UNCERTAINTY ANALYSIS FOR SEISMIC HAZARD IN THE COLLAPSE OF DIP SLOPE ON FREEWAY NO. 3

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ABSTRACT

Long-term water immersion can cause the weakening of slope rocks and a substantial decline in the parametersof the rock layer, such as cohesion C and friction angle Ø, thus leading to slope failure. Generally, the traditional analysis of slope failure sets the fixed valuesto the required parameters by means of limit equilibrium, while leaving out the uncertainties related to the parameters. It fails to fully reflect the slope safety factor, resulting in insufficient reliability of the calculated safety factor.

This study examines the collapse of Dapu dip slope on Freeway No. 3 as a case to review the uncertainties in the variation of safety factor before and after water immersion and the weakening of slope rocks. It conducts 4 kinds of probability analysis, ROSEN, MDPE, HARR and GMCS to calculate the corresponding slope safety factors for comparison, takes advantage of Monte Carlo method to establish the reliability of slope slide and probability of slope failure, and finally explores the effect ofρon safety factor, wherein ρ is the correlation coefficient of the rock parameters c and tanØ.

Keywords : Dip Slope, Rock Weakening, Reliability, Monte Carl

I. INTRODUCTION

Taiwan is located at the junction of the Eurasian plate and the Philippine Sea plate, with faults and joints caused by plate movement that gives rise to crustal extrusion. At the same time, its young age of deposition and vulnerability to erosion and weathering results in poor cementation and soft rocks in some areas. Together with rain in Taiwan, severe weathering makes the stratum even weaker.

The methods of slope stability analysis proposed by the current study consists of limit equilibrium analysis and limit analysis. Limit analysis needs to identify the stress-strain relationship for the slope material, which is too complex to grasp, so the traditional engineering design mainly conducts limit equilibrium analysis for slope stability analysis. Slope stability analysis is often represented by Factor of Safety (FS) in many ways, for example, the intensity ratio of anti-sliding shear stress to sliding shear stress for the sliding surface FS on most infinite slopes, anti-sliding force to sliding force on finite slopes, anti-sliding moment to sliding moment on arc sliding slopes.Slope height ratiocan also be applied, namely the ratio of the critical slope height calculated via a theoretical formula to the actual slope height, as well as the method of strength reduction. If we take into account all these representations, Factor of Safety (FS) can be represented by the ratio of anti-sliding factor to sliding factor on the sliding surface:

$$FS = \frac{R}{D}$$,R= resistance factor, D= sliding factor (1)

As long as FS> 1, the slope is stable; if FS <1, the slope instability will cause sliding or collapse. Almost all the representationsfail to consider the uncertainties of variables or parameters in the analysis model. For example, resistance and sliding factors contain many objective and subjective uncertainties, so sometimes collapse or sliding occurs even though the designed slope FS> 1. The two systems have the same mathematical formula, each variable has the same mean (e.g., $\mu_A = \mu_B$), which brings the same design FS (e.g., $FS_A = FS_B$), but variable measurement contains uncertainties, and the variances of variablesare not the same (e.g., $\sigma_A \neq \sigma_B$), resulting in different probabilities of failure in the two systems, (e.g., $P_f(A) \neq P_f(B)$), so damage occurs to some of the traditional slope designs that meet the FS requirements.Fig.1 shows the systems of A and B. The means are the same ($\mu_A = \mu_B$), so the safety factors are also the same ($FS_A = FS_B$). However, $\sigma_A < \sigma_B$, so $P_f(A) < P_f(B)$. This is the case in which the two systems have the same safety factor but different probabilities of failure.

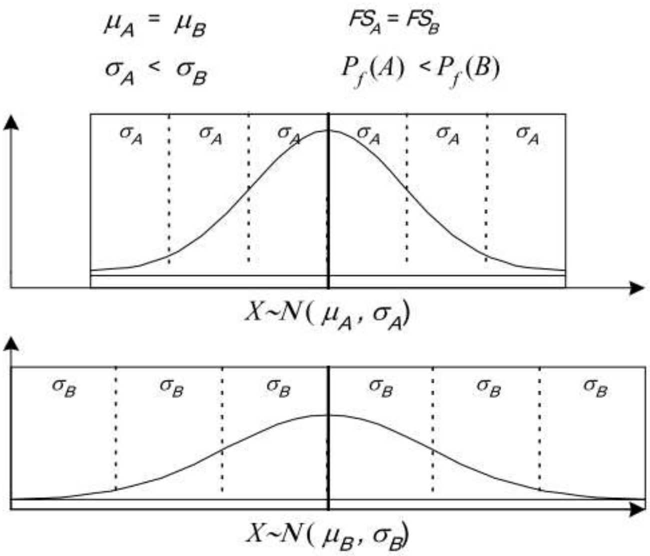

Fig.1 Systems of A and B

Therefore, the “safe design” based on traditional FS may not be the true reflection of “safety”. The Reliability Analysis Model, which considers the effect of the variance of each variable or parameter, can calculate the probability of failure and Reliability Index (RI) or Safety Index (SI). It is a better way to show the degree of slope safety and reliability and achieve the effect of warning.

