

An experimental study on rock slopes against flexural toppling failure under dynamic loading and some theoretical considerations for its stability assessment

Ömer AYDAN^{*1} and Mehdi AMINI^{*2}

Abstract

Flexural toppling failure is one of common failure forms of slopes in layered rock mass. The flexural toppling failure can occur due to bending of layers with or without cross joints. The active flexural toppling failure is well known and it occurs under mainly gravitational loading with or without water pressure and seismic loading. However, the passive flexural toppling, which is wrongly named as creep failure by geologists and geomorphologists as pointed out by Aydan *et al.* (1988), can also occur when a overload surcharge load resulting from superstructures, weathered rock or glaciers and seismic load acts. However, the effect of seismic loads is expected to be more prominent for inducing the passive flexural toppling failure. In this paper, first, a brief review of current studies on flexural toppling failure is presented. Then, the results of experiments of single column and rock slopes with potential of flexural toppling failure are explained. Finally, some theoretical considerations on the stability assessment against flexural toppling failure are presented and discussed. It seems that the existing formulations for stability assessment may be used with due modifications.

Keywords: flexural toppling, passive failure, earthquake, layered, rock slope

1. INTRODUCTION

Flexural toppling failure is one of the most probable instability of rock slopes and underground openings (Figure 1). This failure occurs due to bending stress. The toppling failure was first mentioned by Muller (1968) in literature. Several articles were published on the toppling failure using physical model tests in laboratory and case histories from 1968 till 1976 (Erguvanli &, Goodman, 1970; De Freitas and Watters, 1973; Bukovansky *et al.* 1974; Hoffman, 1974). Goodman and Bray (1976) classified the toppling failure into four principal types: flexural, blocky, blocky-flexural and secondary. Also, they suggested some kinematic simple methods for preliminary stability analysis of slopes against block toppling failure.

Aydan (Aydan and Kawamoto, 1987, 1992; Aydan *et*

al. 1988) was first to present a complete theoretical method to analyze slopes and underground openings prone to flexural toppling failures, on the basis of limiting equilibrium method utilizing the bending theory of cantilever beam with the consideration of gravity, earthquake and water pressure. Adhikary *et al.* (1997, 2007) tried another approach based on the Cosserat type modeling and centrifuge model tests. However, the Cosserat model is incapable of taking into account the interlayer slip between layers. Amini *et al.* (2008) proposed a procedure based on the displacement compatibility of layers. Since this model does not allow slip between layers, this method may yield very low safety factors for short layers near the slope toe, which may only be true for small deflections. Also, Discrete Element Method (DEM), Discrete Finite Element Method (DFEM) and Finite Element Method (FEM) were used to model flexural toppling failure of slopes (i.e. Pritchard (1989); Aydan *et al.* 1992; Adhikary *et al.* 1997).

2009年3月13日受理

*1 東海大学海洋学部建設工学科, 静岡 (Tokai University, School of Marine Civil Engineering, Shizuoka, Japan)

*2 Tehran University, School of Mining Engineering, Tehran, Iran



(a) Galandrood mine (Iran) (b) Ergenli-Bayindir (Turkey) (c) Oya-kuzure (Japan)

Figure 1. Examples of active flexural toppling failure of rock slopes

However, all these methods considered the active mode of failure under gravitational load except Aydan and Kawamoto (1992). Aydan (1987) proposed first theoretical treatment for active and passive modes sliding and toppling failures of a single block and discontinuous rock slopes. These theoretical relations were later published in a paper by Shimizu *et al.* (1989). Aydan presented a first example of passive flexural toppling of rock slope nearby Ergenli-Bayindir (Turkey) within Menderes metamorphic massive in one of his articles on flexural toppling failure and his doctorate thesis (Aydan *et al.*, 1988; Aydan, 1989) (Figure 2).

In this article, the results of physical modeling of flexural toppling failure of single column and slopes are described considering active and passive modes under dynamic loading. In addition, some considerations for the assessment of single column and rock slopes against

flexural toppling failure under active and passive modes are presented and some theoretical estimation compared with experimental results.

2. PHYSICAL MODEL MATERIALS

Columns (layers) used for physical models of a single column and layered rock slopes were created through the consolidation of a special mixture consisting of BaSO₄, ZnO and Vaseline oil under a chosen pressure (Fig. 3). Various researchers determined the properties of this solid material (Egger, 1983; Kawamoto, 1983; Aydan and Kawamoto, 1987; Aydan *et al.*, 1988; Aydan and Kawamoto, 1992). In this experimental study, some new samples prepared under consolidation pressures of 600 and 800 kPa. Frictional angle, unit weight, tensile and compressive strength of these samples have been measured in the laboratory. The results of these tests



(a) Akamizu waterfall (Japan) (b) Ergenli-Bayindir (Aydan 1989)

Figure 2. Examples of passive flexural toppling failure of rock slopes



Figure 3. Material for the physical modeling before and after consolidation

are presented in the following sub-sections.

2.1 Uniaxial Compression Experiments

The test was done to measure the uniaxial compressive strength (UCS) of the solid material (Fig. 4). The shapes of the samples were cubic and their length to width (or thickness) ratio (L/t) was around 2. The results of these tests for 600 and 800 kPa consolidation pressures have been shown in Figure 5. As it can be seen from the figure, UCS of samples with 600 and 800 kPa consolidation pressures ranges between 50–110 kPa and 70–110 kPa, respectively.

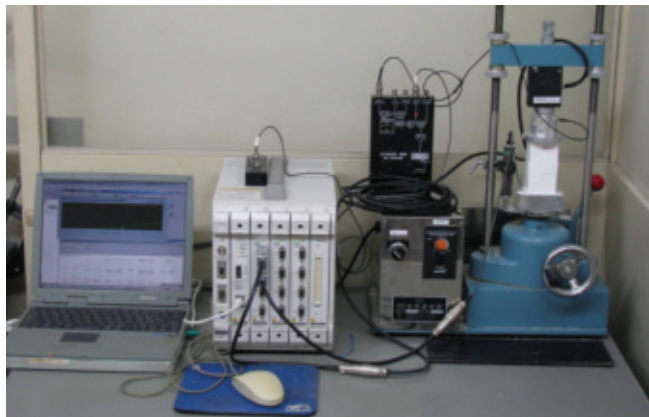
2.2 Three and Four Point Bending Experiments

This test was done to measure the tensile strength of the solid blocks (Fig. 6). The consolidation pressure

of these samples was 800 kPa and the length, thickness and width of all sample were selected 10, 5, 4.5 cm, respectively. For testing of these beams, 3 and 4 points bending test methods were used. The results of these tests have been presented in Fig. 6(a). As it can be seen from this figure, range of the tensile strength of these beams is 36–57 kPa. If these results are compared with uniaxial compressive strength of these materials (70–110 kPa), that the results were somewhat very high. It is thought that this might be caused by arching phenomenon developing during experiments.

