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MODELS FOR PHYSICAL AND NUMERICAL SLOPE FAILURE OF LOOSE SAND UNDER DYNAMIC LOADING

MODEL FISIK DAN NUMERIK KELONGSORAN LERENG MATERIAL PASIR LEPAS AKIBAT PEMBEBAN DINAMIS

ZULFAHMI
R & D Centre for Mineral and Coal Technology
Jalan Jenderal Sudirman 623 Bandung 40211
Ph. 022-6030483, fax. 022-6003373,
email: zulfahmi@tekmira.esdm.go.id

ABSTRACT

Excessive vibration that causes damage to model the medium such as mine slope can physically and numerically be modelled. The slope of this study simply represents the actual view of the slopes that has a smaller size than the actual one, while the numerical model is relates to a mathematical form of slope condition that based on physical and mechanical data of the medium. The slope failure has experimentally been built several variations. Effect of vibration is echieved by connecting the models into the vibration instrument with bearing that can horizontally move free in line within the determined track. The instrument is attached to a spring that can pull the model to side out. The spring is placed in an iron frame. Proviously, the slope has been formed in critical condition one (angle of 30°). Physical model and laboratory test results were used as an input for numerical modelling of the slope failure. Based on the numerical analysis, the SRF was 0.47 for D equal to 2 cm g around 0.0025. If the g's were around 0.0057 and 0.0088, the obtained SRF for both g's were 0.44 and 0.41 respectively. While the D of 4 cm and g of 0.0024 came the SRF of 0.54, the g of 0.0064 derived the SRF of 0.48, and the g of 0.0106 obtained the SRF of 0.44. For D equal to 2 cm and g 0.0024, 0.0106; the obtained SRF was 0.54, 0.48 and 0.44 respectively. Increasing the D to 6 cm within variation of g from 0.0025, 0.0062 and 0.0106, the SRF was 0.51, 0.48 and 0.44 respectively. It is assumed that there is a correlation between the thickness of quartz sand layer and the decrease of SRF value. The correlation also occurs between the increase in vibration (g value) and the SRF.

Keywords: Physical modelling, numerical modelling, vibration, seismic loading coefficient, strength reduction factor

SARI

*Getaran berlebihan yang menyebabkan kerusakan pada medium seperti lereng tambang dapat dimodelkan baik secara fisik maupun numerik. Model fisik lereng pada penelitian ini adalah representasi sederhana suatu kondisi lereng yang memiliki ukuran lebih kecil dari ukuran sebenarnya, sedangkan model numerik adalah bentuk matematis kondisi lereng berdasarkan data fisik dan mekanik media yang dijadikan model lereng. Dalam penelitian ini kelongsoran lereng telah dibuat eksperimen dengan beberapa variasi. Efek getaran diperoleh bila model dihubungkan dengan alat getaran yang memiliki bantalan bergerak bebas secara horizontal sesuai dengan jalur yang telah ditetapkan. Alat ini dikaitkan dengan pegas yang dapat menarik model ke salah satu sisi dengan pegas terkait pada rangka besi. Sebelumnya lereng dibentuk sedemikian rupa sehingga berada dalam kondisi kritis (kemiringan sudut 30°). Model fisik dan hasil uji laboratorium digunakan sebagai masukan (*input data*) untuk pemodelan numerik kelongsoran lereng tersebut. Berdasarkan analisis numerik, dapat diketahui bahwa untuk tebal lapisan pasir kuarsa (D) = 2 cm dengan nilai koefisien beban seismik (g) = 0,0025 diperoleh nilai faktor reduksi kekuatan (SRF) = 0,47, untuk nilai g = 0,0057 akan diperoleh SRF = 0,44 sedangkan bila nilai g = 0,0088 diperoleh SRF = 0,41. Bila D = 4 cm dan nilai g = 0,0024 maka diperoleh SRF = 0,54; untuk nilai g = 0,0064 diperoleh SRF = 0,48 dan nilai g = 0,0106 diperoleh SRF = 0,44, selanjutnya untuk D = 6 cm dan nilai*

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g = 0,0025 diperoleh SRF = 0,51 sedangkan untuk nilai g = 0,0062 diperoleh SRF = 0,48 dan nilai g = 0,0106 diperoleh SRF = 0,44. Dari hasil tes dapat disimpulkan bahwa terdapat hubungan antara tebal zona lapisan kritis yang diilustrasikan sebagai lapisan pasir kuarsa dengan penurunan nilai SRF. Di sisi lain ada hubungan antara peningkatan getaran seperti yang ditunjukkan oleh kenaikan nilai koefisien beban gempa (g) dengan nilai faktor reduksi kekuatan (SRF).

Kata kunci : pemodelan fisik, pemodelan numerik, getaran, koefisien beban seismik, faktor reduksi kekuatan

INTRODUCTION

Blasting operations may cause excessive vibration impacts on the structures. Excessive levels of structural vibration caused by ground vibration from blasting can result in damage to or failure of a structure. A quantitative assessment of soil slope stability is important when a judgment is needed about whether the slope is vulnerable to failure due to various dynamic loads caused by blasting or not. This assessment is made in terms of either determining the displacement along the face of slope or critical acceleration under seismic conditions.

The excessive vibration which causes a damage to the medium such as mine slope can be modelled both physically and numerically. Physical model of the slope that has a smaller size than the actual size is a simple representation of the actual view of the slopes. Slope geometry created in the physical model is the application of dimensional analysis which is expected to represent the actual size, while the numerical model is a mathe-matical form of a condition based on physical and mechanical data of the medium (Lu and Xu, 2003). Numerical modelling is used to model the condition of the rock. According to Arif (1997), it can be divided into continuous, discontinuous and combination of both methods (hybrid). Numerical modelling using continuous method has been modelled by several researchers such as Maxwell and Young (1998), Sato et al. (2000), Sheng et al. (2002), Tonon et al. (2001), Young and Collins (2001). This continuous model is used for continuum or regarded as continuum media. Finite element methods (FEM) is one of very popular continuous methods which is used for the numerical analysis in the continuum media. The FEM is a numerical technique for finding approximate solutions to partial differential equations and their systems as well as integral equations. In a simple terms; such a method divides up a very complicated problem into small elements that can be solved in relation to each other. Subsequently, matrix formation of each element and global can be done, for example stiffness matrix. The FEM assumes implicitly that the rocks within discontinuous representation have influence on the physical behavior such as deformability or strength and is considered to follow the law as constitutive of discontinuous equivalent continuous form.

