DERIVATION OF EMPIRICAL EQUATIONS FOR DETERMINATION OF SAFE PIT SLOPE GEOMETRY AT PHUKHAM MINING

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Abstract - Objective of this study is to investigate rock shear strength parameters of fault and shear zone at South Wall of Phu Kham open pit mine by direct shear testing. The test results are simulated in numerical models using distinct element method (UDEC code) under different slope faces and angles and shear strength parameters. The model results indicate that the safety factors of slope increase with increasing cohesions, friction angels and distances between fault and slope face, but decreases with increasing slope face angles. An empirical equation is derived here using the simulation results. It can be used to predict the safety factor of slopes under various slope face angles, cohesions and friction angles of the hanging wall. Distances between fault from crest are not derived in equation because they are not sensitive to the factor of safety.

Keywords - Slope Angle, Fault, Friction Angle

I. INTRODUCTION

Rock slope stability is considered crucial to public safety in engineering project areas, such as open pit mine, dam, road cut, and railway. Slope failure can be affected by rock mass characteristics, slope geometries, groundwater and other operating conditions. Geology comprehension is necessary for rock slope design. In the mining process, small changes in the slope face angle may have large consequences on the mining operation economy. Phu Kham open pit mine is the largest copper reserve in Laos, which is located in PanAust's Phu Bia Mining Contract Area, Xaisomboun Province, Lao PDR. A problem of Phu Kham open pit is slope instability at the South Wall. This location has specific discontinuities that lead to the decrease of the slope stability. To prevent slope failures, several methods have been proposed by researchers. Empirical method or rock mass classification system is often used for preliminary evaluation of rock mass behaviors [2]. It is simple and convenient to use. Numerical method is used for a more complex slope geometry and failure mechanisms. It is useful when the other methods cannot represent rock mass behaviors [9, 10]. Even though the empirical methods have been proposed by many researchers to quantify rock mass behavior, the effort of rock mass classification for specific pit slope design of Phu Kham open pit has rarely been attempted. To correlate the shear strength parameters with the rock slope behavior, empirical equations for predicting the slope failure are needed. Such equations can be readily applied, and hence allows a quick excavation and production of the ore.

The objective of this study is to determine rock shear strength parameters of fault and shear zone at South Wall of Phu Kham open pit mine by direct shear testing. The laboratory results are demonstrated in numerical simulations with a distinct element method using the Universal Distinct Element Code (UDEC) [5]. The numerical results are used to develop mathematical relationships between shear strength parameters under different factors of safety (FS).

II. LABORATORY TESTINGS

Three types of samples are used in this study. They are volcanic tuff (VTF), Red Beds (SLR), and faults collected from the South Wall zone in the Phu Kham pit. The VTF and SLR, and fault are in hanging wall, footwall, and major fault, respectively (Fig. 1 and 2). All samples are collected from diamond drill cores, there are three groups of sample preparation for different test methods, 1) uniaxial compressive strength tests (UCS), 2) direct shear tests of rock specimens on rough fracture (DS), and 3) direct shear tests of soil samples (DS- soil). The core specimens with a nominal diameter 61 mm (HQ bit size) tested here were drilled from depths ranging between 15.50 to 183.40 m. Twenty-eight of VTF specimens and thirty-one of SLR specimens are cut to obtain a nominal length-to-diameter ratio (L/D) ranging between 2.0 to 2.5 for UCS testing, following the ASTM D7012 standard practice. Six pairs of VTF specimens and ten pairs of SLR specimens are selected from a single discontinuity and cut to have a nominal length 10 cm for DS testing, following the ASTM D5607 standard practice. Three soil samples of fault for DS-soil test are collected for three specimens. They packed in plastic wrap to minimize disturbance, following the ASTM D3080 standard practice. Figs.3 and 4 are show some pre-test and post-test of VTF and SLR samples.

A. Uniaxial Compressive Strength Test (UCS)

The rock strength or compressive strength of the joint walls is required as data input in numerical simulations. The laboratory testing procedure strictly follows the American Society for Testing and Materials (ASTM D7012-14). Cylindrical specimens of VTF and SLR with a nominal diameter of 61 mm and L/D ratio between 2.0 to 2.5 are axially loaded to

failure. Twenty-eight of VTF and thirty-one of SLR specimens have been tested. The UCS is calculated from the maximum load divided by the initial cross-sectional area. The elastic modulus (Young's modulus) is determined from the stress-strain curves at 50% of the maximum stress. Test results are shown in Table 1.