2.3 Cantilever Beam Experiments

Since tensile strength obtained from bending experiments was unexpectedly high, a specially designed cantilever beam experimental set-up was used to measure



(a) Compression test set-up



(b) 3 point bending test set-up

Figure 4. Compression and bending test set-ups

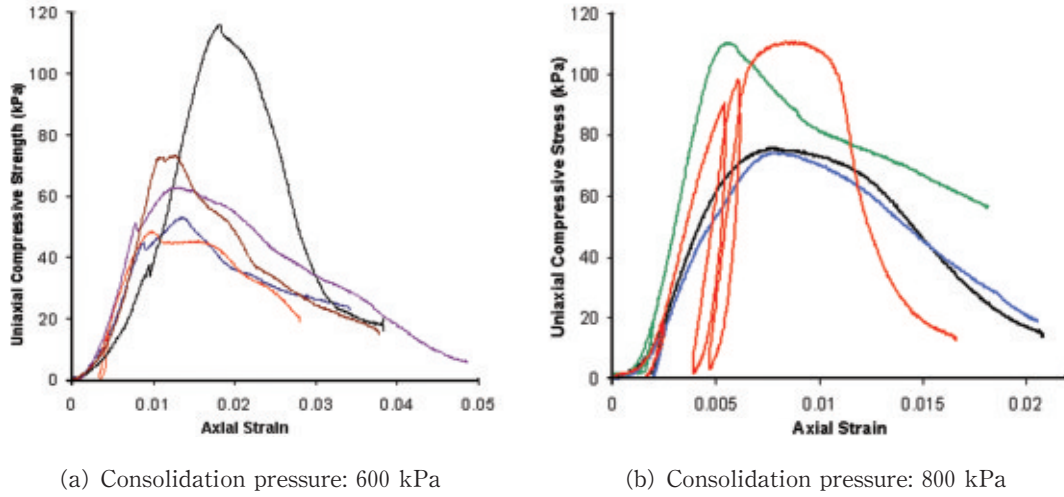
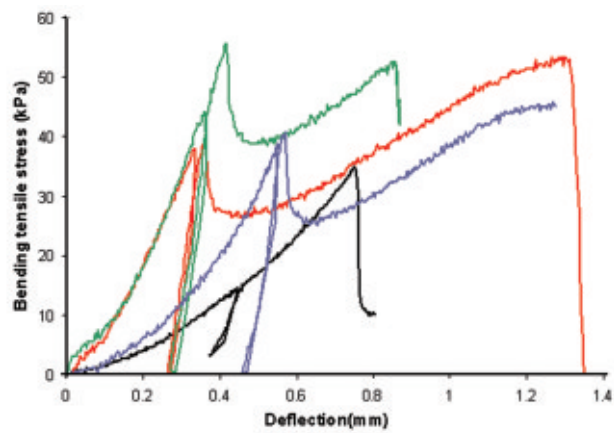


Figure 5. Uniaxial compressive stress-strain relations



(a) Experimental set-up



(b) Deflection-bending stress responses

Figure 6. Experimental set-up and deflection-bending stress responses

the tensile strength of the solid materials (Fig. 7(a)). The test results have been shown in Fig. 7(b) for 600 kPa consolidation pressure. Load on samples was increased through the increase of water level in a water tank with an internal diameter of 60mm and dead weight of 230gr. The range of tensile strength of beams is 15–35 kPa and for 600 kPa consolidation pressure.

2.4 Inter-layer Shear Properties

Inter-layer shear properties were measured through tilting tests and stick-slip tests (Figure 8). Both experiments can yield static and kinetic friction properties when cohesion is not present. However, displacement responses should be measured to obtain the kinetic friction coefficient. The frictional angle between the solid blocks obtained from tilting tests ranges 35 and 42 degree. The results of these tests have been presented in

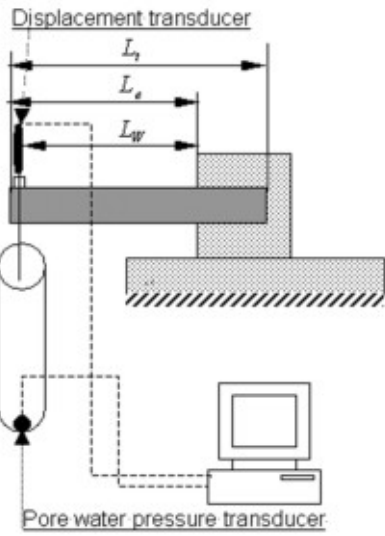
Figure 9. As it can be seen from these figures, range of inter-layer friction angle is 38–44 degree, which is remarkably similar to tilting test results.

2.5 Unit Weight

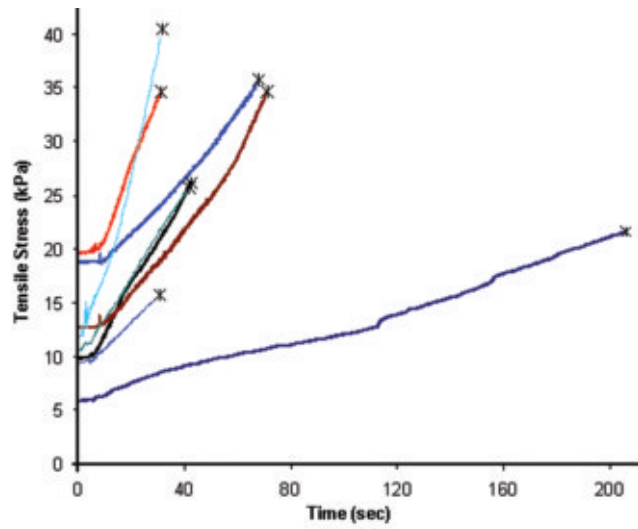
Unit weights of the solid blocks are related to applied consolidation pressure. Strength properties of solid blocks vary with the applied consolidation pressure. The unit weights of all samples before tensile and compressive tests, were determined and it is found that it ranges between 17 and 22 kN/m^3 .

2.6 Estimation of Material Properties

As mentioned, unit weight, tensile and compressive strengths of the solid materials used in this study are inter-related and they depend on the consolidation pressure. Some relations are established among these param-

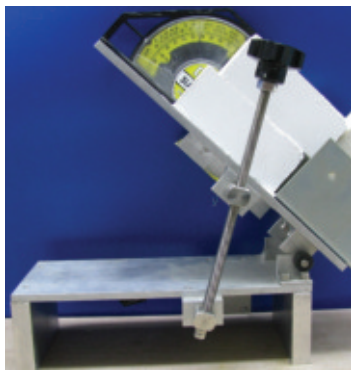


(a) Experimental set-up

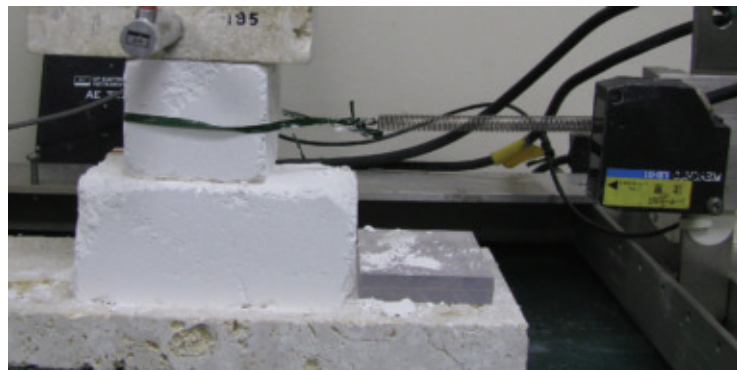


(b) time-tensile stress responses

Figure 7. Experimental set-up and time-tensile stress responses (CP: 600 kPa)