In this research, the slope failure has been studied experimentally in the form of physical and numerical modelling. The aim is to determine the mechanisms that occur on the factual slopes failure as well as the contribution of each factor in slope and the external factors that trigger the failure.

The slopes of physical model were created using of river sand material size of -20 mesh and quartz sand of + 20 mesh. Parameters used in this study include effect of vibration acceleration by varying spring distance with slope fixed dimensions at height of 9 cm and slope angle of 30° that reffered as the critical angle of sand material. Quartz sand zone is considered as a representation of the material that was damaged by blasting at the face of the slope. The quartz thickness sand was varied from 2, 4 and 6 cm. Meanwhile, the spring track distance was varied from 10, 20 to 30 cm.

In numerical models, dimensions of slope, physical and mechanical properties of river- and quartz sands were very important data to be inputted for calculation. The software used was Phase2, version 6.0.

METHODOLOGY

The main concept of this study is to compare the behavior shown by slope models (physical modelling) when subjected to dynamic load variations (vibration) versus numerical modelling. The next assessment is to compare results of the material strength of fine- (river sand) and coarse sands (quartz sand) tested in lab. The test was performed at the Laboratory of Soil Mechanics, R & D Center for Mineral and Coal Technology (*tek*MIRA) while physical modelling was performed at the Geomechanics & Mining Equipment Laboratory, Bandung Institute of Technology using the equipment of slope failure simulation. Theoretically, the parameter values such as cohession (c), internal friction angle (ϕ) and σm (at low confining stress) in this study adopted the ideas that were developed by Diederichs (1999), Hajiabdolmajid et.al. (2003, 2002), Cai et.al. (2004) and Saiang (2008). They stated that at low confining stress the yielding process for brittle rocks or loose materials is governed by a cohesion weakening – friction mobilisation phenomenon (see Figures 1).

Under low confining stress conditions, the most likely failure mechanisms are tensile nature leading to spalling, axial splitting and direct tension (Figure 2). Stacey (1981) in Saiang (2008) states that the in situ strength around excavations is determined by the extension strain capacity of the rock. Under such conditions, the failure can occur at stress levels in which the σ1 (major principle stress) is lower than the σ1s (major principle stress at failure) as long as the σ3 (minor principle stress) overcomes the tensile strength of the rock mass.

According to Rocscience (2001-2004), slope stability analysis is affected by three major aspects. Those are material properties of slope model, influence of calculating safety factor on slope stability and definition of slope failure.

Material Properties

Mohr-Coulomb constitutive model is used to describe soil (or rock) material properties. The Mohr-Coulomb criterion relates shear strength of material to cohesion, normal stress and angle of internal friction of the material. The failure surface of the Mohr-Coulomb model can be presented as equation 1, 2, 3 and 4.

$$f = \frac{I_1}{3}\sin\phi + \sqrt{J_2}\left[\cos\theta - \tfrac{1}{3}\sin\theta\sin\phi\right] - C\cos\phi \quad \text{(eq. 1)}$$

where ϕ is the angle of internal friction and C is cohesion. Others can be calculated using equation :

$$I_1 = (\sigma_1 + \sigma_2 + \sigma_3) = 3\sigma_m \quad \text{(eq. 2)}$$

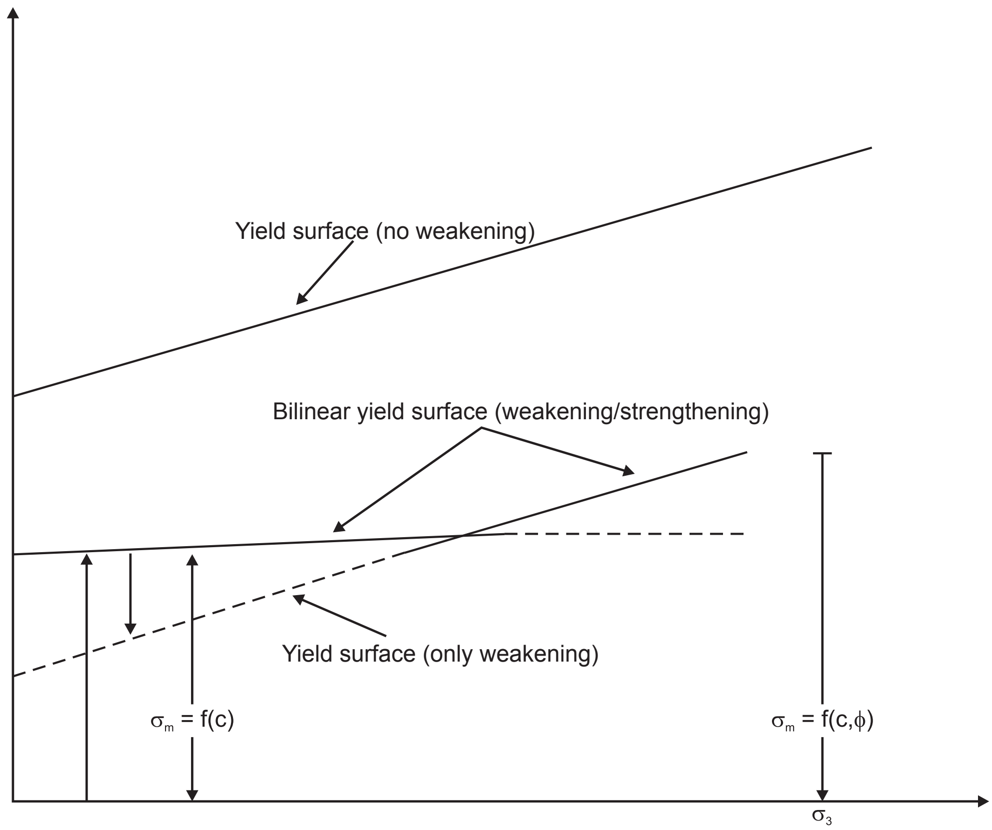

Figure 1. Cohesion weakening – friction mobilisation failure process (Hajiabdolmajid et al, 2003.)

