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Effects of progressive strength degradation on seismic-induced displacements of rock slopes

F.M. Soccodato & G. Tropeano
Department of Civil, Environmental Engineering and Architecture, University of Cagliari, Cagliari, Italy

ABSTRACT: Seismic performance of slopes is typically assessed by evaluating the permanent displacements as a function of some earthquake response representative parameters. This paper presents the main results of a numerical study devoted to the evaluation of the influence of the progressive material strength reduction (from peak to ultimate values) induced by shear displacements on the seismic performance of jointed rock slopes. The parametric study refers to the simple one–dimensional problem of a rock slope characterized by the presence, at some fixed depth, of one planar joint. A simple modification of the Newmark rigid block model has been adopted in order to simulate the reduction of shear strength; different problem geometries (joint angle and depth) were considered in evaluating the distribution of permanent displacements induced by a selection of accelerometric records. Results seems to indicate that semiempirical relationships developed with reference to the original Newmark model (with constant shear strength) may be considered valid also in the case of gradually reducing shear strength, if the final value of yield acceleration is properly estimated. With increasing initial joint roughness only minor differences in the distribution of permanent displacements as obtained from constant or reducing shear strength rigid block models were found.

1 INTRODUCTION

Landslides induced by earthquakes are major sources of vulnerability of the natural and built environment. For natural slopes, the deformation phenomena are linked substantially to the landslide susceptibility of the territory, and landslide re-activations can occur for relatively low energy events. Conditions that bring to a possible collapse of a slope under seismic actions are mainly related to the interaction between the seismic waves and the crossed medium (rock or soil), increasing the destabilizing actions and reducing the resistant ones. In fact, the main effects are: a) the increase of the destabilizing inertial forces and their amplification (stratigraphic and topographic effects); b) the reduction of resistant forces due to the degradation of material strength parameters, or due to the increasing of pore water pressure in saturated soils.

The slope performance is typically assessed by evaluating the permanent displacements as a function of some earthquake response representative parameters. For this purpose, methods based on rigid Newmark block model and following modifications can be easily adopted to estimate the statistical distribution of displacements for a given intensity seismic motion parameter. According to a common representation, the displacements computed through the Newmark method for a database of acceleration records properly selected are expressed as function of the ratio of the yield acceleration of the slope, a_y, and a measure of the earthquake intensity, such as the maximum acceleration, $a_{\max}$ (typically, the peak of the recorded acceleration time histories in the direction of the slope) and/or other ground motion parameters.

The aim of this study is to include the effects of a variable shear strength of the rock joint in the semi-empirical approach for the probabilistic estimation of permanent, seismic induced, displacements of rock slopes characterized by the presence of planar joints.

In the following, the modification of the rigid-perfectly plastic block model in order to take account for degradation of the rock joint asperities will be briefly discussed; the simple rock block (SRB) scheme will be then introduced and the adopted solution technique described. Successively, it will be shown the dataset of the recording time histories considered in the parametric study of the SRB scheme. Finally, the results of the analyses will be illustrated and discussed, highlighting the effects of decreasing shear strength on permanent displacement distributions as compared to those predicted by the original Newmark method.

2 METHOD

2.1 *Newmark rigid block method with strength reduction*

The rigid block method, in its original form (Newmark, 1965), assumes that the material strength parameters affecting the available shear strength on the slip surface are constant; in other words, for a given normal stress, no changes of shear strength due to plastic shear.

In this study, a modified rigid block method has been considered, following, with some minor changes, the procedure proposed by Qi & Liu (2015).

For rock joints, with reference to a rigid-plastic constitutive model, the following strength rule (Patton, 1966) is widely used:

$$\tau = \sigma_n \cdot \tan(\varphi + \alpha) \tag{1}$$

where the asperity angle, α, which is dependent on the joint surface roughness, on the material strength and on the normal stress, σ_n, increases the (residual) friction angle of the material, φ (Barton, 2013).