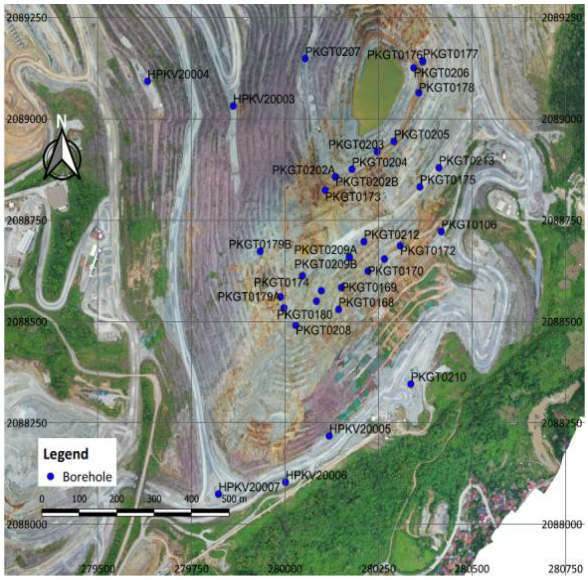
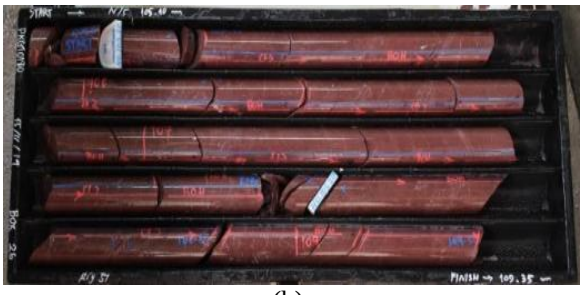


Fig.1. Borehole locations in the study area



(a)



(b)



(c)

Fig.2. Core samples: VTF (a) and SLR (b) and fault samples (c) collected for testing.



(a) (b)

Fig.3. Some VTF (a) and SLR (b) specimens prepared for UCS testing.



(a) (b)

Fig.4. Post-test VTF (a) and SLR (b) specimens of UCS testing

Properties	VTF	SLR
Density (g/cm^3)	2.73 ± 0.08	2.84 ± 0.17
UCS (MPa)	16.14 ± 15.30	15.42 ± 17.54
E (GPa)	20.16 ±12.10	14.57±7.88
Poisson's ratio	0.26 ±0.03	0.22 ±0.04

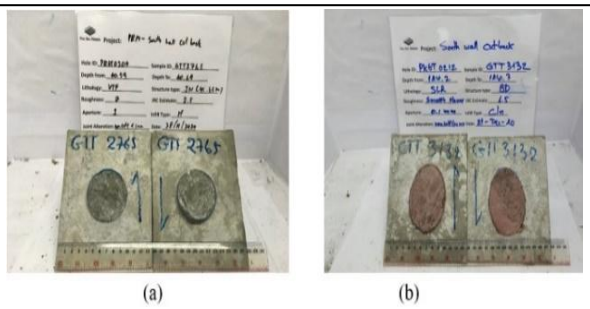
Table I Summary UCS Testing Results

B. Direct shear tests of rock specimens with rough fracture

The rock shear strength is a significant parameter for the design of rock slopes, foundations, and structures. The test procedure is followed the American Society for Testing and Materials standard (ASTM D5607-16) and the method suggested by the International Society of Rock Mechanics (ISRM) [3]. Six pairs of VTF specimens and ten pairs of SLR specimens with a nominal length 10 cm contain discontinuities (i.e., bedding, joint), and are separated into upper and lower halves of the encapsulated cement molds (Fig.5). The shear displacement is applied under constant rate of 0.2 mm/min. A minimum of 10 sets of monitoring and recording data points is suggested to be taken before obtaining the peak shear strength. The loading continues until a residual shear strength is established. Post-test VTF and SLR specimens are shown in Fig.6. The test results are shown in Table 2.

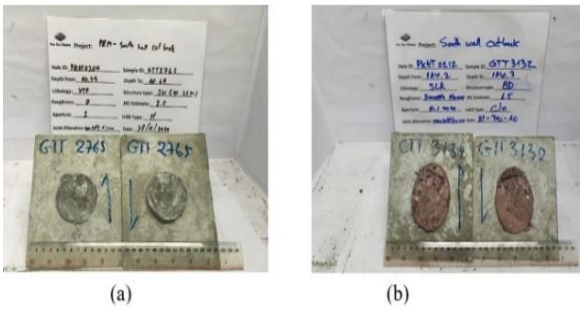
C. Direct shear tests of soil samples

A set of direct shear tests on compacted soil samples has been conducted on fault samples. After compaction, the soil samples are placed and set up in a direct shear load box and frame. All samples are sheared under a constant displacement rate of 0.1 mm/min. The test results are shown in Table 2.



