code for Earthquake Resistant Design (Ministry of Environment, Physical Planning and Public Works, 2000) the ratio of horizontal earthquake acceleration to gravitational acceleration α, for the specific area, is equal to 0.16. The maximum earthquake force is analysed into two components:

Horizontal component : $E_h=0.50\alpha W$
Vertical component : $E_v=\pm0.25\alpha W$

The factor of safety (ratio of stabilizing to driving forces), for the

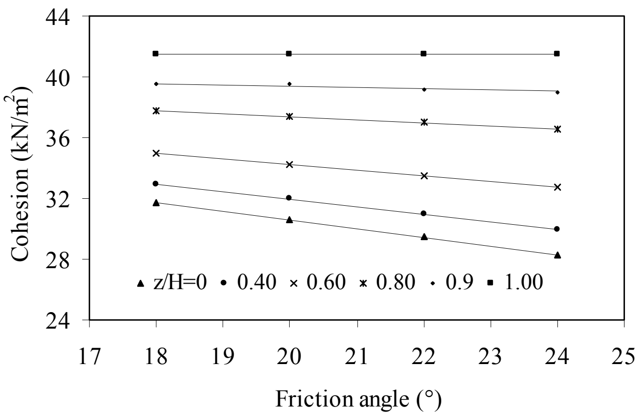


Fig. 4 Relation between cohesion and friction angle of planes of schistosity at failure.

given geometry, is equal to :

$$FS = \frac{cA + (W \pm E_v)\cos\psi_p - U - E_h \sin\psi_p}{W \sin\psi_p + E_h \cos\psi_p \pm E_v \sin\psi_p}\tan(\phi_m + \psi) \quad (5)$$

For each analysis, the maximum volume which geometrically can be involved in the slide (i.e. that corresponding to a sliding plane passing through the toe of the slope), is considered. Three loading cases were examined, according to the design criteria for road cuts set by EGNATIA ODOS SA (2001):

1. No earthquake – No water, with minimum accepted factor of safety equal to 1.3.
2. Water –No earthquake, minimum accepted factor of safety 1.2
3. Water-Earthquake, minimum accepted factor of safety 1.0

Table 1. Values of factor of safety for three loading cases

Loading case	1 (z/H_1=0)	2 (z/H_1=0.8)	3 (z/H_1=0.4)
U (kN/m)	-	490.0	78.0
Earthquake acceleration	-	-	0.16g
Horizontal	-	-	0.08g
Vertical	-	-	0.04g
E/Q force (kN/m)	-	-	71.2
Friction angle φ_m (deg)	22.0	21.0	21.0
Dilation angle (deg)	0.5	0.5	0.5
Cohesion c (kPa)	35.0	35.0	35.0
Factor of Safety	1.15	0.84	0.98
Min F.S. required	1.30	1.20	1.00
External force required (kN/m)	99.5	263.0	21.0

The results of the analysis in terms of the factor of safety, are given in Table 1. In the loading case no 2 the analysis is based on the assumption of a slope with z/H_1=0.80, representing the worse scenario over a period of 50 years.

When earthquake force is considered, the depth of water z/H_1 is taken equal 0.4, which is estimated to correspond to the worse scenario over a period of one year. As shown in Table 2, the factor of safety is well below unity when z/H1=0.80 and marginally lower than unity when the case of a water depth equal to $0.4H_1$ and additionally earthquake action is considered. The external force per meter of slope length, required to raise the factor of safety to the minimum acceptable level is also given in Table 2. The determination is based on the selection of fully grouted rock anchors inclined at an angle of 10º downwards.

Probabilistic analysis

The probabilistic analysis followed the procedure described by Hoek(1998) and performed using the computer program @RISK, developed by the Palisade Corporation (2005). The selection of values and distributions of the variable parameters are as follows:

Cohesive strength c. A normal distribution has been assumed for cohesion, with a mean value equal to 35 kPa and a standard deviation of 6 kPa. The normal distribution is truncated by a minimum value of 29 kPa and a maximum value of 41 kPa (Fig. 5.a). This range is believed to safely cover the expected values for this parameter.