The asperity angle can be evaluated by the semi-empirical relationship proposed by Barton & Choubey (1977):

$$\alpha = JRC \cdot \log(JCS/\sigma_n) \tag{2}$$

where *JRC* is the *Joint Roughness Coefficient* (Barton, 1973) and *JCS* is *the Joint wall Compressive Strength*. The parameters *α, φ, JRC* and *JCS* may be evaluated in different ways, for example by carrying out laboratory or in situ tests and/or by using semi-empirical relationships with the rebound number obtained with the Schmidt hammer. Joint filling material, weathering of the joint wall material and scale effects should be also considered in the selection of appropriate values of the joint parameters just above introduced (Barton, 2013).

During sliding, rock joint asperities are progressively damaged.

To evaluate the progressive reduction of strength along the surface of a rock joint, Crawford & Curran (1981, 1982), basing on a limited set of experimental observations, proposed the following relationships, in which the frictional strength of discontinuities is function of displacements, u, and rate of displacements, v. For a given instant of time, the mobilized dynamic friction angle, φ_m, may be obtained by evaluating the mobilized yield acceleration, $(a_y)_m$:

$$(a_y)_m = \begin{cases} (a_y)_{m,0}[1-(1-p)u/u_0] & (u \le u_0 \wedge v < v_0) \\ (a_y)_{m,0}[1-(1-p)u/u_0]\cdot[1+R\cdot 0.20\log(v/v_0)] & (u \le u_0 \wedge v \ge v_0) \\ p\cdot(a_y)_{m,0} & (u > u_0 \wedge v < v_0) \\ p\cdot(a_y)_{m,0}[1+R\cdot 0.20\log(v/v_0)] & (u > u_0 \wedge v \ge v_0) \end{cases} \tag{3}$$

where $(a_y)_{m,0}$ is the initial value of the yield acceleration (with $\varphi_m = \varphi + \alpha$), p the amount of strength reduction ($0.0 < p \le 1.0$), u_0 and v_0 threshold values of displacements and rate of displacements, and R a material parameter. Crawford & Curran (1982) suggested $u_0 = 5.0$ cm, $v_0 = 1$ cm/s and $R = 1$ or $R = -1$ for weak and hard rocks, respectively. The parameter p can be simply defined as the ratio between the final (computed assuming $\alpha = 0$) and the initial values of yield acceleration.

Plesha (1987) proposed a constitutive law for the behaviour of rock joints that includes dilatancy and contact surface degradation based on a physical approach. The formulation considers macroscopic and microscopic effects on the contact surface. One of the major features of this constitutive law is the development of an explicit relationship which describes the evolution (degradation) of the asperity angle:

$$\alpha_t = \alpha_0 \cdot \exp\left(-c\, W_t^P\right) \tag{4}$$

where α_0 is the initial value of the asperity angle, W_t^P is the (plastic) work done by the shear stress acting along the slip surface in the relative displacement u ($dW_t^P = \tau\, du$). For the constant c that rules the rate on which asperities are damaged Hutson (1987) and Hutson & Dowding (1990) proposed the following relationship:

$$c = 0.114 \cdot JRC(\sigma_n/JCS) \tag{5}$$

In this study, the relationship proposed by Plesha (1987) is used.

2.2 *Simple rock block scheme*

Figure 1 shows the geometry of the SRB model, which broadly corresponds to a rock slope characterized by the presence of a single and planar joint. This scheme can refer to the case of a sliding block on an inclined plane or to that of an indefinite slope

The rigid block (mass m and height h) may slides along a planar surface with slope β; the basement is subjected to an acceleration time history, $\ddot{u}_g$, eventually inclined by an angle θ to the horizontal direction.