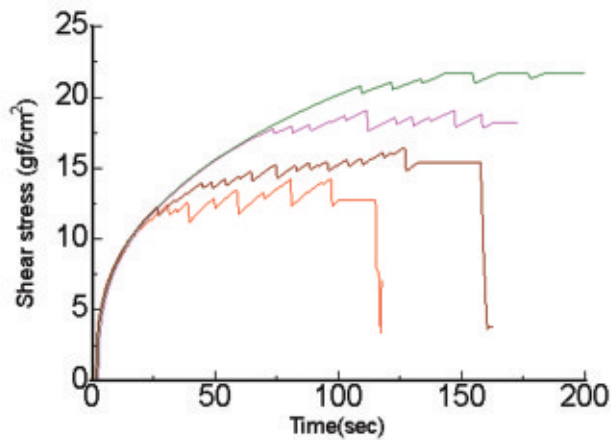


(a) Tilting test set-up

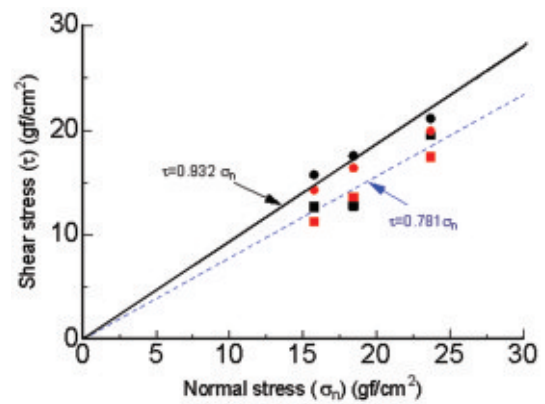


(b) Stick-slip test set-up

Figure 8. Tilting and stick-slip test set-ups



(a) Time-shear stress



(b) Shear stress vs. normal stress

Figure 9. Response and shear strength characteristics obtained from stick-slip tests

eters and consolidation pressure. The properties of physical model materials can be chosen by varying consolidation pressure. Some empirical relations between unit weight, tensile and compressive strengths of the solid materials relations are shown in Figure 10.

3. PHYSICAL MODEL EXPERIMENTS

As was mentioned in abstract, flexural toppling failure occurs due to bending of rock layers. When layers dip into mountain side, body force of the rock layers and water pressure are the primary cause of bending (Figure 11(a)). This situation corresponds to the well-known flexural toppling failure. However, this flexural toppling failure mode is called the active flexural toppling failure since the vertical motion of layers will be in the same direction of gravity force. On the other hand, if the layers dip into slope and the inclination of layers greater than slope angle, flexural toppling failure of layers can not be induced under body force of rock layers and water pressure (Figure 11(b)). If dynamic force resulting from earthquakes or blasting and/or surcharge load by weathered rock or soil or glaciers act on layers, flexural toppling failure may be induced. This flexural toppling failure will be called as passive flexural

toppling failure as the initial vertical motion will be opposite to the direction of gravity.

Before explaining experiments on model rock slopes, the experimental investigation on the response and stability of a single column under dynamic loading condition should provide essential data on the governing mechanism of flexural toppling failure in relation to layer inclination and dynamic conditions for failure. Therefore, a series of model tests on a single column was first carried out and then model tests on slopes were performed. The details are given in the following sub-sections:

3.1 Experiments on Single Column Under Dynamic Loading

For physical modeling of a single column, some samples were prepared using a consolidation pressure of 600 kPa. The selected inclinations were 60°, 90° and 120°. The end of samples was firmly fixed on the shaking table (Figure 12). A supporting base for single column was prepared in the case of the inclination of 60° (Figure 12(b)). During the test, the acceleration of the shaking table, deformation of the sample and crack initiation were monitored by accelerometer, laser transducers and

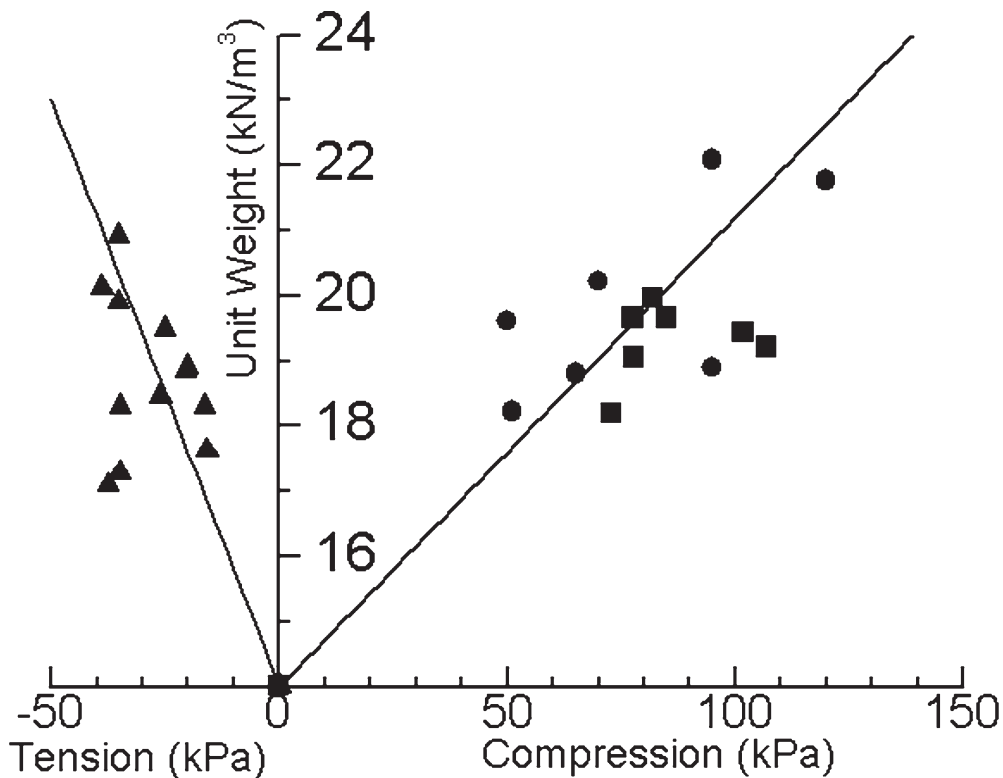


Figure 10. Relations between unit weight and tensile and compression strength

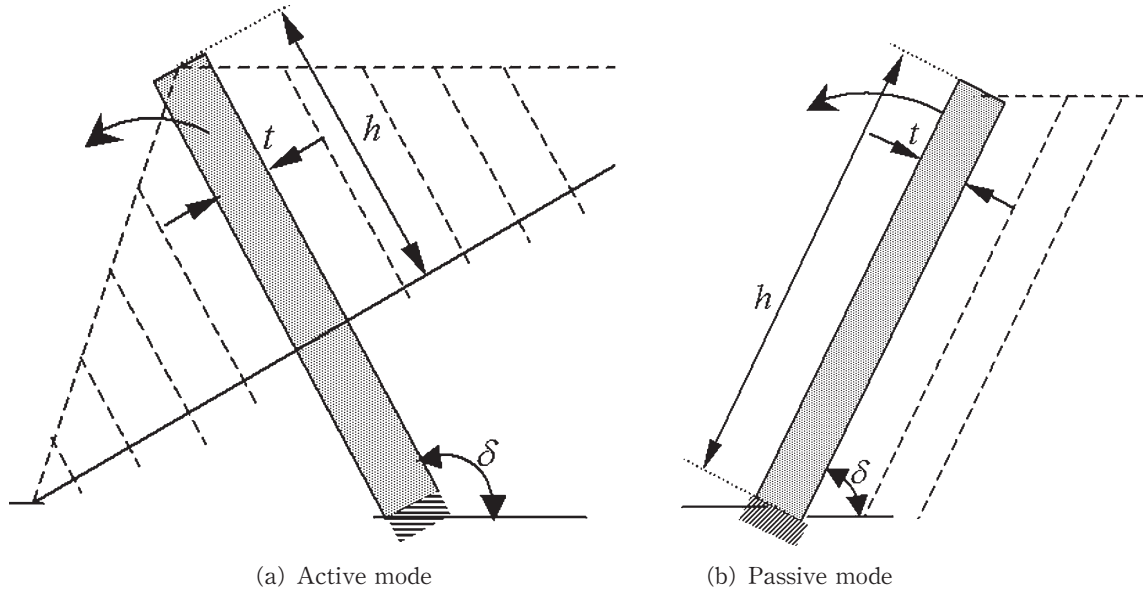


Figure 11. Illustration of slopes subjected to active or passive flexural toppling modes

AE (acoustic emission) sensors, respectively. The acceleration of shaking table was gradually increased up to the maximum capacity of the shaking table or until failure occurs (Figure 13). Figure 14 shows views of samples before and after experiments.