This study examines the failure of Dapu dip slope on Freeway No. 3 as a case to establish a slope model according to the field conditions without considering external forces and anchor reinforcement,in an attempt to explorethe variations of slope safety factorand variability of rock slope parameters (γ, c, Ø) before and after immersion and the weakening of slope rocks. It conducts 4 kinds of probability analysis, ROSEN, MDPE, HARR and GMCS, to explore the effect of parameter uncertainty on slope stability,as well asMonte Carlo method to establish the reliability of slope slide and the probability of slope failure, and finally explores the effect of correlation coefficientof rock parameters c and tanØ, ρ, on safety factor.

II. CASE DESCRIPTION

A severe slope failure occurred at 3.1K of Freeway No. 3 on April 25, 2010 (seeFig.2). The region's strata are composed of the Talio formation of Early Miocene and the Shihti formation of Middle Miocene, both of which strike NE-NNE and tilt southeastward, with the geological cross-section shown in Fig.3, in which slide occurred mainly along the thin interbed and thin laminae of sandshale. The slope failure is about 185m long from the collapse source to the freeway slope, and about 155m wide at the bottom. The collapse source fell by 15.8m from about 161.5m to 145.7m, resulting in a damaged area of 14,000m^2.

Fig.2 Large-scale landslide of dip slope at 3k + 100 of Freeway No. 3

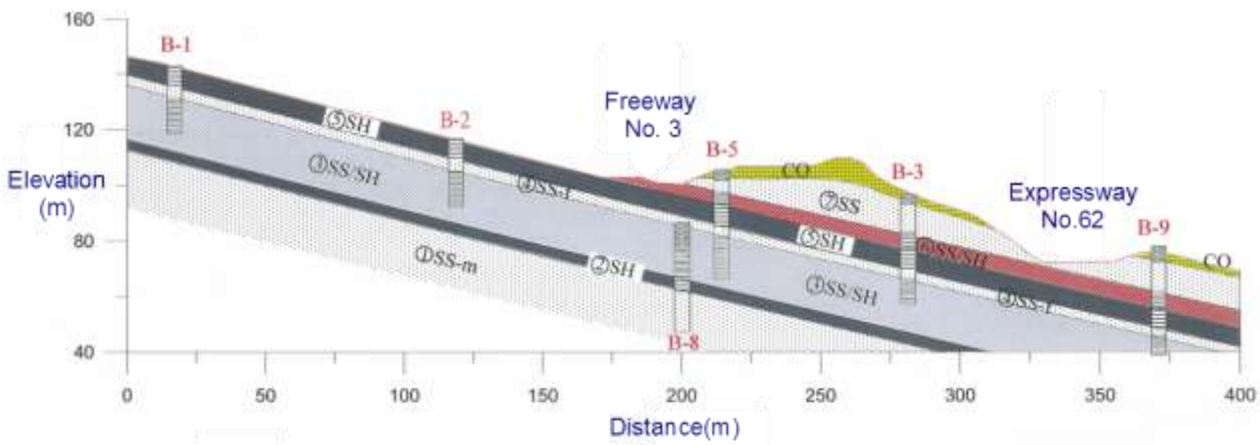

Fig.3 Stratigraphic section perpendicular to the freeway (MOTC, 2011)

After the landslide occurred at 3.1 km of Freeway No. 3 in Qidu on April 25, 2010, the Ministry of Transportation and Communications (MOTC) presented a report on the disaster in the following year, and the landslide findings and the formation parameters are summarized as follows:

1. The disaster is mainly a typical dip slope slide. Judged from the features, the critical slip condition had been

reached before the destruction as the groundwater infiltration softened the sliding surface of the dip slope wedge. At that time, only the rusty anchors maintained their stability and the steel strand of the anchors was overloaded in the disaster, which was the result of the continuous destruction of the independent anchors within a short time.