Friction angle φ_m . Similarly, a truncated normal distribution has been assumed, for friction angle with a mean value equal to 21°, which is the approximate centre of the estimated range of shear strength parameters illustrated in Fig. 4., and equal to the value of friction angle used for the deterministic analysis. A standard deviation of 4° is assumed and the normal distribution is truncated at a minimum value of 17° and a maximum value of 25° which are estimated to represent the extremes for this parameter (Fig. 5b).

Dilation angle (ψ). The dilation angle is assumed to follow an exponential distribution with truncation, represented by a minimum value of 0°, a mean of 0.5° and a maximum equal to 3° (Fig. 5c). This implies that there is a 5% probability for the dilation angle to be equal to or lower than 0.33 and 95% equal to or lower than 0.95.

Water pressure (U). The build-up of water pressure is assumed to be according to the triangular distribution described earlier. A truncated exponential function is used, with truncation represented by the U_{max} value (z/H_1=1) and the U_{mIN} value by z/H_1=0.30 (Fig. 5d). With this distribution, there is a probability of 5% for the water depth (z/H_1) to be equal to or lower than 0.03 and 95% equal to or lower than 0.91.
A synopsis of the values and assumed probability distributions for the random variables used for the determination of the factor of safety are given in Table 2. The fixed parameters are the same as those used for the deterministic approach.

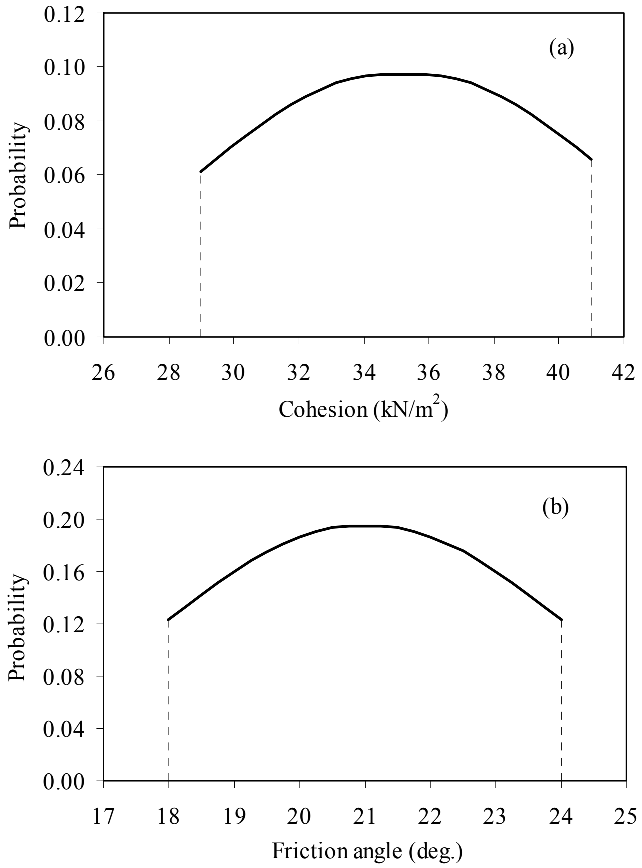


Fig. 5(a,b). Probability distribution of random variables. (a) Cohesion. (b). Friction angle