3.2 Experiments on Model Rock Slopes Under Dynamic Loading

The instability of single column and rock slopes differs from each other due to interaction between layers and slope geometry. To investigate the failure mechanism of rock slopes against flexural toppling failure under dynamic condition, a series of physical models on rock slopes were carried out. For these experiments, a rigid frame was strongly fixed on the shaking table and models of layered rock slopes were built in the frame by continuous columns (Figure 15). Before shaking models up to failure, the response of models under dynamic loads with varying frequency were measured under constant acceleration (100-200 gals) as it can be seen in Figure 16. As seen from this example, the acceleration responses differ from place to place and the maximum acceleration is observed at the crest of the slope. The results also implies the existence of natural frequency of slopes. Then, the acceleration waves of shaking table with a frequency range between 3 to 5 Hz were applied while gradually increasing their amplitude up to the maximum capacity of the shaking tables or until failure occurs (Figure 17). The acceleration of the shaking table, deformation of the sample and crack initiation were monitored by accelerometer, laser

transducer and AE (acoustic omission) sensors, respectively during each experiment (Figure 18). One can easily notice that AE activities start long before cracking induced is detected by laser transducers. The inclinations of the layers were selected as 45°, 60°, 90°, 120° and 135°. Since the maximum length of the columns is 35 cm due to handling limitation, the height of the models was less than 25 cm.

4. RESULTS AND CONSIDERATIONS

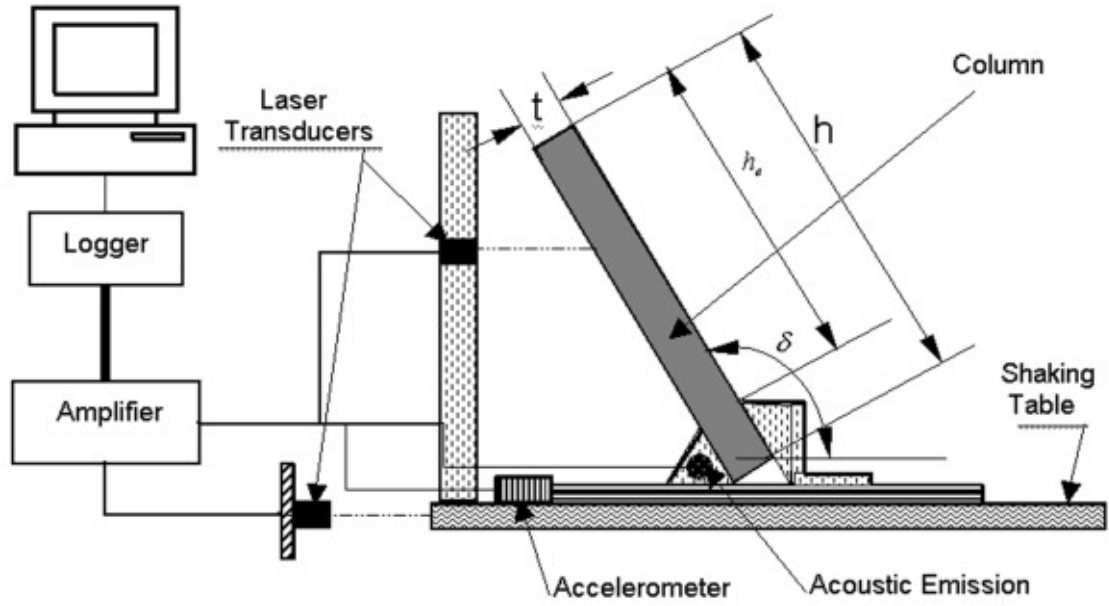
4.1 Single Column Experiments

As mentioned in the previous section, a number of experiments were carried out on single column by changing the column inclination and column height. The column inclinations were 60, 90 and 120 degrees. Experimental results for each column inclination and different column heights are presented in Tables 1-3. The stability condition of single column can be easily derived using the seismic coefficient method. Aydan and Kawamoto (1992) have already proposed such a method for flexural toppling failure and this method can be easily simplified for a single column with due consideration of failure mechanism in the following form:

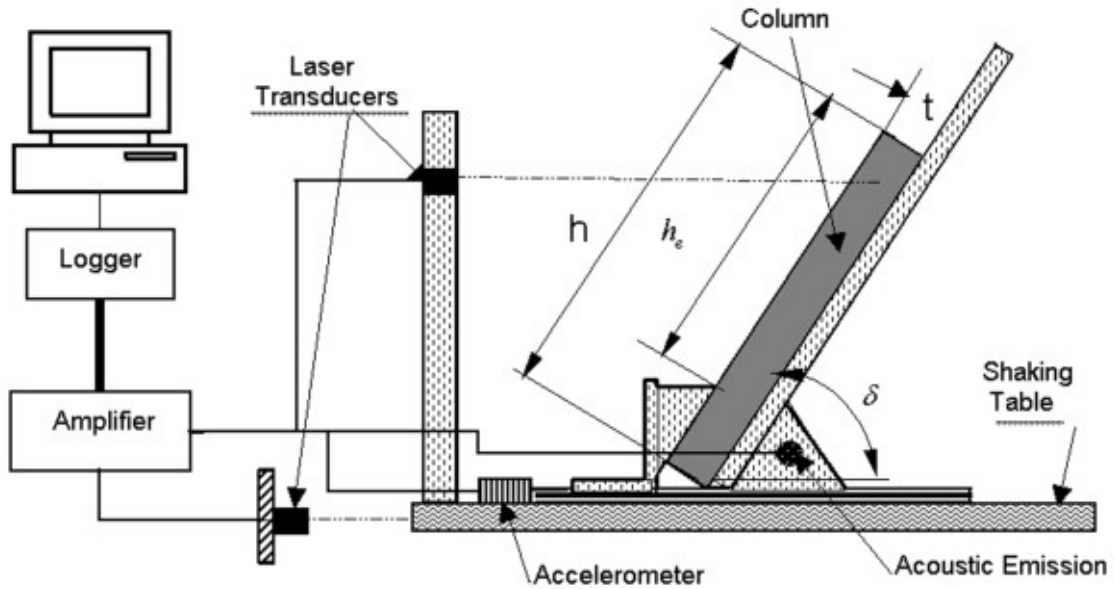
$$F_s = \frac{\sigma_t}{\gamma h} \frac{1}{\eta \left(3 \frac{h}{t} \cdot \sin(\alpha + \beta) - \cos(\alpha + \beta) \right) - \left(\sin \alpha + 3 \frac{h}{t} \cdot \cos \alpha \right)}$$

Where

- η : seismic load coefficient
- α : inclination of column.
- β : inclination of the seismic force



(a) Set-up for Active flexural toppling mode



(b) Set-up for passive flexural toppling mode

Figure 12. Experimental set-ups for active and passive flexural toppling modes

h : effective length of the single column

γ : unit weight of column material

F_s : factor of safety.

σ_t : tensile strength of rock columns

t : single column thickness

Figures 19 shows a comparison of experimental results for different column heights while tensile strength of columns were fixed to 15 and 35 kPa on the basis of outcomes of cantilever tests presented in Section 2. As seen from these figures, good correlations between theoretical and experimental results are found.