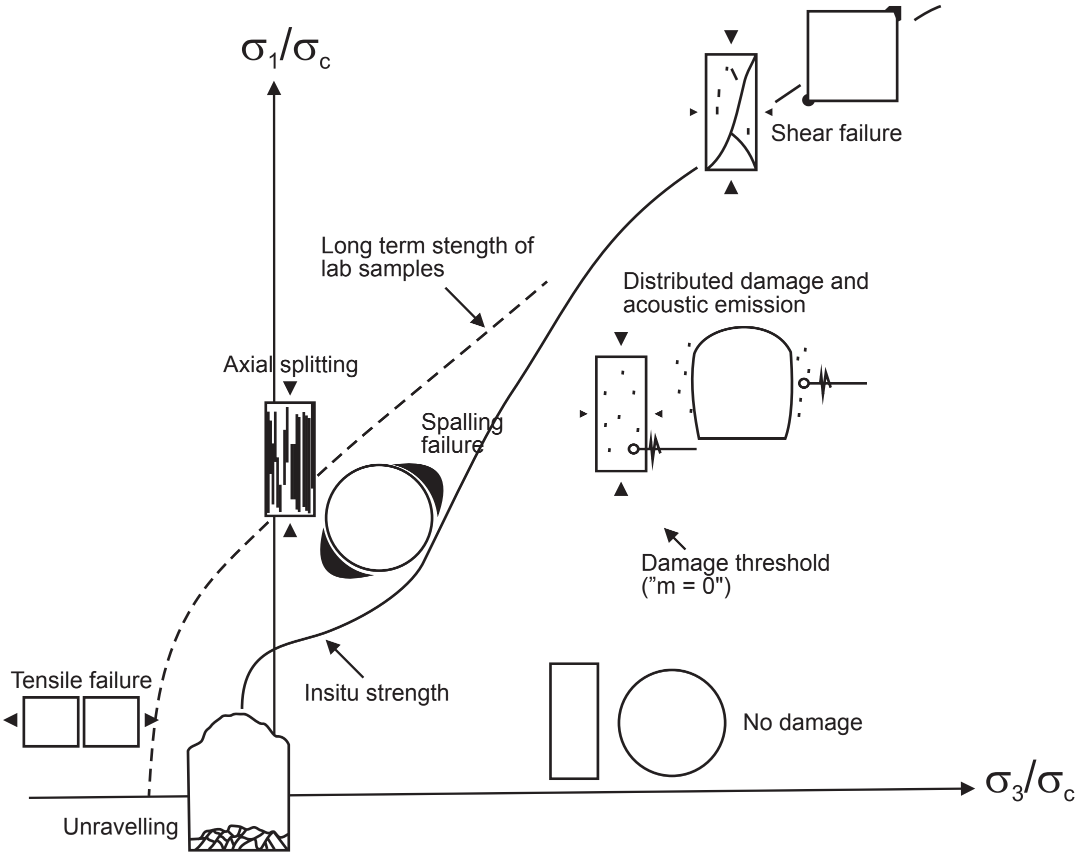

Figure 2. Failure characteristics and mechanisms at different confining stress levels (Diederichs, 1999).

$$J_2 = \left(\tfrac{1}{2}(s_x^2 + s_y^2 + s_x^2) + \tau_{yz}^2 + \tau_{zx}^2\right) \text{...... (eq. 3)}$$

$$\theta = \tfrac{1}{3}\sin^{-1}\left[\frac{3\sqrt{3J_3}}{2J_2^{3/2}}\right] \text{............................. (eq. 4)}$$

where :

$$J_3 = S_x S_y S_z + 2\tau_{xy}\tau_{yz}\tau_{zx} - S_x\tau_{yz}^2 - S_y\tau_{zx}^2 - S_z\tau_{xy}^2,$$
$$S_x = \sigma_x - \sigma_m,\ S_y = \sigma_y - \sigma_m,\ S_z = \sigma_z - \sigma_m$$

Six material properties are required if using Mohr-Coulomb model. These properties are the friction angle ϕ, cohesion C, dilation angle ψ, Young's modulus E, Poisson's ratio ν and unit weight of soil γ.

Safety Factor (SF) and Strength Reduction Factor (SRF)

Slope can be fail if the material shear strength on the sliding surface is insufficient to resist the actual shear stresses. Safety factor is a value to examine the stability state of slopes. SF value is greater than 1, it means that the slope is stable, while the SF is lower than 1, it means that the slope is unstable. In accordance within shear failure, the safety factor against slope failure is simply calculated by equation 5 and 6.

$$SF = \frac{\tau}{\tau_f} = \frac{C + \sigma_n \tan\phi}{C_f + \sigma_n \tan\phi_f} \text{.................. (eq. 5)}$$

$$C_f = \frac{C}{SRF} \quad \text{and} \quad \phi_f = \tan^{-1}\left(\tan\phi / SRF\right) \text{.. (eq. 6)}$$

Where τ is shear strength of slope material, ϕ_f is shear stress on sliding surface, C_f and ϕ_f are factored shear strength parameters and SRF is strength reduction factor. This method has been referred as the 'shear strength reduction method'. To achieve the correct SRF, it is essential to trace the value of SF that will cause slope failure.