Considering the equilibrium condition of forces in the direction perpendicular to the sliding plane, the normal force, N, is given by:

$$N = m\, g \cdot \cos\beta + m\, \ddot{u}_g\, g \cdot \sin(\theta - \beta) \tag{6}$$

where g is the gravity acceleration. In the direction of the sliding plane (AB), the equilibrium condition is:

$$m\, \ddot{u}_g \cdot \cos(\theta - \beta) + m\, g \cdot \sin\beta - N \cdot \tan(\varphi + \alpha) = m\, \ddot{u} \tag{7}$$

The equation of motion of the block (displacement u) is obtained from the dynamic equilibrium of the forces acting on the block introducing eqn. (6) in eqn. (7), and it can be written in the following compact form:

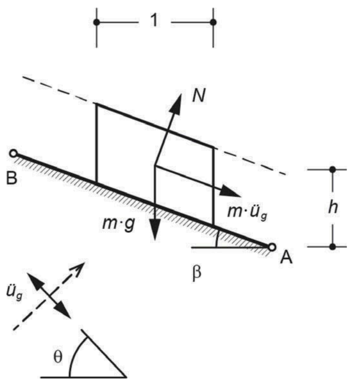

Figure 1. Simple rock block scheme.

$$\ddot{u} = F_1 \cdot \ddot{u}_g - F_2 \tag{8}$$

where:

$$F_1 = \cos(\theta - \beta) - K \cdot \sin(\theta - \beta); F_2 = g\,(K \cdot \cos\beta - \sin\beta) \tag{9}$$

and K:

$$K = \tan(\varphi + \alpha) \tag{10}$$

By imposing $\ddot{u} = 0$ in eqn. (8), the yield acceleration is equal to:

$$a_y = F_2/F_1 \tag{11}$$

At each time instant, eqn. (4) can be used to evaluate the current asperity angle, α_t, while eqn. (11) furnishes the current yield acceleration $a_{y,t}$. The mobilized friction angle, φ_m, can be thus obtained by using eqn. (10).

The solution of the motion needs, for each time interval, the updating of strength parameters: an iterative solution procedure is thus necessary because α_t depends on the displacement at the generic time instant t.

2.3 *Seismic database*

The set of accelerometric records used in this study was extracted from the databases PEER NGA WEST 2 (Ancheta et al, 2013) and ESM (Luzi et al, 2016), by selecting, for the time period 1954 to 2016, 52 recordings of 36 earthquakes with moment magnitude, M_w, greater than 5.0, and Joyner and Boore distance less than 50 km, as shown in Figure 2a. More in