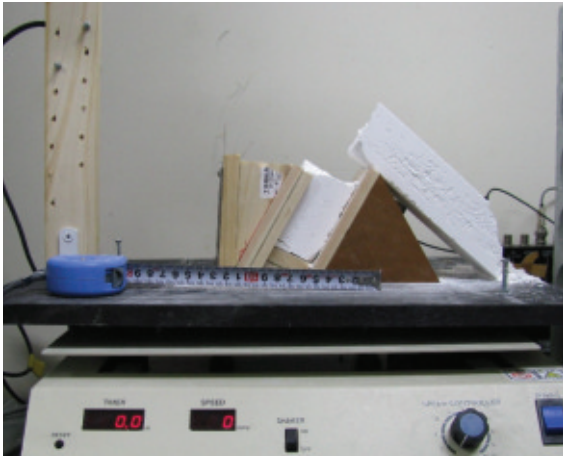
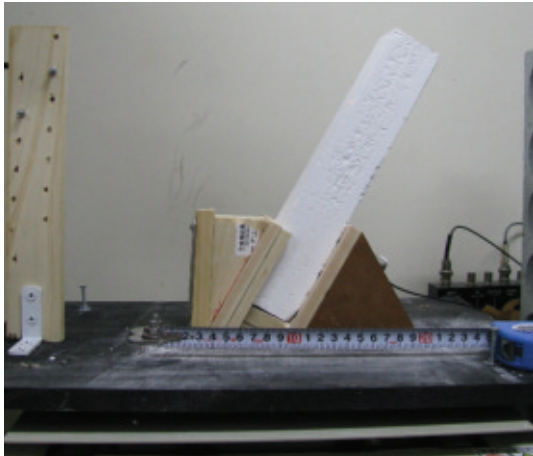
This implies that the proposed approach can also be used for flexural toppling failures of single column under dynamic loading conditions.

4.2 Model Slope Experiments

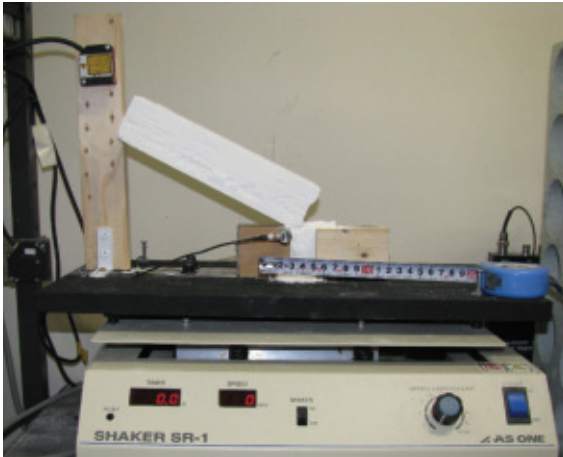
Model slope experiments have not been completed yet. The preliminary experimental results are given in Table 4. As noted Table 4, the acceleration levels to initiate flexural toppling failure differ depending upon the slope angle, layer inclination and slope height. The general tendency of experimental results is quite similar

Before

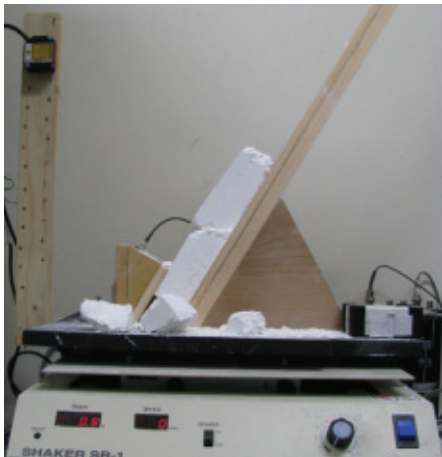
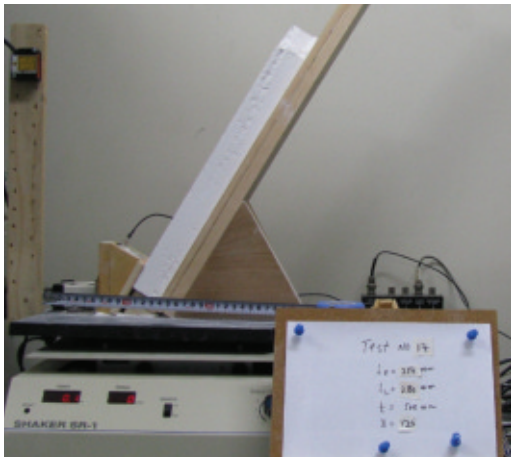
After



(a) 120°



(b) 90°



(c) 60°

Figure 13. Views of a single column before and after experiments

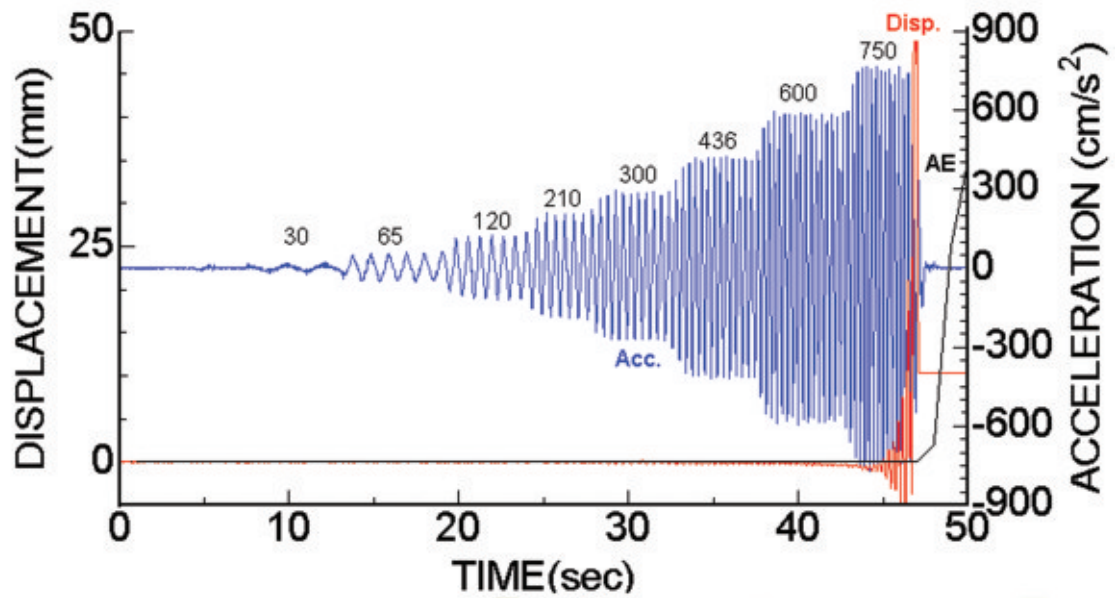


Figure 14. Applied acceleration and displacement and AE response for a single column