Slope Collapse

Non-convergence within a user-specified number of iteration infinite element program is taken as a suitable indicator of slope failure. This actually means that no stress distribution can be achieved to satisfy both the Mohr-Coulomb criterion and global equilibrium. Slope failure and numerical non-convergence take place at the same time and are joined by an increase in the displacements. Usually, the value of the maximum nodal displacement derived just after slope failure, has a big jump if compared to the one before failure.

RESULTS AND DISCUSSION

Test of Physical Properties

Sieve analysis

In this study, sorting of fine and coarse sands had been conducted by sieving the materials to obtain +10, +12, +14 and +20 mesh. Number of items most widely was taken as the material to be modeled. Results of sieve analysis and grain size distribution are shown in Table 1.

Based on such analysis, most grains of quartz sand is concentrated at the grain size of + 20 mesh, while the sand river is concentrated at the grain size of -20 mesh.

Density

This test is intended to determine the weight per volume unit of sand. Weight-volume ratio of the sand is the weight of sand including the water content divided by the total volume of sand. Results of the test can be seen in Table 2. Based on the results, it is found that the densities of fine sand and coarse sand was 1.35 gr/cm^3 and 1.52 gr/cm^3 respectively.

Mechanical Properties Test

Laboratory analysis for mechanical properties of the material included direct shear test. The purpose was to determine shear parameters directly on the condition of Unconsolidated-Undrained

Table 1. Result of size distribution analysis

Grain size (mesh)	Sample weight (gram)							
	S1		S2		S3		Total	
10	193.1	18.9%	334.1	23.1%	584.6	31.1%	1111.8	25.6%
14	643.7	63.2%	910.0	62.9%	1042.5	55.4%	2596.2	59.7%
20	171.8	16.9%	192.9	13.3%	242.6	12.9%	607.3	14.0%
25	10.5	1.0%	10.0	0.7%	12.9	0.7%	33.4	0.8%
Total	1019.1		1447.0		1882.6		4348.7	

Note : S1 = 1st test, S2 = 2nd test and S3 = 3rd test

Table 2. Result of density test

	Notes	Fine (river sand)		Coarse (quarzt sand)	
		Nature	Dry	Nature	Dry
1.	Ring diameter – d (cm)	6.38	6.38	6.38	6.38
2.	Ring height – h (cm)	2.55	2.55	2.55	2.55
3.	Ring volume – V (cm^3)	81.59	81.59	81.59	81.59
4.	Ring weight – W1 (gr)	82.31	82.31	82.31	82.31
5.	Ring weight + wet sand weight – W2 (gr)	194.90	189.86	208.09	205.19
6.	Wet sand weight W2 – W1 (gr)	112.59	107.55	125.78	122.88
7.	Weight volume of sand (density - γ) (gr/cm^3)	1.38	1.32	1.54	1.51
8.	Average density (gr/cm^3)	1.35		1.52	

(UU). Parameters of shear strength were internal friction angle (ϕ) and cohesion (C). The tests on the sand samples were performed three times for 8, 16 and 32 kg of loading. Results of direct shear test for the sand can be seen in Figure 3, while testing result of mechanical properties for quartz- and river sands, especially direct shear test, is shown in Table 3.

Physical and Numerical Modelling

Equipment for slope failure simulation consists of a slope model in the form of two stacks of fine and coarse sands (see Figure 4). The models were connected to a vibration tool that had a freely, horizontally moving bearing .... established path. This tool was associated with a spring that can pull to the side. The spring was attached to the iron frame. The physical model of the slope was placed on a transparent box (acrylic) as seen in Figure 5.

Slope Failure Test

Experimental slope failure had been carried on by varying the distance of the spring track that ranged from 10, 20 to 30 cm and the thickness of quartz sand layer from 2, 4 and 6 cm. The slope has been formed previously in critical condition

Table 3. Result of laboratory test for mechanical properties

Material type	C (kPa)	Phi (o)
Quarzt sand (coarse)	0.03	31.08
River sand (fine)	0.01	28.1

(slope angle of 30°). The physical model and the laboratory test results were used as an input for numerical modelling of the slope failure. Data of physical modelling are presented in Table. 4.

Numerical Modeling

Numerical modelling for slope failure employs Phase 2 software version 6.0 that utilizes finite-element method and shear strength reduction technique to determine the factor of slope safety. Utilizing of these approaches by a standard engineering practice to determine slope stability and is extremely importance for engineering slope design. The material parameters of the slope are given in Table 5. Slope dimensions in numerical modelling are adjusted to the size of the physical model. One of the numerical models can be seen in Figure 5.