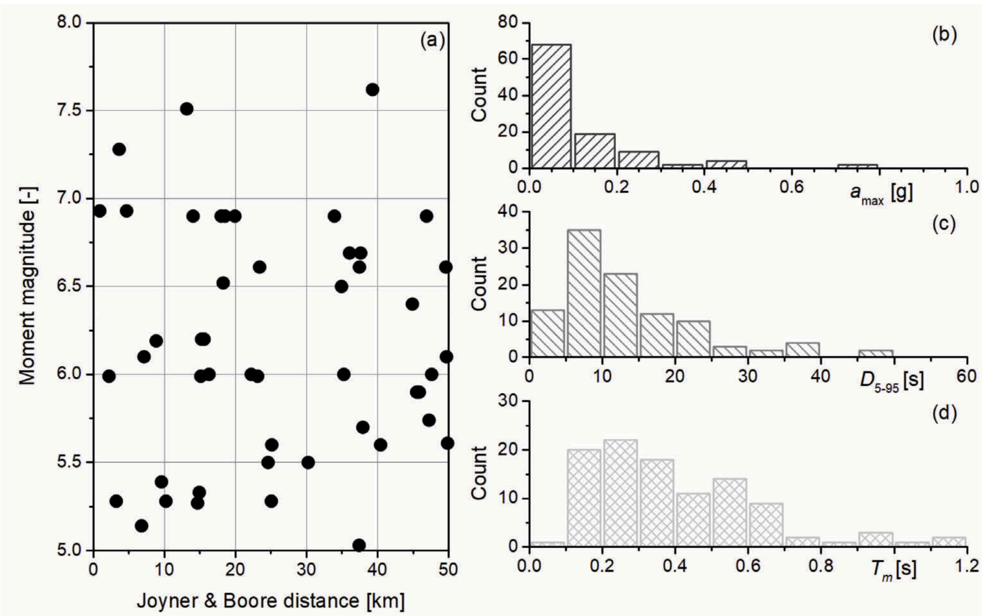

Figure 2. Seismic dataset used in this study (a) and sampling frequency distribution of main ground motion parameters: peak ground acceleration (b), significant duration (c) and median period (d).

detail, the database includes 104 horizontal components of records from accelerometric stations located on rock-soil outcrops with $V_{S,30} > 800$ m/s, $V_{S,30}$ being the weighted average shear wave velocity measured through reliable seismic site tests in the first 30 m of depth.

The horizontal components of the selected records have been pre-processed in order to obtain some of the most significant ground motion parameters (maximum acceleration, $a_{\max}$; significant duration, $D_{5\text{-}95}$; mean period, T_m) and integral parameters (Arias intensity, I_a; Housner intensity, I_H).

The graphs in Figure 2 shows the frequency distribution of $a_{\max}$, $D_{5\text{-}95}$ and T_m. The modal value of $a_{\max}$ falls between 0.05 and 0.1 g (Figure 2b). The significant duration has its most frequent value less than 10 s and only few accelerograms have duration higher than 30 s (Figure 2c). The mean period shows an almost uniform distribution, with values between 0.1 and 0.7 s (Figure 2d). A few accelerograms are characterized by T_m greater than 0.8 s: they roughly correspond to records affected by near-source effects.

In a first set of analyses, the horizontal components of the selected records have been used to evaluate the Newmark rigid block (with constant shear resistance) permanent displacements for $a_y/a_{\max}$ varying from 0.1 to 0.9.

The sample distribution of the normalized permanent displacements versus the yield acceleration ratios are shown in Figure 3. The sampling distribution of all data series are shown with box-plots that permit to summarize the main confidence values (median, lower and upper quartiles, 1% and 99% percentiles). Box-plots are also compared with the ranges of the permanent normalized displacements (median prediction, black line, 1% - 99% non-exceeding percentiles, grey-filled areas) evaluated with the semi-empirical relationship proposed by Tropeano et al. (2017) for Italian seismicity, showing a reasonable agreement.

2.4 *Parametric study*

The values of the relevant parameters adopted in this study are listed in Table 1. In order to compare the results with those related to the original Newmark method, the inclination of the seismic action, θ, was set equal to the inclination of the sliding surface, β; in such a way, the normal stress is not time dependent.

The physical and strength parameters adopted are typical of a limestone rock joint with a slightly rough surface ($JRC = 2.5 \div 7.5$); different slope angles have been considered. The study was carried out by varying the normal stress acting on the sliding surface, i.e. the height of the sliding block, h. The initial values of the asperity angle, α_0, and, consequently, the initial yield acceleration, $(\alpha_y)_0$, ranges between 0.287 g ($JRC = 2.5$, $\beta = 30°$ and $h = 5$ m) and 1.573 g ($JRC = 7.5$, $\beta = 12°$ and $h = 1$ m).