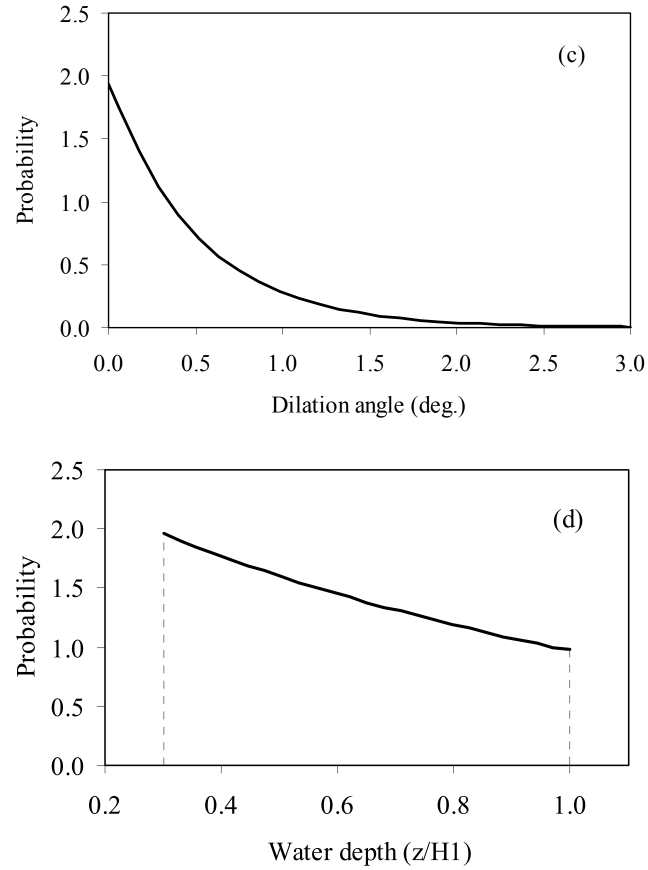


Fig. 5(c,d). Probability distribution of random variables. (c)Dilation angle. (d). Water depth.

The probabilistic analysis was carried out using a Latin Hypercube sampling with n=20000. Using the distributions shown in Fig. 5 and the model shown in Fig. 2, the probability distribution function of the factor of safety, is shown in Fig. 6a. This graph gives a mean factor of safety of 0.97 with a standard deviation of 0.09, a minimum of 0.62 and a maximum of 1.21. There is a 5% probability that the factor of safety will be lower than 0.82 and 95% lower than 1.11. The probability of failure of the slope is P(failure) = P(F<1.0)=0.62, that is, during the lifetime of the slope and for the assumed combinations of water pressure, seismic acceleration, cohesion, friction and dilation angle, the probability of failure is 62%. This is an unacceptably low value for the factor of safety.

The external force required to raise the mean factor of safety to 1.00 is 21 kN/m. In this case the minimum factor of safety is 0.64, the maximum 1.22 and the standard deviation 0.09. A value of the factor of safety equal to 0.84 corresponds to a probability of 5% and 1.14 to 95%. The probability of failure is P(F<1.0)=52.45%. This is also an acceptably low value.

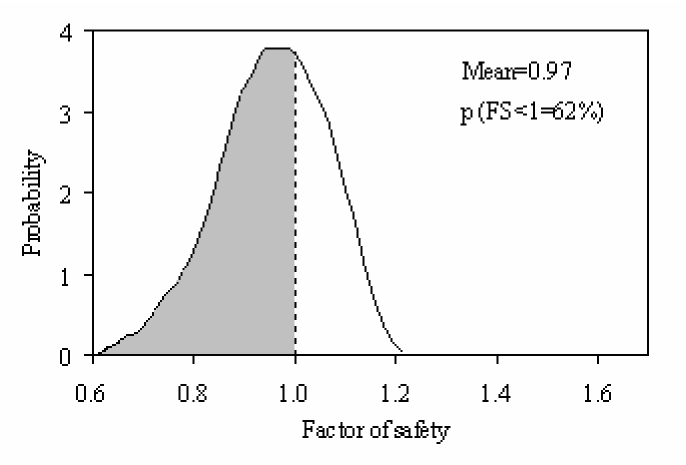
The characteristics of the distribution of the factor of safety are presented in Table 3 for external force equal to 0, 21, 100, 150, 200 and 250 kN/m. The probability of failure is 61.86%, 52.45%, 19.25%, 6.73%, 2.25% and 0.72% respectively.

Table 2. Characteristics of distributions of random variables

Variable	Cohesion (kN/m^2)	Friction angle (deg)	Dilation angle (deg)	Water (z/H_1)
Distribution	Normal	Normal	Exponential	Exponential
Mean	35.0	21.0	0.50	0.4
Min	29.0	17.0	0.0	0.0
Max	41.0	25.0	3.0	1.0
SD	± 3.14	± 2.16	± 0.48	± 0.28
P<5%	29.9	17.5	0.03	0.01
P<95%	40.1	24.5	1.50	0.57

It is considered that, a value of a risk of failure less than or equal to 2.25%, corresponding to an external force of200 kN/m is acceptable for this type of problem, where the consequence of failure will be minor due the small height of slope and the safe distance of the building from the slope toe.