2. The subsequent endoscopic investigation of the anchor components exposed at the dip slope site show significant anchor corrosion, which not only gradually reduced the tensile strength and safety factor of the anchor, but also caused the slope to collapse with in the shortest time.
3. Destruction course: The disaster is a dip slope slide caused by up to more than a decade of gradual weakening and anchor component corrosion due to long-term groundwater infiltration. The main reasons for destruction are developed slope joints, obvious geological structure of the dip slope, groundwater infiltration and seasonal water level changes that softened the rocks and corroded the anchor strand.
4. The shear strength parameters of the rocks on the site of the dip slope slide of Freeway No. 3 are excerpted in Table 1, where the test numbers RDS (D)-4, RDS (D)-5, RDS (W)-3 and RDS (W)-4 are the formation parameters of the sliding surface for shale and sandshale.

III. ESTABLISHMENT OF SLOPE FAILURE MODEL

The landslide at 3.1 km of Freeway No. 3 is mainly a dip slope failure due to the slide between sandshale strata. Lateral sliding analysis and simulation is carried out in this paper, as shown in Fig.4, and is described as follows:

$$F.S. = \frac{cL+}{W} \tag{2}$$

C and φ are the interface parameters of shear strength, W is the weight of the slider △ABC, L is the length of the sliding surface, θ is the inclination of the sliding surface, and N is the positive force acting on the sliding surface. Because:

$$W = \frac{1}{2}rH(H\cot\theta - H\cot\beta) = \frac{1}{2}rH^2\frac{\sin}{\sin} \tag{3}$$

$$N = V \tag{4}$$

$$L : \tag{5}$$

Table 1Shear Strength Parameters

No.	Hole No.	Depth (m)	normal stress (kg/cm^2)	peak strength Cp (kg/cm^2)	peak strength *Øp* *(degree)*	residual strength Cr (kg/cm^2)	residual strength *Ør* *(degree)*	lithologic character	plane of shear

RDS(D)-1	B-1	3.00-4.00	2.0/5.0/8.0	2.5	28.0	0.0	25.0	shale	Intact Rock
RDS(D)-2	B-2	4.00-5.00	2.0/4.0/5.0/7.0	2.6	30.1	0.0	28.0	shale	Intact Rock
RDS(D)-3	B-4	2.60-3.60	2.0/5.0/7.0/8.0	1.5	26.7	0.0	22.0	shale	Intact Rock
RDS(D)-4	B-6	16.60-17.00	2.0/5.0/7.0	0.28	22.5	0.0	19.8	Alternations of S.S. & Sh.	bedding plane
RDS(D)-5	B-7	18.00-19.00	2.0/4.0/5.0/6.0	3.2	28.5	0.0	22.7	shale	Intact Rock
RDS(D)-6	B-8	16.00-17.00	2.0/4.0/5.0/7.0	0.7	36.5	0.0	29.0	Alternations of S.S. & Sh.	Intact Rock
RDS(W)-1	B-1	0.00-1.00	2.0/4.0/5.0/7.0	2.1	29.0	0.0	17.2	shale	Intact Rock
RDS(W)-2	B-2	3.40-3.80	2.0/3.0/5.0/7.0	0.5	46.0	0.0	22.0	shale	Intact Rock
RDS(W)-3	B-3	16.60-17.00	2.0/5.0/7.0	0.9	27.7	0.0	23.2	Alternations of S.S. & Sh.	bedding plane
RDS(W)-4	B-5	16.60-17.00	2.0/3.0/5.0/7.0	1.1	26.2	0.0	14.1	shale	Intact Rock
RDS(W)-5	B-9	38.00-39.00	2.0/3.0/5.0/7.0	0.5	34.6	0.0	21.0	Alternations of S.S. & Sh.	Intact Rock
RDS(W)-6	B-10	10.25-11.00	2.0/4.0/5.0/7.0	1.4	37.0	0.0	24.6	Alternations of S.S. & Sh.	Intact Rock
RDS(W)-7	B-6	22.40-22.50	1.0/2.0/4.0	-	-	0.0	21.5	shale	Intact Rock
RDS(W)-8	B-6	17.30-17.40	2.0	-	-	0.0	20.0	Alternations of S.S. & Sh.	bedding plane