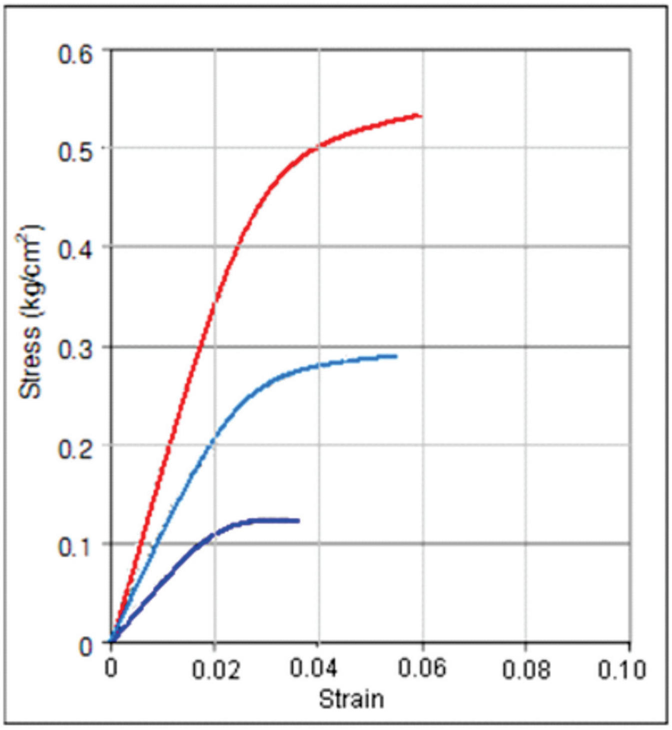

Figure 3. Stress – strain curve of direct shear test result

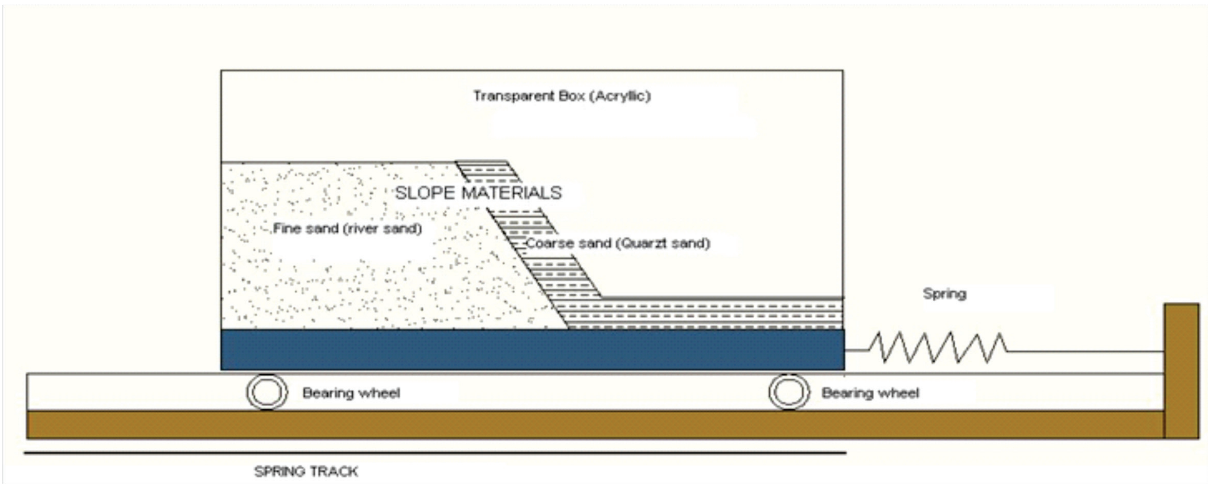

Figure 4. Equipments for physical modelling

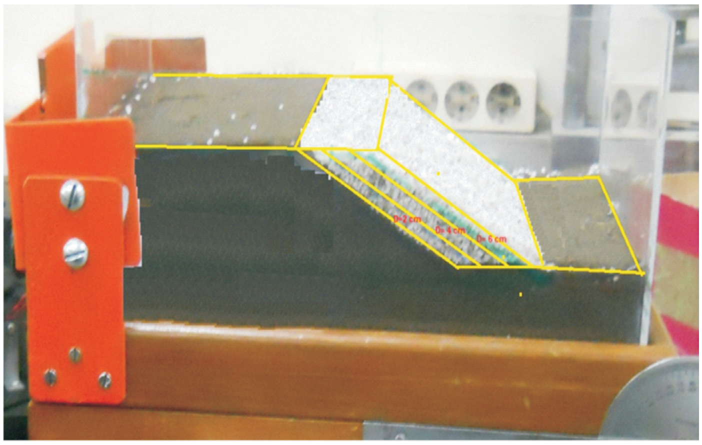

Figure 5. Forms and materials for physical modelling

Table 4. Result of slope failure modelling

D (cm)	H (cm)	t (sec.)	a (cm/sec.2)	Seismic loading coeff. (g)	Height of Slope(cm)	Base (cm)	Width of Slope (cm)	Volume of failure(cm^3)
2	10	0.50	2.5	0.0025	9.0	18	19	342
	20	0.53	5.6	0.0057	9.0	18	19	342
	30	0.54	8.7	0.0088	9.0	18	19	342
4	10	0.49	2.4	0.0024	9.0	18	19	684
	20	0.56	6.3	0.0064	9.0	18	19	684
	30	0.59	10.4	0.0106	9.0	18	19	684
6	10	0.50	2.5	0.0025	9.0	18	19	1026
	20	0.55	6.1	0.0062	9.0	18	19	1026
	30	0.59	10.4	0.0106	9.0	18	19	1026

Notes : D = thickness of quarzt sand layer; H = distance trajectory; t = time; a = acceleration

Table 5. Data input of material properties for numerical model

Material	E (kPa)	ν	γ (gr/cm3)	Unit weight (kN/m3)	ϕ (o)	C (kPa)	Vibration (g)
Fine sand (river sand)	34500	0.3	1.35	13.547	28.10	0.01	0.0025 until 0.0106
Coarse sand (quarzt sand)	10350	0.3	1.52	15.217	31.08	0.03	

Comparison Physical and Numerical Modelling for Slope Failure

The constitutive laws of material behavior such as the Mohr-Coulomb failure criterion that used for both physical and numerical models should have the same behavior. To assess the behavior and accuracy of the algorithm using numerical model, the simulations are performed for some specific parameters and then compared to the physical model. The studied parameters include slope failure behavior and strength reduction factor (SRF). Therefore, the same simulations of physical and numerical models have been carried out based on same input data. The results of those comparisons can be seen in Figures 6, 7 and 8.

Based on numerical modelling analysis, it is known that the decrease of SRF value with increasing the seismic value loading coefficient (g) have occurred. The resume of numerical modelling and the SRF value for each model can be seen in Table 6. It can be explained that there is significant correlation between the thickness of quartz sand layer (D) and the decrease of SRF value as shown in Figure 9. The decrease of SRF value was also influenced by the increase seismic loading coefficient as shown in Figure 10. Meanwhile the distance trajectory (H) affects the g value.

CONCLUSION

Excessive vibration that causes damage to the medium such as mine slope can be modelled both physically and numerically. The model shows that the seismic loading affects rock strength reduction. This condition is indicated by the trend of the SRF decrease if the g value increases. Furthermore, the thickness of critical zone represented by the thickness of quartz sand layer (D) would be more unstable if the zone is thicker. This condition is indicated by the trend of the SRF decrease if the D is increased. It means that thicker critical zone on the mine slope would reduce the rock strength and the mine slope would be unstable.