The probability distribution of the factor of safety before and after the installation of the remedial measures in Fig. 6. As can be seen from Fig. 6b, the probability distribution of the factor of safety after the installation of the remedial measures adequately resembles a normal distribution . With a mean value of factor of safety equal to 1.22 and a standard deviation equal to 0.11, the



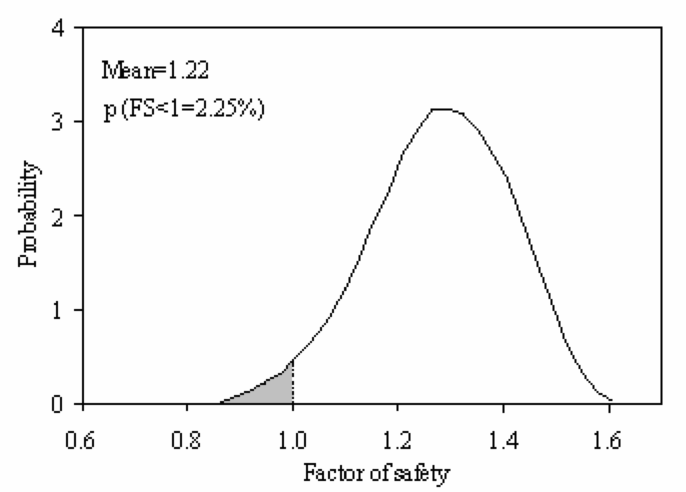


Fig. 6. Distribution of factor of safety before (a) and after (b) the application of the remedial measures.

Table 3. Factor of safety for different values of external force

External force (kN/m)	0	21	100	150	200	250
Mean F	0.97	1.00	1.09	1.15	1.22	1.29
St. Dev.	0.09	0.09	0.10	0.10	0.11	0.11
Min. F	0.62	0.64	0.72	0.76	0.82	0.86
Max. F	1.19	1.22	1.34	1.42	1.51	1.61
P(F)<1 (%)	61.86	52.45	19.25	6.73	2.25	0.72
F (p<5%)	0.82	0.84	0.92	0.98	1.04	1.11
F (p<95%)	1.11	1.13	1.24	1.31	1.39	1.47

normal distribution implies that 68% of the calculated F values are between 1.11(1.22-0.11) and 1.33 (1.22+0.11) and 95% between 1.00(1.22-2x0.11) and 1.44 (1.22+2x0.11).

REMEDIAL MEASURES

The stability of the slope was improved by the application of the necessary number of fully grouted rock anchors. The required external force determined by the two approaches was 263 kN/m for the deterministic and 200 kN for the probabilistic approach. Four rows of fully grouted steel rock anchors (DYWIDAG grade S500), inclined at an angle of 10° downwards and having diameter of 20 mm and a working load of 90kN, were applied. The shear strength of the anchors was taken into account and obtained equal to 40% of the proof stress, whereas their spacing was calculated to 1.4 m and 1.8 m for the two cases. Finally, the lower value for spacing (1.4 m) was selected.

The length of the anchors beyond the failure plane was calculated from the shear strength along the grout-rock interface (BSI, 2000) and found to be equal to 1.0 m, resulting in a mean total length of the anchors equal to 4.0 m.
Complementary measures included:
a) a collection ditch eight meters away from the crest, aiming to collect the surface water and discharge it in a controlled manner at the two sides of the slope and
b) b) a rock trap ditch with a vertical concrete wall 1.25m high, at a distance of 1.50 m from the toe of the slope.

CONCLUSIONS

The effect of water was critical for the stability of this particular slope. Fortunately, there were no serious consequences from the rock slide, but the lesson learnt was significant: The possible accidental action of water forces must not be overlooked.

The stability analysis was carried out using both deterministic and probabilistic approaches. The input data were determined from simple in-situ and laboratory tests and also from back-analysis.

The results from the two approaches showed that the probabilistic approach offers significant advantages, providing a better feeling of the effect of the uncertainty and variability of the input parameters and in this case a more economical solution, given that a risk of failure equal to 2.25% is acceptable.

The remedial measures, consisted of four rows of fully grouted steel anchors at a spacing of 1.40 m. The anchors had a diameter of 20 mm and a working load of 90 kN. The application of these measures raised the safety factors, required by the deterministic approach, to the prescribed by the relevant codes levels whereas the risk of failure in the case of the probabilistic approach was less than 1%.

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