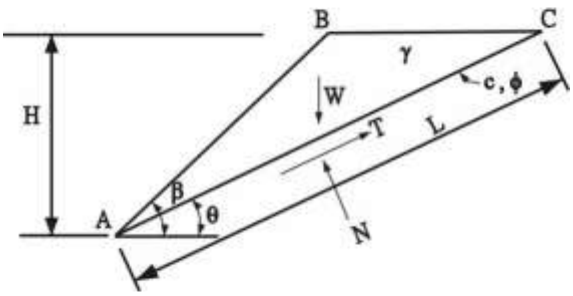

Fig.4 Parallel sliding surfaces of finite slopes

Plug (3), (4), and (5) into (2) to get:

$$F.S. = \frac{\text{Resistance}}{\text{Driving force}} = \frac{2c\sin\beta}{\gamma H \sin(\beta-\theta)\sin\theta} \text{ -} \tag{6}$$

$$\text{Probability of failure Pf} = FS(c, \varphi, \gamma) - \tag{7}$$

This study establishes the parameters for the slope model by simulating the landslide disaster at 3.1 km of Freeway No. 3 in reference to the MOTC report (2011). We take into account the variability of all the formation parameters, γ, c and φ, except for the slope height H = 25m, the sliding surface inclination $\theta = 15^0$ and slope angle $\beta = 20^0$. The parameters of slope failure model thus established are shown in Table **2**.

Table 2 The parameters of the slope failure model

Parameter	Mean	Standard Deviation (std)	Distribution	Remarks
γ(Kn	22.9	0.11	Normal distribution	
Cp(Kn	2.8-32		Uniform distribution	Before immersion (peak strength)
Cr(Kn $/m^2$)	0		Constant	After immersion (residual strength)
Øp(degree)	26.1	2.23	Normal distribution	Before immersion (peak strength)
Ør(degree)	19.95	3.62	Normal distribution	After immersion (residual strength)

IV. SENSITIVITY ANALYSIS

This study used EXCEL to calculate the range of rock safety factor both before and after the immersion, and found out that the safety factor was about 3.04 before and 1 after, which was consistent with the MOTC report (2011), thus verifying the feasibility of the lateral-sliding slope failure model employed in this study.

The MOTC report (2011) shows that the rock unit weight γ had no significant variation before and after the immersion, but the cohesion c and the friction angle φ substantially reduced after the immersion, resulting in the weakening of the rock, so the variability of the parameters c and φ need to be taken in account. This study therefore explores the uncertainty of these two parameters (cohesion c and friction angle φ), which have higher variabilitythatn other parameters, as well as to what extent their uncertainty influences slope safety factor. In this study, we take into account the variability of the formation parameters c and φ for single-factor and multi-factor sensitivity analysis,respectively,based on field conditions except for the slope height $H = 25m$, the sliding surface inclination $\theta = 15^\circ$,the slope angle $\beta = 20^\circ$,and the unit weight γ = constant. The analysis results arepresented as follows:

4.1 Univariate Sensitivity Analysis

In this study, univariate sensitivity analysis is performed when c and φreduce respectively by 10%, 20% and 30% to facilitate the understanding of how the formation parameters influence FS, with the findings shown in Fig.5. It is found that when cohesion c and φreduce by 30% respectively, the safety factor reduces by 27.63% and 14.47% respectively. The sensitivity of cohesion c is nearly 100% higher than that of friction angle φ, showing that cohesion c has a greater effect on slope safety factor than friction angle φ does.