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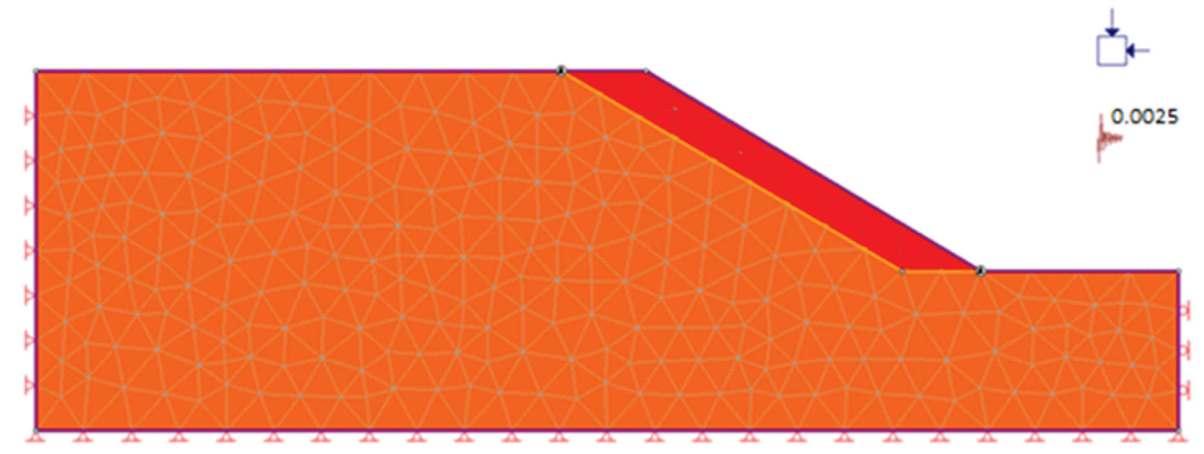