(a) (b)

Fig.5. Some VTF (a) and SLR (b) specimens prepared and molded for DS testing.



(a) (b)

Fig.6. Post-test of VTF (a) and SLR (b) specimens of DS testing.

Properties	VTF	SLR	Fault
Density (g/cm^3)	2.73	2.84	2.03
Cohesion (kPa)	1,000.00	500.00	128.71
Friction angle (°)	35	33	15

Table II Summary DS Testing Results

III. NUMERICAL SIMULATIONS

Discrete element analyses are performed using UDEC [5] to describe the slope stability behavior. The discrete element models are constructed to represent various slope face angles (□) and distances between fault and crest (X). The models use mechanical properties obtained from the laboratory testing (Tables 1 and 2). The slope heigh, fault width, and fault angle are maintained constant at 100 m, 10 m and 63 degrees. The distance between the toe and right boundary is 100 m and between the crest and left boundary is 150 m. The slope face angle (□) varies from 45°, 60°, 75° to 90°, and distance X varies from 10, 25, 50, 75 to 100 m. Fig.7 and 8 show representative boundary conditions and meshes of slope model. The cohesion and friction angle of VTF (hanging wall) are assumed in models for determine effect of shear strength and maintained constant shear strength of SLR (footwall) and fault. The cohesion (c_{hang}) varies from 0.1, 0.5, 1.0 to 2.0 MPa, and friction angle (□$_{hang}$) from 25.0, 27.5, 30.0, 32.5, 35.0 to 37.5. Initial stress due to gravitational loading is first calculated from the density of the overburden. All computer models assume plane strain condition. Simulation results in the form of displacement vectors are shown in Fig. 9.

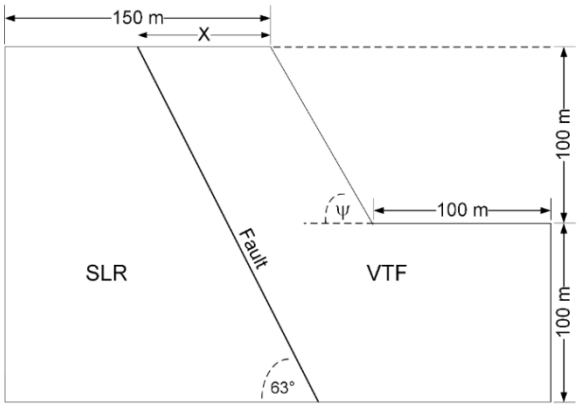


Fig.7. Boundary conditions and geometry of the representative slope model.

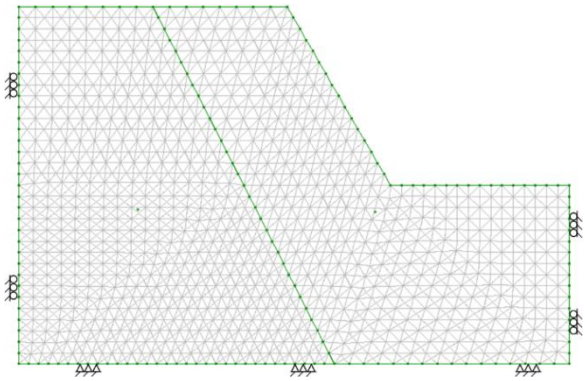


Fig.8. Mesh model.

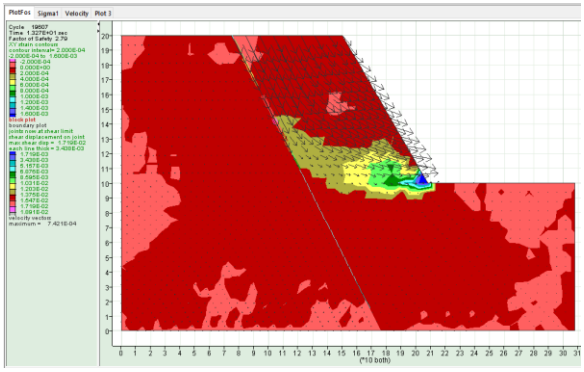


Fig.9. Representative simulation results of displacement vectors.