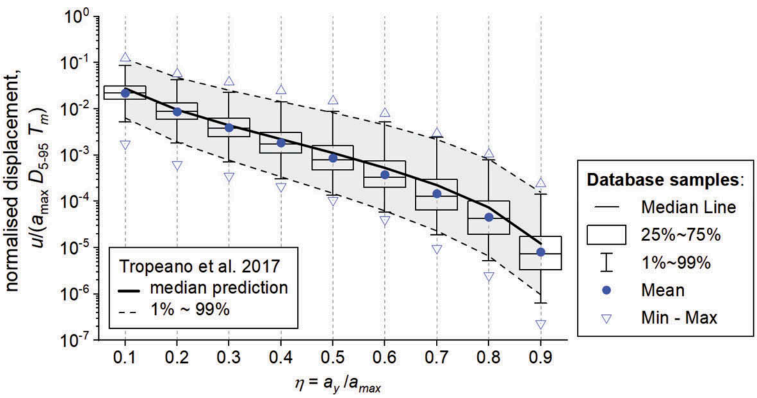

Figure 3. Comparison between the sample distributions of the normalised displacements evaluated with the Newmark method, with constant strength, and the semiempirical relationship proposed by Tropeano et al, (2017).

Table 1. Main parameters used in this study

Parameter		Value
unit weight	γ	25 [kN/m^3]
inclination of sliding surface	β	12, 15, 18, 21, 27, 30 [°]
block height	h	1.0, 1.5, 2.0, 2.5, 3.0, 4.0, 5.0 [m]
joint roughness coefficient	JRC	2.5, 5.0, 7.5
joint wall compressive strength	JCS	75 [MPa]
base friction angle	φ	35[°]

In order to guarantee the activation of sliding for a given initial yield acceleration $a_{y,0}$, it was decided to scale each acceleration time history to peak accelerations able to trigger the sliding with a specific not-exceeding probability. More in detail, the values of $a_{\max}$ were calculated using the relationship by Tropeano et al. (2017) already shown in Figure 3, setting the threshold displacement equal to 1.0 cm and the not-exceeding probability equal to 99%: in this way, values of the initial yield acceleration ratio between 0.6 and 0.9 have been obtained, minimizing scaling effects.

3 RESULTS

Normalised displacements obtained from the analyses are shown in Figure 4 as a function of the final yield acceleration ratio for the three JRC values considered in this study. For the displacements computed with the model characterized by constant shear strength, u_b, (Figure 4, blue symbols), the yield acceleration $a_{y,b}$, evaluated with reference to the friction

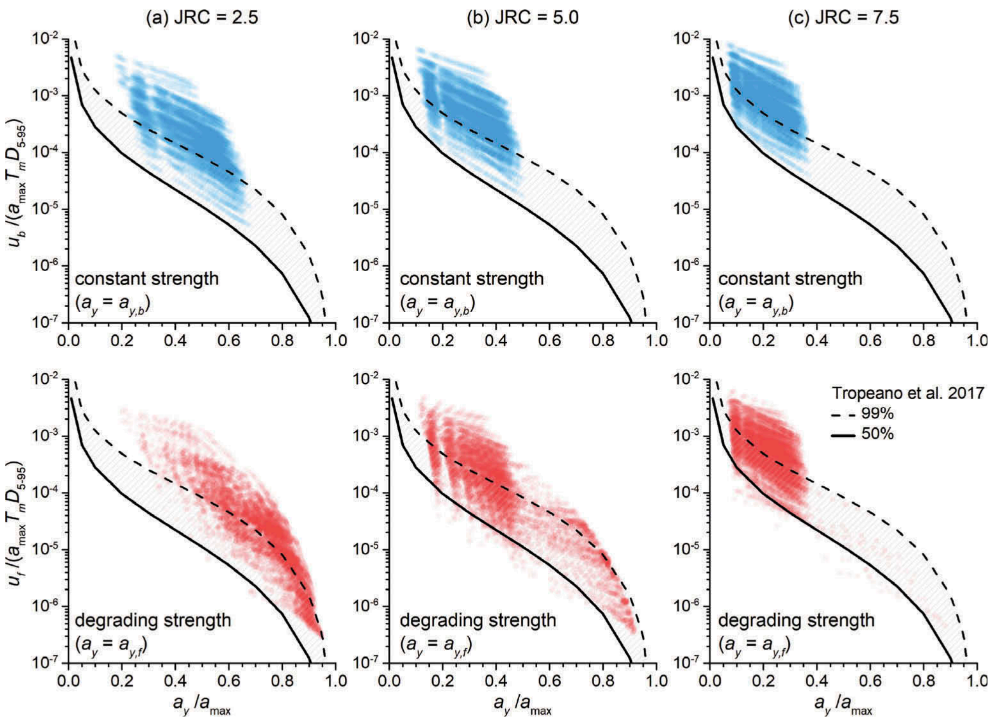

Figure 4. Sample distributions of the normalised displacements evaluated with the Newmark method, with constant (residual) strength (blue symbols), and with degrading strength (red symbols).