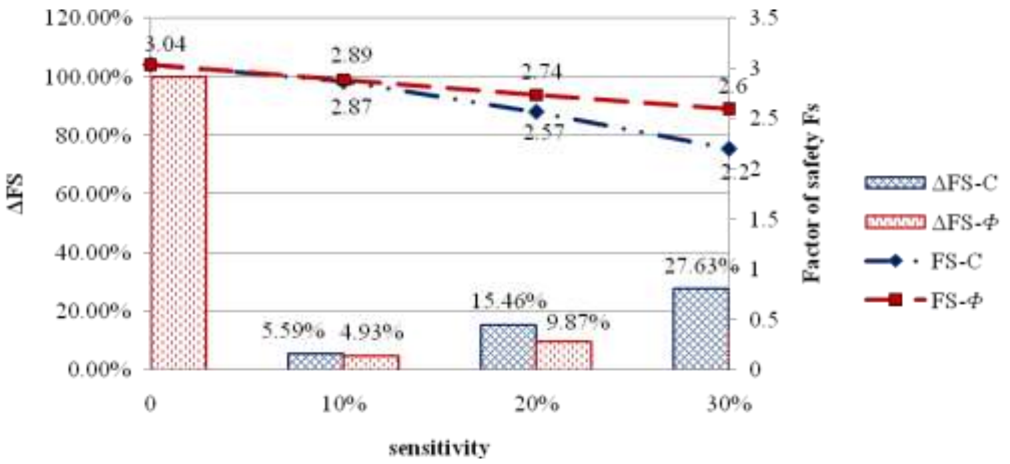

Fig.5 Univariate Sensitivity Analysis

4.2 Multivariate Sensitivity Analysis

The rock weakening due to the immersion will reduce cohesion c and friction angle φ. This study employs multivariate sensitivity analysis to study the changes in slope safety factor after the decrease in the interaction of cohesion c and friction angle φ (as shown in Fig.6). The study has found that when cohesion c and friction angle φ drop down to 6Kpa and 19^0 respectively, the slope safety factor FS <1, which will lead to an unstable state.

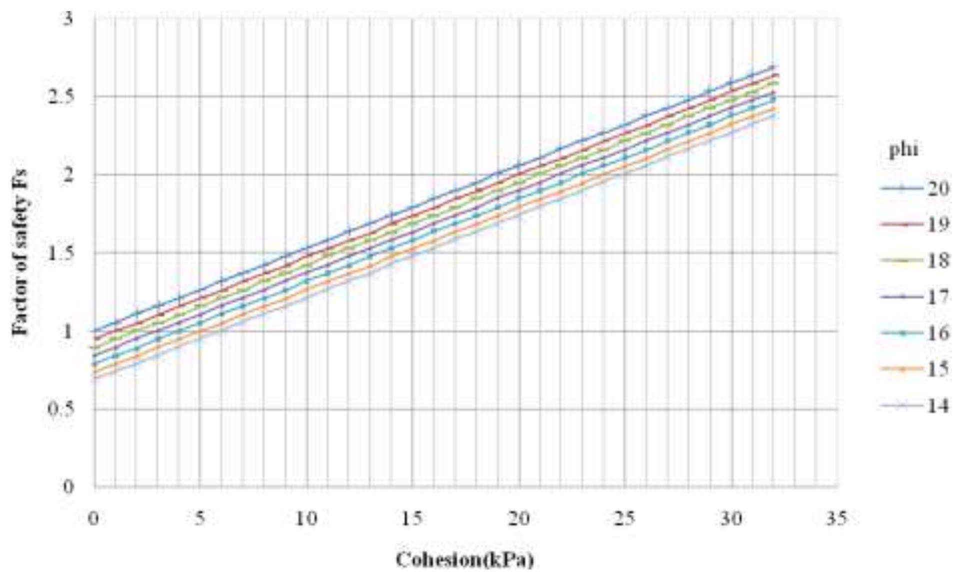

Fig.6 Multivariate Sensitivity Analysis

V. UNCERTAINTY ANALYSIS

In this section, uncertainty analysis is conducted to explore the effect of rock slope parameter (c, φ) variability on stability. Four probability estimation methods, ROSEN, MDPE, HARR and GMCS,are used torespectively calculate the effect of the correlation coefficient for the parameters c and tanφ (ρ = 0 and ρ =-0.5)onthe probability of failure Pf (Pf = FS-1) (in Table 3).The results show thatthe calculated slope safety factor does not changesignificantly with different probability estimation methods regardless of the presence of the correlation coefficient forc and tanφ. The Pf calculatedusing different methods is greater than 1 (Pf> 1) before the immersion (safe side), and after the immersion, the weakening leads to a substantial decline in the slope safety factor, and the probability of failure approaches 0 (Pf≅0) (imminent destruction), thus indicating that the slope is in the state of critical failure.The GMCS method (1,000 samples, detailed in Fig.7) shows that it is consistent with the traditional safety factor analysis FS≅ 1 (Pf≅0), but the probability of failure is up to 50%, thus indicating an urgent need for taking stabilizing measures.It can also be seen from the correlation coefficient for parameters c and tanφ(ρ =-0.5)that when probability P≈ 90% or 10%, the correlation coefficient is more sensitive and affects the probability of failure Pf by approximately ± 4%.