IV. RESULTS AND DISCUSSIONS

Slope failure is geotechnical hazards that occur due to driving forces overcome resisting forces. The driving force is typically gravity, the resisting force is the shear strengths of slope material. The model results indicate that FS increases with increasing X, c_{hang} and ϕ_{hang} but it decreases with increasing ψ (Figs.10 to 12).The mathematical relationship is derived here using the simulation results. The empirical equation can be used to predict the safety factor of slopes under various slope face angles, cohesions and friction angles of the hanging wall (cohesion and friction angle). Distances between fault from crest are not derived in equation

because they are not sensitive to the factor of safety. Regression analyses are performed using the SPSS code [11] to fit the model results with the derived empirical equation. An indicator of the predictive capability of equation is the correlation coefficient (R^2). Fig. 13 plots the FS as a function of friction angle of hanging wall which is normalized by slope face angle ($\phi_{\eta\alpha\nu\gamma}/\psi$). The relation between the FS, $\phi_{\eta\alpha\nu\gamma}/\psi$ and c_{hang} can be best represented by:

$$FS = 3.4 \times (c_{hang}^{0.458}) \times (f_{hang}/y)^{0.42} \quad (1)$$

The empirical constants in the equation above are generated from the non-linear regression analysis of the model simulation results. The $R^2$of 0.997 indicates a good correlation with the data.

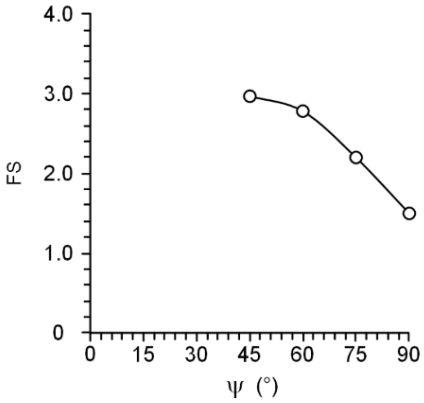


Fig. 10. Factor of safety as a function of slope angle (ψ)

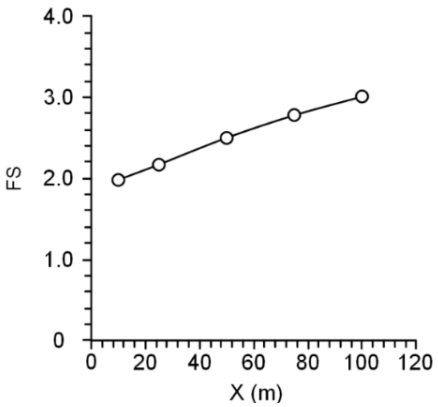


Fig. 11. Factor of safety as a function of fault distance (X)

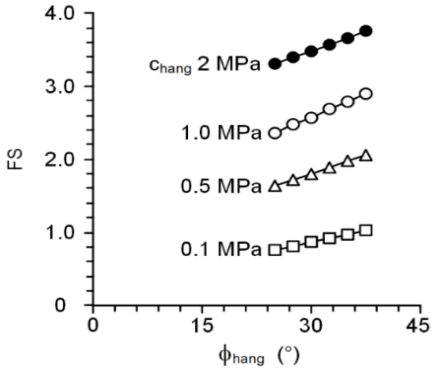


Fig. 12. Factor of safety as a function of hanging wall friction angle (ϕ_{hang})

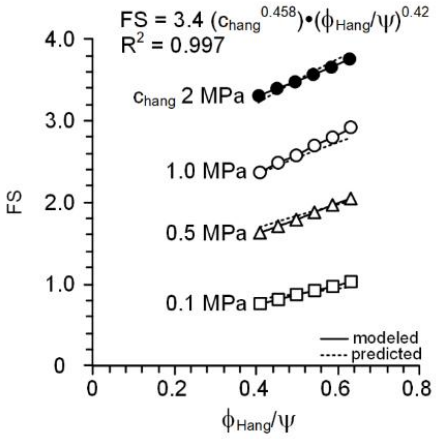


Fig. 13. Predicted FS as a function of ϕ_{hang}/ψ

IV. CONCLUSIONS

Thirty-three numerical models are simulated in this study with the same boundary conditions and basic properties of rock at the footwall and fault. The direct shear tests determine the shear strength parameters of the hanging wall, footwall and fault materials. The simulation results agree well with those of [4, 6, 7, and 8]. The factor of safety depends on shear strength parameters of slope materials and slope face angle, but not sensitive to the distance between fault and slope face.

ACKNOWLEDGMENTS

This study is funded by Suranaree University of Tech. and by the Higher Education Promotion and National Research University of

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