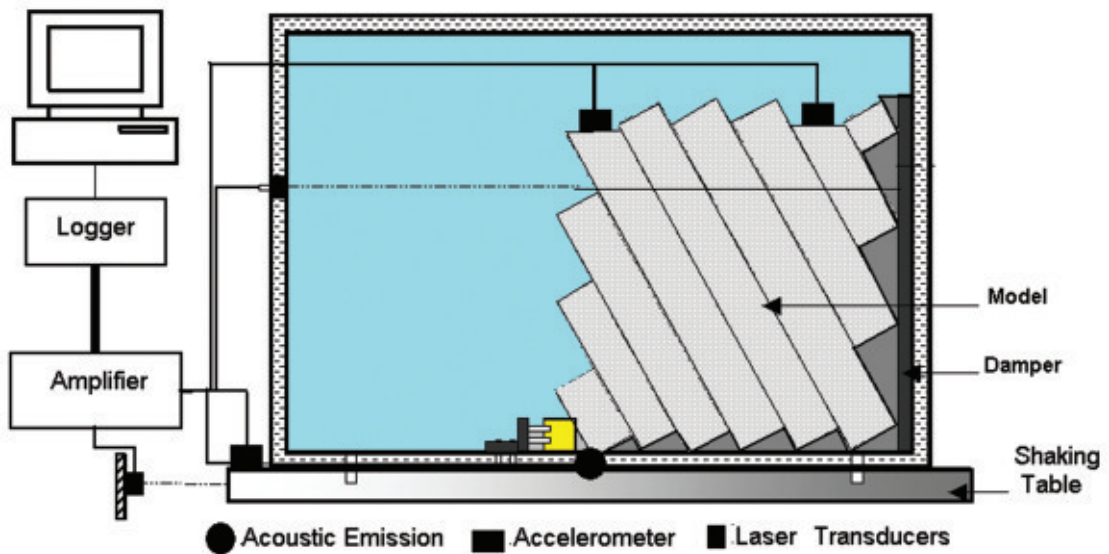


Figure 15. Experimental set-up for model rock slopes

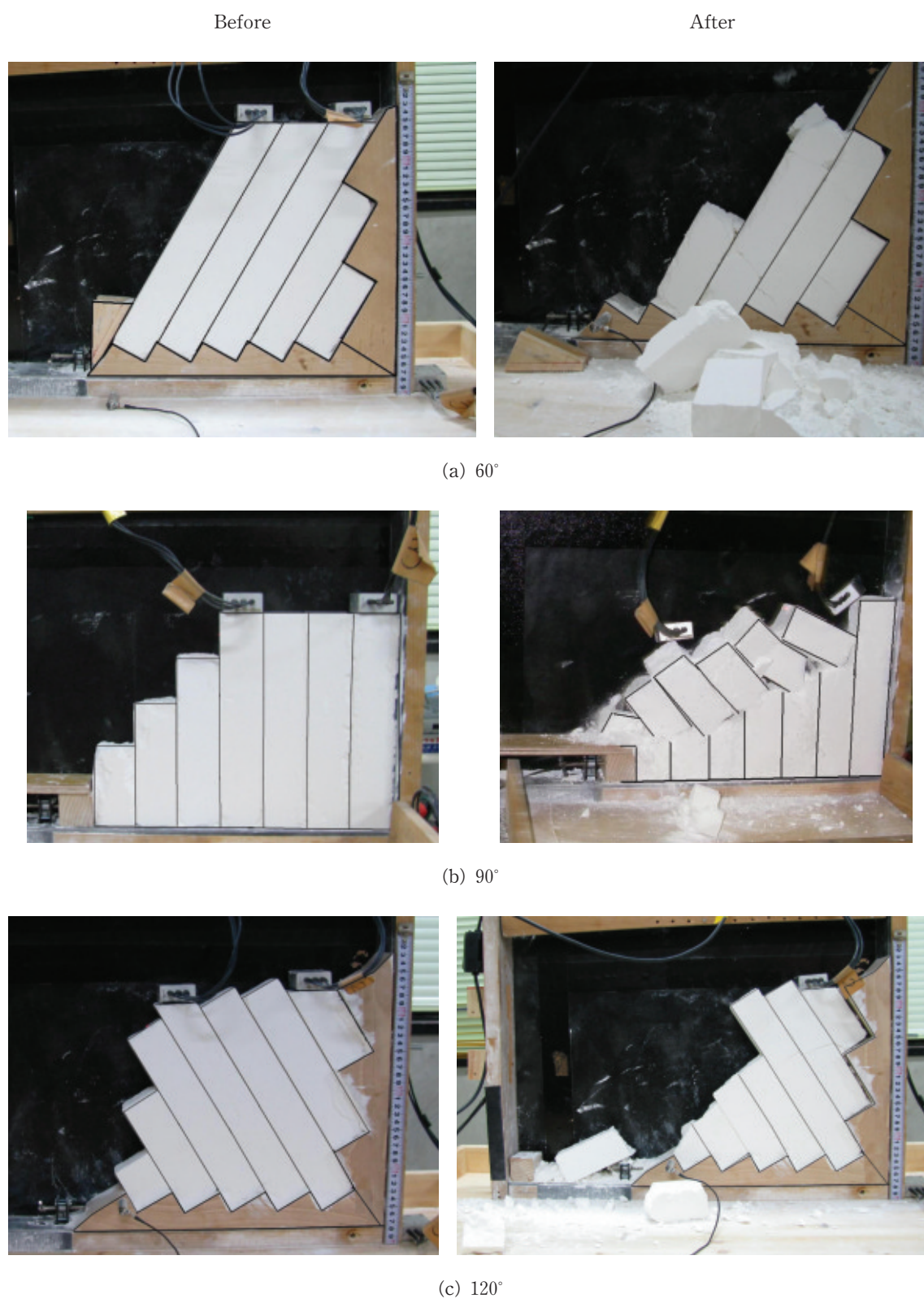


Figure 16. Views of a model slopes before and after experiments

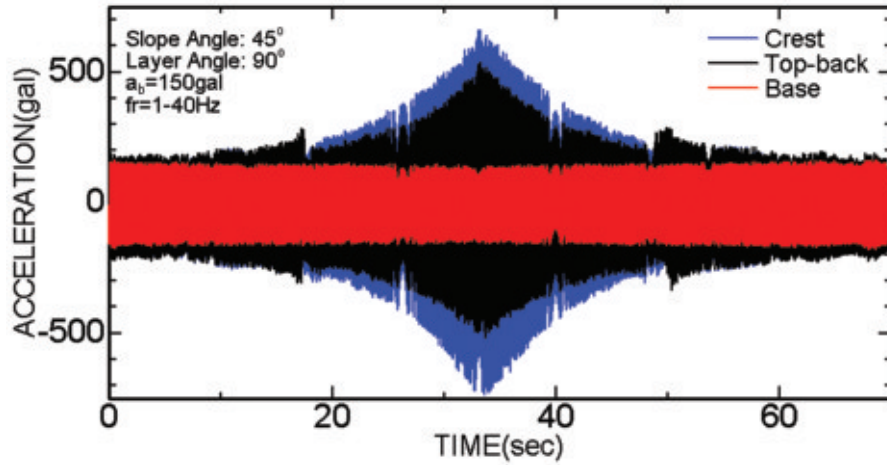


Figure 17. An example of sweeping frequency test (1-40Hz) for a model slope

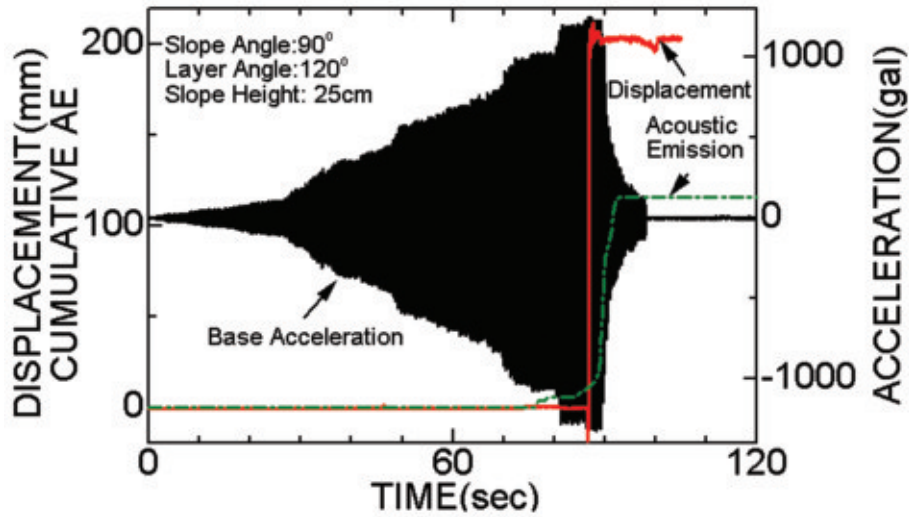


Figure 18. An example of base acceleration and displacement and AE response of model rock slope