Figure 5. Condition of slope in numerical model

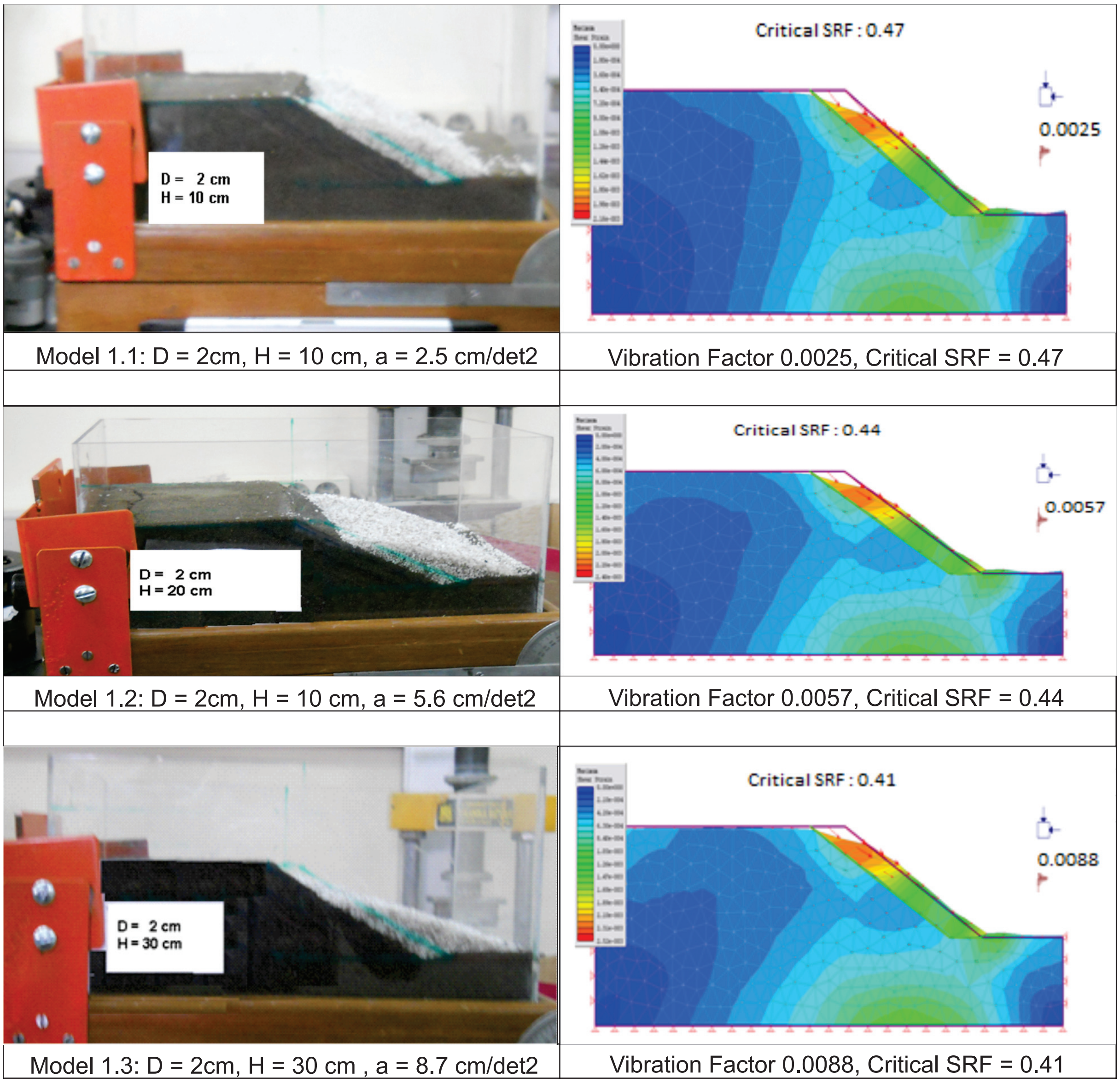


Figure 6. Physical and numerical modelling for the thickness failure of 2 cm

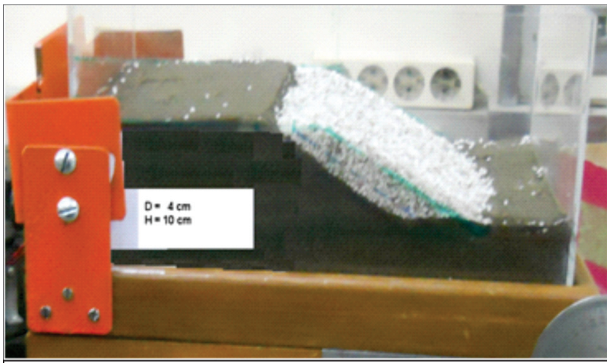
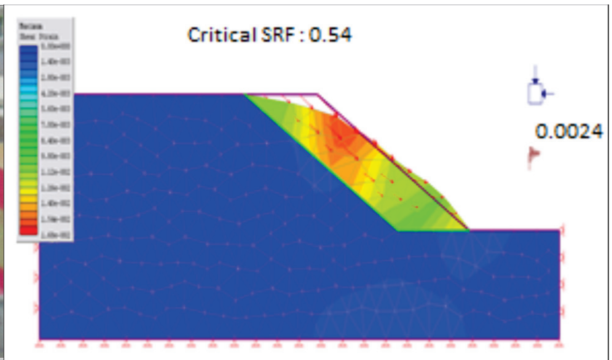
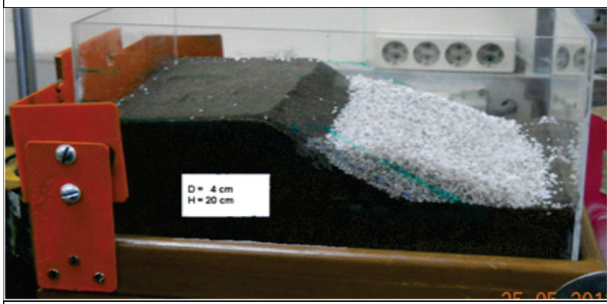
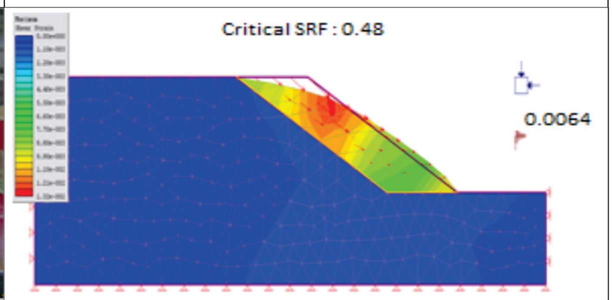
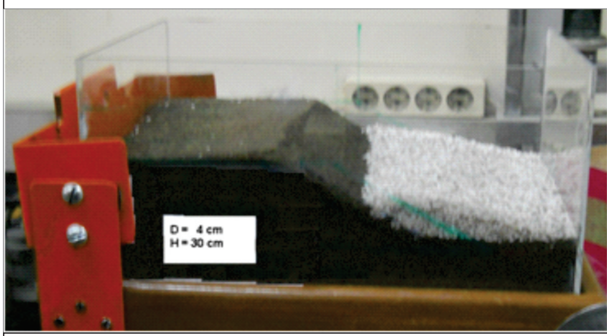
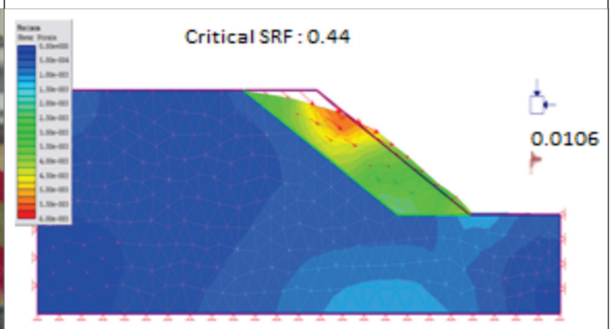
	
Model 2.1: D = 4 cm, H = 10 cm, a = 2.4 cm/det2	Vibration Factor 0.0024, Critical SRF = 0.54
	
Model 2.2: D = 4 cm, H = 10 cm, a = 6.3 cm/det2	Vibration Factor 0.0064, Critical SRF = 0.48
	
Model 2.3: D = 4 cm, H = 30 cm , a = 10.4 cm/det2	Vibration Factor 0.0106, Critical SRF = 0.44