Table 3 Probability estimation methods to assess probability of failure

method		ρ = 0 before the immersion	ρ = 0 after the immersion	ρ = -0.5 before the immersion	ρ = -0.5 after the immersion
ROSEN	MEAN	1.2697	0.0018	1.2697	0.0018
	STD. DEV.	0 .4302	0.1968	0.3970	0.1968
MDPE	MEAN	1.2697	0.0018	1.2697	0.0018
	STD. DEV.	0.4302	0.1977	0.5433	0.1969
HARR	MEAN	1.2697	0.0018	1.2697	0.0018
	STD. DEV.	0.4657	0.1977	0.3969	0.1968
GMCS	MEAN	1.2779	0.0060	1.2596	-0.0030
	STD. DEV.	0.4629	0.1964	0. 5263	0.1964

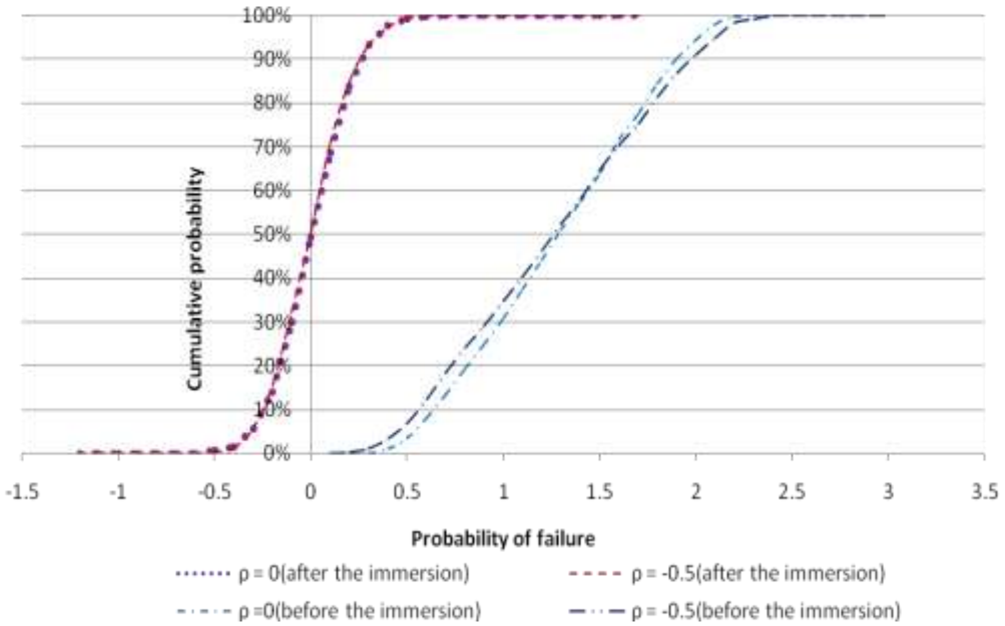

Fig.7 Probability of failure before and after the immersion

VI. CONCLUSIONS AND RECOMMENDATIONS

This study is carried out onthe slope failure at 3K+100 of Freeway No. 3 to establish a slope failure model in reference to field conditions, whichconsiders the uncertainty of rock parameters based on the reliability index method.It uses EXCEL forunivariate and multivariate sensitivity analysis on cohesion C and friction angle φ, and four probability estimation methods, ROSEN, MDPE, HARR and GMCS to assess the variability of rock strength parameters, including the differences inprobability of failure Pf for ρ = 0 and ρ =-0.5, where ρ is the correlation coefficient for c and tanφ. The important conclusions are listed as follows:

1. Before the immersion, the slopewas in a steady state, wherein FS> 2.The weakeningcaused by the immersion reduces cohesion C from 32Kpa to 0Kpa and friction angle φ from to below 1', and the safe coefficient FS is less than 1, which will give rise to slope instability, so water is the biggest factor affecting slope stability.
2. Before and after the immersion, the rock unit weight changes little, but cohesion c and friction angle φ change a lot. The tests of single factor (c or φ) on the slope safety factor show thatcohesion c is more sensitive to slope stability uncertainties than friction angle φ.
3. The multivariate sensitivity analysis of cohesion c and friction angle φ shows that when cohesion c drops to 6Kpa and friction angle φ to , the slope safety factor FS <1 and the slope is subject to an unstable state.
4. ROSEN, MDPE, HARR and GMCS were all applied forthe estimation and they differ little from one another in calculating the slope safety coefficient before and after the immersion.The GMCS method (1,000 samples) shows that it is consistent with the results of the traditional analysis FS≈1, but the probability of failure is up to 50%, thus indicating an urgent need for taking stabilizing measures.
5. The study also considered the correlation coefficient for c and tan φ (ρ = 0 and ρ =-0.5), and thetested probability isP≈ 90% or 10%.The higher sensitivity of correlation coefficient when ρ =-0.5 affects the probability of failure Pf by approximately ± 4%.

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