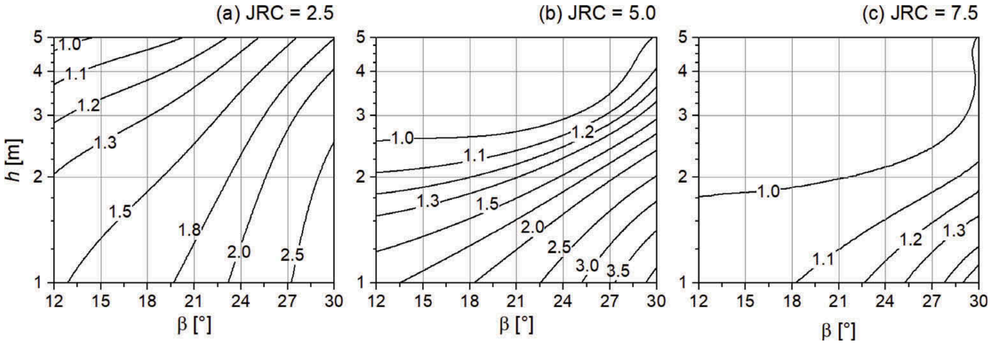

Figure 5. Ratio of the final yield acceleration achieved in the analysis, $a_{y,f}$, to final available (ultimate) yield acceleration, $a_{y,b}$, as a function of block height, h, and inclination of the sliding surface, β, for JRC equal to: 2.5 (a), 5 (b) and 7.5 (c).

angle φ (being, in this case, $\alpha = 0$), is a (different) constant in each single analysis. For the displacements computed with the model characterized by degrading shear strength (Figure 4, red symbols), u_f, the yield acceleration a_y decreases from the initial value $a_{y,0}$ to a final, achieved value $a_{y,f}$, which may be greater than $a_{y,b}$.

It can be noted that, for all the analyses, normalized displacement distributions correspond, on the average, to the same percentile of the semiempirical relationship (Figure 3) which was used to scale the accelerograms. Normalized displacements obtained from the analyses carried out with variable a_y show, with increasing JRC, a larger $a_{y,f}/a_{\max}$ range than that obtained from the analyses carried out with constant shear strength.

Data scattering, at the same JRC value, appears to be quite similar for constant and degrading shear strength analyses; furthermore, no significant correlation with other integral ground motion parameters (I_a, I_H) was found by evaluating residual values. Differences in displacements should be thus only linked to differences between the final yield acceleration achieved, $a_{y,f}$, and the final available (ultimate) yield acceleration, $a_{y,b}$. In other terms, differences are related to the amount of damage of the initial roughness of the joint induced by sliding.

The ratio of the final yield acceleration achieved, $a_{y,f}$, to the final available (ultimate) yield acceleration, $a_{y,b}$, is showed as function of the block height, h, and of the slope angle, β, in Figure 5. These plots may be used in order to evaluate $a_{y,f}$ and, successively, to predict expected normalized permanent displacements by using Figure 4 (or Figure 3).

It may be noted that, with increasing JRC, roughness of the sliding surface is more and more completely destroyed ($a_{y,f}/a_{y,b} = 1$). For a given JRC value, this effect increases with increasing thickness of the block and/or with decreasing slope angle of the sliding surface.