Table 1: Test results of single column with 60 degree inclination (passive mode)

Parameters	Total length of the model	Thickness of the model	Width of the model	Distance between the monitoring point and the base	Effective length of the model	Maximum acceleration coefficient	Test results
Number of sample	$h_T (cm)$	$t (cm)$	$D (cm)$	$h_m (cm)$	$h_e (cm)$	(a/g)	
1	50	4.6	5	33	47	0.77	Unstable
2	39	4.5	5	28	36	0.75	Unstable
3	36.5	4.6	5	32	33.5	0.72	Unstable
4	25	4.6	5	19	22	1.21	Unstable
5	21	5	5	19	18	1.4	Unstable
6	20	4.7	5	17	16	1.6	Unstable
7	20	4.6	5	17	17	0.8	Stable
8	25	4.8	5	18	21	0.87	Stable
9	25	5	5	18	23.5	0.88	Stable
10	25	4.5	5	18	22	0.83	Stable

Table 2: Test results of single column with 90 degree inclination

Parameters Number of sample	Total length of the model $h_T(cm)$	Thickness of the model $t(cm)$	Width of the model $D(cm)$	Distance between the monitoring point and the base $h_m(cm)$	Effective length of the model $h_e(cm)$	Maximum acceleration coefficient (a/g)	Test results
1	25	4.6	5	22.5	20	0.32	Unstable
2	25	4.5	5	20.5	20	0.32	Unstable
3	20	4.5	5	18	15	0.33	Unstable
4	20	4.5	5	18	15	0.36	Unstable
5	12	4.5	5	9.2	10.5	0.75	Unstable
6	12	4.8	5	11	7	0.88	Stable
7	12	4.5	5	9.9	9.2	0.87	Stable

Table 3: Test results of single column with 120 degree inclination

Parameters Number of sample	Total length of the model $h_T(cm)$	Thickness of the model $t(cm)$	Width of the model $D(cm)$	Distance between the monitoring point and the base $h_m(cm)$	Effective length of the model $h_e(cm)$	Maximum acceleration coefficient (a/g)	Test results
1	20	4.5	5	19	13	0.87	Unstable
2	25	4.6	5	20.5	18	0.44	Unstable
3	25	4.7	5	21.5	18	0.43	Unstable
4	25	4.6	5	17	18	0.406	Unstable
5	18	4.5	5	15	11	0.9	Stable

to those for single column experiments. Aydan and Kawamoto (1992) have already proposed a method for analyzing the flexural toppling failure under dynamic loading condition and it can be easily used for active mode of flexural toppling failure. Therefore, the available limiting equilibrium methods based on the bending theory of cantilever methods (i.e. Aydan and Kawamoto 1992; Amini *et al.* 2008) may be used for analyzing the experimental results in this study. Nevertheless, the most critical issue with these methods is how to designate the inclination of failure plane, above which layers are subjected to bending. Aydan and Kawamoto (1987, 1992) suggested the failure plane should be equal to the normal to layers for active flexural toppling failure. This proposal was confirmed by experiments. The experiments presented in this study showed that inclination of the failure plane ranges between 0 and 15 degree

above normal to discontinuities for active flexural toppling failure under dynamic condition. However, the present experiments are still insufficient to make a proposal for the inclination of failure plane of passive mode flexural toppling failure under dynamic condition. Therefore, the authors will refrain themselves to compare the experimental results with theoretical estimation in this article. Nevertheless, the authors plan to carry out such comparisons in a following research work on flexural toppling failure of rock slopes under dynamic loading.

5. CONCLUSIONS

Although the experiments on model slopes still continue, the preliminary outcomes of this experimental research may be summarized as follows:

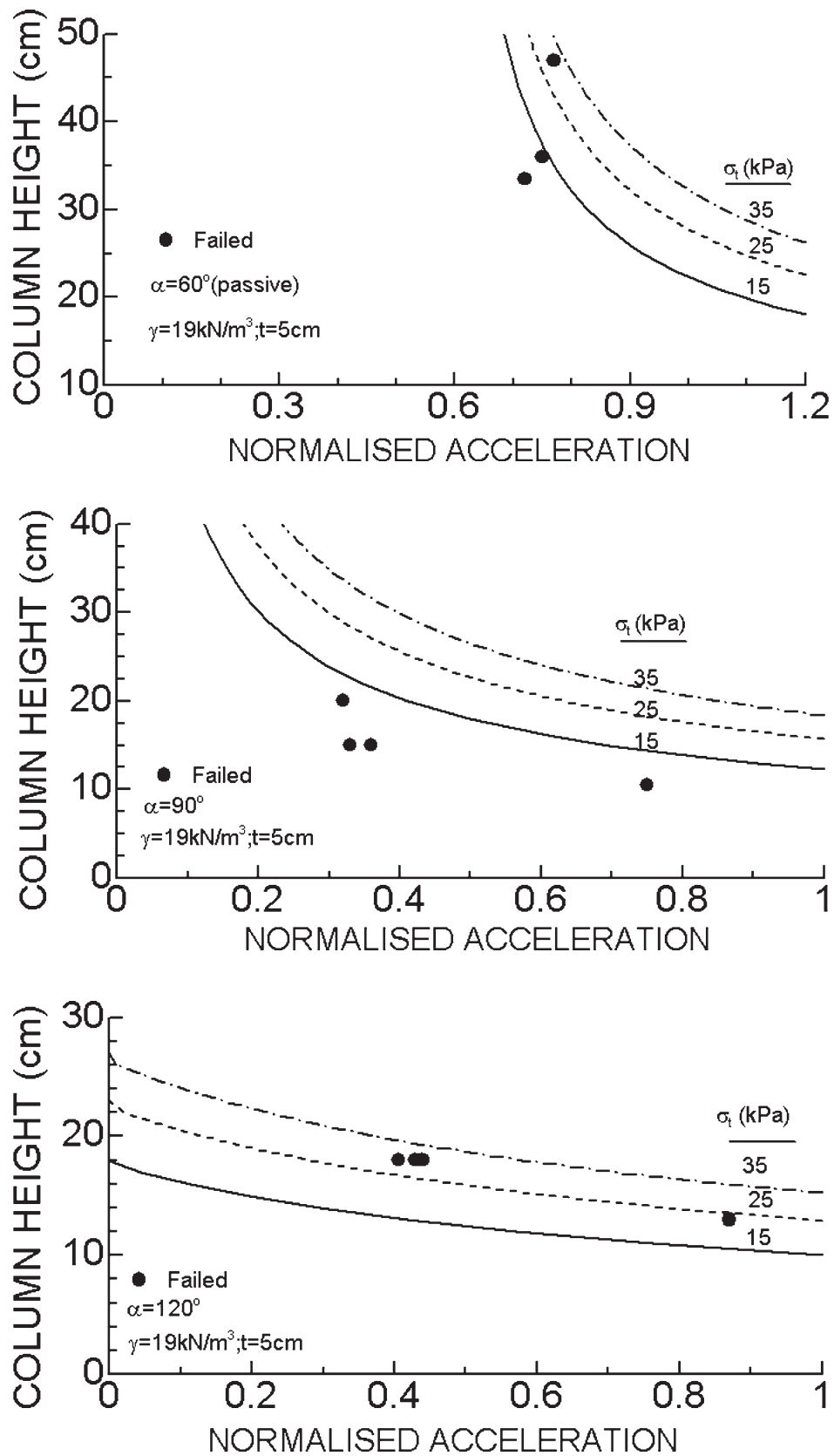


Figure 19. Comparison of experimental results with theoretical estimations

Table 4: Experimental results for rock slope models

Experiment No	Layer inclination degree	Slope Angle (degree)	Slope Height (cm)	Seismic coefficient (a/g)	Condition
1	135	90	22	1.2	Stable
2	120	90	25	1.3	Unstable
3	90	90	20	0.25	Unstable
4	90	90	20	0.29	Unstable
5	90	60	20	0.88	Unstable
6	90	45	20	0.94	Unstable
7	90	90	25	0.25	Unstable
8	90	45	25	0.7	Unstable
9	90	90	15	1.15	Unstable
10	90	45	15	1.64	Stable
11	60	60	26	0.7	Unstable
12	45	45	22.6	1.26	Stable

1. If dip direction of layers is approximately the same as dip direction of the slope, the rock mass will not have a potential of flexural toppling failure under static condition.
2. Flexural toppling failure under dynamic condition can be classified into two modes: active and passive.
3. Experiments indicated that the required seismic coefficient is much higher for single column subjected to passive toppling as compared with that of columns subjected to active toppling.
4. The experiments on model rock slopes also indicated that the required seismic coefficient is much higher for slopes subjected to passive toppling as compared with that of slopes subjected to active toppling.
5. Predictions of seismic coefficient for the initiation of flexural toppling failure of a single column under active or passive modes by analytical method are close to experimental results.

ACKNOWLEDGEMENTS

The second author would sincerely acknowledge the scholarship provided by the Ministry of Science, Research and Technology of Iran for his studies in The Marine Civil Engineering Department of Tokai University and Tokai University for the kind permission for this study in its education and experimental premises. The experiments reported in this article have been

carried out at the Aydan's laboratory and structural laboratory of Marine Civil Engineering of Tokai University and it is gratefully acknowledged.

References

- Adhikary, D. P., Dyskin, A. V., and Jewell, R. J. (1996): Numerical modelling of the flexural deformation of foliated rock slopes. *Int. J. Rock Mech. Min. Sci. Geomech. Abstr.* **33**(6), 595-600.
- Adhikary, D. P., Dyskin, A. V., Jewell, R. J. and Stewart, D. P., (1997): A Study of the Mechanism of Flexural Toppling Failure of Rock Slopes, *Rock Mech. Rock Engng.* (1997) **30**(2), 75-93.
- Adhikary, D. P., Dyskin and A. V., (2007): Modeling of progressive and instantaneous failure of foliated rock slopes, *Rock Mech. Rock Engng.* **40**(4), 349-362.
- Amini M., Majdi A., Aydan Ö, (2008): Stability Analysis and Stabilization of Flexural Toppling Failure, *Rock Mech. Rock Engng.*, DOI 10.1007/s00603-008-0020-2.
- Aydan, Ö. and T. Kawamoto (1987). Toppling failure of discontinuous rock slopes and their stabilization (in Japanese). *Journal of Japan Mining Society*, **103**(1197), 763-770.
- Aydan, Ö. (1987): A note on the seismic stability of discontinuous rock slopes. Interim report, December 7, Geotechnical Engineering Department, Nagoya University, 16 pages.
- Aydan, Ö., T. Kyoya, Y. Ichikawa, and T. Kawamoto (1988). Flexural toppling failures in slopes and underground openings. *The 9th Western Japan Rock Engineering Symposium*, 72-80.

- Aydan, Ö. (1989): The stabilisation of rock engineering structures by rockbolts. Doctorate Thesis, Nagoya University, 204pages.
- Aydan, Ö. and Kawamoto, T., (1992): Stability of slopes and underground openings against flexural toppling and their stabilization, *Rock Mech. Rock Engng.* **25**(3), 143-165.
- Bukovansky, M., Rodriguez, M., Cedrun, G. (1974): Three rock slides in stratified and jointed rocks. In: *Proc., 3rd Int. Congr. on Rock Mechanics*, ISRM Denver 2B, 854-858.
- De Freitas, M. H. and Watters, R. J. (1973): Some field examples of toppling failure. *Geotechnique* **23**(4), 495-514.
- Erguvanli, K., Goodman, R. E. (1970): Applications of models to engineering geology for rock excavations. *Bull. Assoc. Eng. Geologists*, 9.
- Egger, P. (1983): A new development in the base friction technique. *Coll. Phys. Geomech. Models*, ISMES, 67-87, Bergamo.
- Goodman, R. E., and Bray, J. W. (1976): Toppling of rock slopes, *ASCE Specialty Conference on Rock Engineering for Foundations and Slopes*, Boulder Colorado, Vol. 2, pp. 201-234.
- Hoffmann, H. (1974): Zum Verformungs- und Bruchverhalten regelmäßig geklufteter Felsboschungen. *Rock Mech. Suppl.* **3**, 31-34.
- Kawamoto, T., Obara, Y., Ichikawa, Y. (1983): A base friction apparatus and mechanical properties of a model material (in Japanese). *J. Min. Metall. Inst. of Japan* **99**, 1-6.
- Muller, L. (1968): New considerations on the Vajont slide. *Rock Mech. Eng. Geol.* **6**, 1-91.
- Pritchard, M. A., (1989): Numerical modeling of large scale toppling, M.A.Sc. Thesis, the University of British Columbia, Vancouver, B.C.
- Shimizu, Y. Ö. Aydan, Y. Ichikawa, T. Kawamoto, (1988): The stability and failure of discontinuous rock slopes (in Japanese), *Journal of Civil Engineering, Geotechnical Division*, 400-III-10, 189-198.

要 旨

振動下における岩盤斜面のたわみ性トップリング破壊に関する
実験的研究およびその安定性評価に対する理論的考察

アイダン・オメル

東海大学海洋学部建設工学科, 静岡

メディ・アミニ

Tehran University, School of Mining Engineering, Tehran, Iran

たわみ性トップリング破壊は岩盤斜面の破壊様式の中で重要な破壊様式の一つである。この破壊様式は曲げによって層状岩盤の場合に発生するものである。一般的に主動型たわみ性トップリング破壊がよく知られており、たわみ性トップリング破壊は一般的に重力、水力および地震力によって生じる。主動型たわみ性トップリング破壊と別に地震力や氷河あるいは表層部の風化岩盤による上載荷重によって受動型のたわみ性トップリング破壊をすることがある。この現象は地質および地形学の研究者の間で斜面のクリープと呼ばれていたが、Aydan ら (1988) はその解釈が間違っていることを指摘し、受動型たわみ性トップリング破壊として解釈すべきことを表明した。受動型たわみ性トップリング破壊は氷河あるいは風化層の与える影響よりも地震力によって発生することが多いと思われる。本研究で振動模型実験機を用いて地震力を受ける岩盤斜面の受動型たわみ性トップリング破壊のメカニズムを実験的に検討した。最初に単独のブロックについて実験、その後斜面について模型実験を行った。また、既存の解析法とその適用性について考察した。