Figure 7. Physical and numerical modelling for the thickness failure of 4 cm

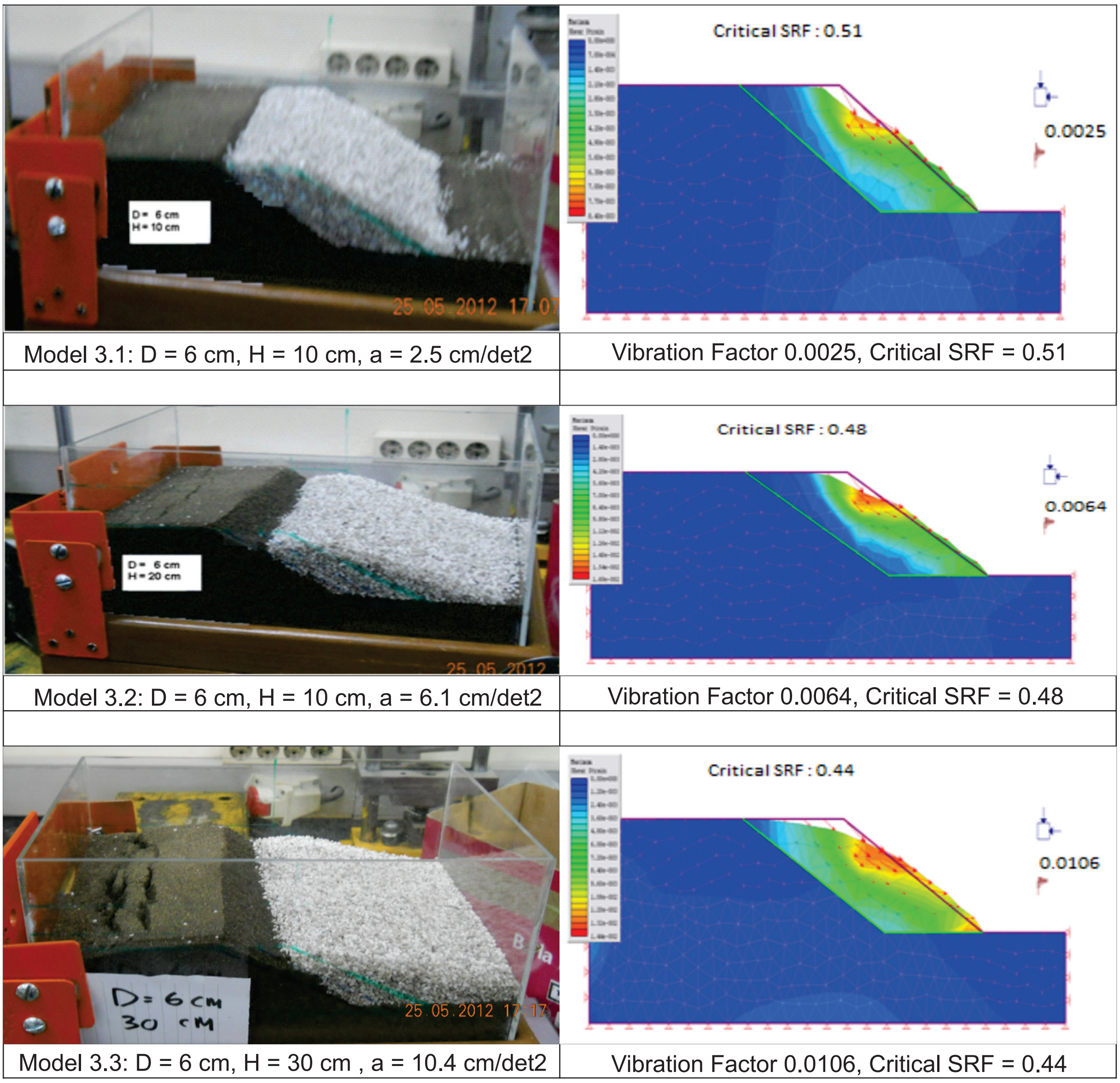


Figure 8. Physical and numerical modelling for the thickness failure of 6 cm

Table 6. Resume of numerical modelling

D (cm)	H (cm)	Seismic loading coeff. g	SRF
2	10	0.0025	0.47
2	20	0.0057	0.44
2	30	0.0088	0.41
4	10	0.0024	0.54
4	20	0.0064	0.48
4	30	0.0106	0.44
6	10	0.0025	0.51
6	20	0.0064	0.48
6	30	0.0106	0.44

Notes : D = thickness of quarzt sand layer;
H = distance trajectory; t = time;
g = acceleration

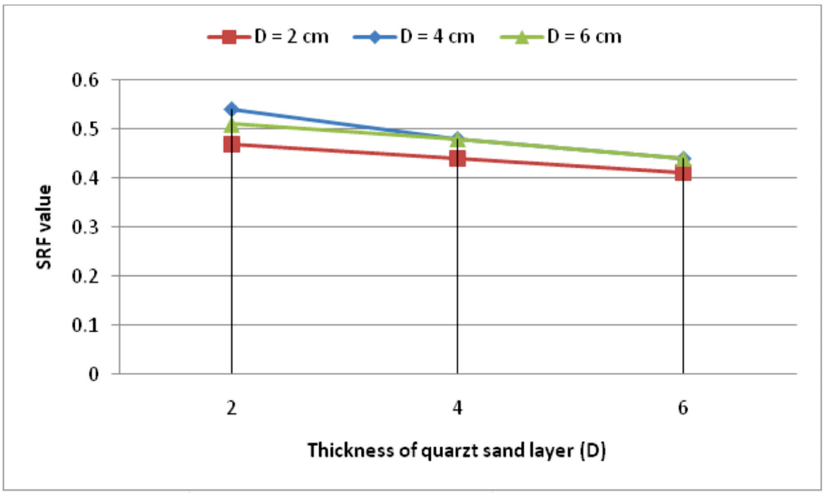


Figure 9. Correlation between the thickness of quarzt sand layer and SRF value

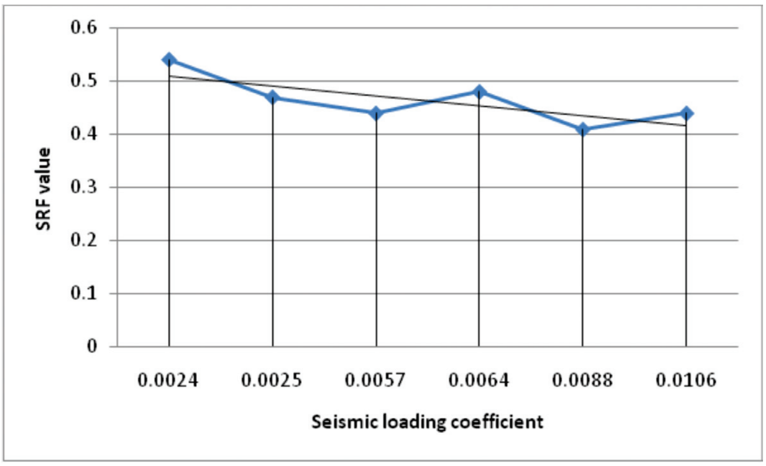


Figure 10. Correlation between seismic loading coefficient (g) and SRF value

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