4 CONCLUSIONS

The results of this study indicate that the effects of the degradation of rock joint strength on permanent seismic induced displacements are significantly affected by the final yield acceleration values reached, which, in the model adopted, depend on the amount of damage of the asperities.

Ground motion parameters appear to be of minor relevance in describing differences between displacements evaluated considering constant or decreasing shear strength models.

Semiempirical relationships developed with reference to the original Newmark model (with constant shear strength) may be considered valid also in the case of gradually reducing shear strength only if the final value of yield acceleration is properly estimated.

Results appear to point out that with increasing initial joint roughness only minor differences in the distribution of permanent displacements as obtained from constant or reducing shear strength rigid block models were found.

Further analyses are needed in order to validate the correlations obtained in this study, by expanding the parametric studies.

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REFERENCES

Ancheta, T.D. & Darragh, R.B. & Stewart, J.P. & Seyhan, E. & Silva, W.J. & Chiou, B.S.J. & Wooddell, K.E. & Graves, R.W. & Kottke, A.R. & Boore, D.M. & Kishida, T. & Donahue, J.L. 2013. PEER NGA-West2 Database. *PEER Report 2013/03*. Pacific Earthquake Engineering Research Center Headquarters at the University of California, Berkeley

Barton, N. 1973. Review of a new shear-strength criterion for rock joints. *Engineering Geology*. 7:287–332. doi: 10.1016/0013-7952(73)90013-6

Barton, N. 2013. Shear strength criteria for rock, rock joints, rockfill and rock-masses: Problems and some solutions. *Journal of Rock Mechanics and Geotechnical Engineering* 5: 249-261.

Barton, N.& Choubey, V. 1977. The shear strength of rock joints in theory and practice. *Rock Mechanics Felsmechanik Mécanique des Roches* 10:1–54. doi: 10.1007/bf01261801

Crawford, A.M. & Curran, J.H. 1981. The influence of shear velocity on the frictional resistance of rock discontinuities. *International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts* 18:505–515. doi: 10.1016/0148-9062(81)90514-3

Crawford, A.M. & Curran, J.H. 1982. The influence of rate- and displacement-dependent shear resistance on the response of rock slopes to seismic loads. *International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts*. 19:1–8. doi: 10.1016/0148-9062(82)90704-5

Hutson, R.W. 1987. Preparation of duplicate rock joints and their changing dilatancy under cyclic shear. *Ph. D. Thesis, Northwestern University*, Evanston, Illinois.

Hutson, R.W. & Dowding, C.H. 1990. Joint asperity degradation during cyclic shear. *International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts*. 27:109–119. doi: 10.1016/0148-9062(90)94859-r

Luzi, L., & Puglia, R. & Russo, E. & ORFEUS WG5. 2016. Engineering Strong Motion Database, version 1.0. *Istituto Nazionale di Geofisica e Vulcanologia, Observatories & Research Facilities for European Seismology*. doi: 10.13127/ESM

Newmark, N.M. 1965. Effects of Earthquakes on Dams and Embankments. *Géotechnique* 15:139–160. doi: 10.1680/geot.1965.15.2.139

Patton, F.D. 1966. Multiple modes of shear failure in rock. *Proc. Ist Cong. ISRM*. Vol. 1, Lisbon.

Plesha, M.E. 1987. Constitutive models for rock discontinuities with dilatancy and surface degradation. *International Journal for Numerical and Analytical methods in Geomechanics*. 11:345–362. doi: 10.1002/nag.1610110404

Qi, S. & Liu, C. 2015. Permanent displacement of rock slope considering degradation of slide surface during Earthquake, 10th Asian Regional Conference of IAEG (2015), Kyoto, Japan.

Tropeano, G. & Silvestri, F. & Ausilio, E. 2017. An uncoupled procedure for performance assessment of slopes in seismic conditions. *Bulletin of Earthquake Engineering*. 15(9): 3611–3637. doi: 10.1007/s10518-017-0113-y