Hazard assessment in dynamic slope stability analysis

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Abstract

The estimate of risk in urban planning activities should cover a primary role in order to avoid natural disasters. The risk assessment results from the product between the vulnerability of the value at risk and the hazard. The hazard measures the probability of the occurrence of the adverse natural event for human lives and activities, such as flood, slides, earthquakes, etc. Therefore the most important point in risk assessment is the hazard evaluation. Here the hazard of landslides generated by seismic events are investigated. The stability of a slope during or after a seismic event can be studied by means of different methods according to the approximation accepted. In fact the pseudo-static approach estimates the stability of a slope under dynamic loads by the dynamic safety factor. It results from the pseudo-static approach, by means of the ratio between the shear strength of the soil and the stress condition induced by seismic loads. This approach is not able to account for either the seismic displacements of the slope or the influence of the duration and the time variation of the seismic struck acceleration over the slope displacements. These latter two aspects of the problem are the most critical points to deal with in seismic slope stability. In this paper Newmark's method has been employed for studying the stability under seismic conditions of a slope in Pomarico's village. The physical and mechanical soil properties are accounted for as random variables in order to estimate the failure probability and the reliability index of the permanent displacements estimated.
Keywords: hazard assessment, dynamic slope stability, pseudo-static method, sliding block method, inherent variability, random field, Montecarlo method.

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1 Introduction

The estimation of landslide risk in areas at medium-high seismic level is a complex activity. The risk is affected by the probability of occurrence of numerous events whose spatial and time variability is difficult to evaluate as:

- The occurrence of a seismic event whose intensity is able to induce the slope collapse;
- The decrease of the shearing resistance of soils within the slope due to distinct reasons as: significant rainfalls happened before the earthquake and seismic actions which can increase the pore pressures and/or can induce the cyclic degradation phenomenon;
- The reactivation of previous instability phenomena;
- The triggering time of the landslide which can be delayed to post-seismic phase.

Such factors are difficult to forecast in studies developed in preventive phase which do not use back-calculation. These types of studies are carried out to supply the local administrations with useful tools in order to program landscape management by means of the risk assessment. Three types of methods are commonly applied in the evaluations of dynamic stability of natural and artificial slopes as (1) Pseudo-static analysis, based on the limit analysis method of comparing resistance with acting forces; (2) Kinetic method, based on the model of the sliding block proposed by Newmark; and (3) Numerical simulations (finite elements or finite differences).

In the following study the first two methods are employed joined to probabilistic methods in order to account for the uncertainty of many variables which affect the seismic hazard evaluation.

2 Deterministic methods for evaluating the seismic stability of slopes

The pseudo-static method and Newmark's displacement method have been applied to the evaluation of the slope stability in this study. These methods are based on the hypotheses of a rigid-plastic behaviour model of the materials within the slope. Seismic actions are represented by means of the horizontal inertia force which is constant in time and equal to $W \cdot a_{max}$ where a_{max} is the peak horizontal acceleration awaited to the site investigated. According to the pseudo-static method the stability is measured by means of the calculation of a Dynamic Safety Factor (DSF) conceptually similar to the Static Safety Factor. Such safety factor must turn out higher than the unity to ensure the stability of the slope. The state of collapse will be identified by a value of the SF=1.

On the other hand the method of the sliding block, Newmark [4], is based on the hypothesis that the permanent displacements are induced by the earthquake on the unstable mass, modelled as a rigid block, only whether the seismic

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acceleration exceeds the slope critical acceleration which corresponds to the incipient collapse condition. The displacement method evaluates the sliding block permanent displacements under variable actions according to the accelerograms of earthquakes considered. For such reason the slope can register limited displacements instead of a destructive collapse even if the Dynamic Safety Factor gets down beneath the unity. Besides D'Elia [9] highlighted that the stability of slopes, is strongly affected by the amplitude of displacements. For this reason in the last 30 years a lot of relations have been formulated to relate permanent displacements, the critical acceleration and the representative parameters of the regional seismic activity.

Ambraseys and Menu [3] took under consideration a small number of earthquakes with magnitude 6.9±0.3 and determined that the predominant period doesn't affect the induced permanent displacements. The epicentral distance, the magnitude (Ms) of the earthquake and the duration of the shaking (D) don't affect the displacements as well. For this reason they suggest the following expression, for the calculation of the permanent displacements:

$$\log(d) = 0.9 + \log\left[\left(1 - \frac{k_c}{k_m}\right)^{2.53}\left(\frac{k_c}{k_m}\right)^{-1.09}\right] + 0.3t \tag{1}$$

which is valid for $k_c/k_m \in]0.1,0.9[$ and under the hypothesis of normal distribution probability function of displacement. In eqn. (1) t is the standardized normal variable. When t=0 the mean value of displacements is calculated. Therefore Ambraseys and Menu state that the attention must be focused on the ratio k_c/k_m since the characteristic critical acceleration (k_c) of the slope is representative of the slope resistance even though it is assumed constant in this simplified Newmark's model. The maximum acceleration (k_m) is representative of the earthquake shaking action.

3 Variability in accessing the dynamic stability of a slope

Uncertainties and variabilities, which differences were stressed by Cherubini and Orr [6], relate to the description of the resistance parameters of and can be drawn from different sources as: (1) the epistemic variability concerning the presence of weakness surfaces (interfaces between layers or previous sliding surfaces); (2) systematic errors in parameter measuring and modelling simplification concerning the instability phenomenon and the behaviour of the materials which made up of the slope; (3) the inherent variability and the heterogeneity of natural materials due to their formation history. The epistemic variability as well as the systematic errors can be investigated by increasing and extending field measurements. In this paper, the attention is focused on the inherent variability of soils. For this reason the stochastic field theory reported by Vanmarcke [5] has been employed and applied to the resistance parameters of soils. This variability model describes the parameters as spatial random variables characterized by mean value, "local" variance and spatial variability. This latter can be described by means of the scale of fluctuation and the variance reduction

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function. According to a rigid-plastic behaviour model of soils, Mohr-Coulomb criterion is used. Soil resistance is described through undrained cohesion, effective cohesion and friction angle. As long as the dynamic stability of a slope is a two-dimensional problem then it involves the horizontal and vertical spatial variability of the resistance parameters. Accordingly Babu and Mukesh [8] state the horizontal and vertical scale of fluctuation values for c, c_u e ϕ are far each other up to two orders of magnitude (δ_h=10m÷200m; δ_v=0.1m÷10m). In this study the following scales of fluctuation will be taken into account for c,ϕ e c_u:

$$\delta_h = 50\text{m} \quad \delta_v = 1\text{m} \tag{2}$$

Therefore, the variance reduction function assumes the following expression Vanmarcke [5]:

$$\gamma(T) = \begin{cases} 1 - \dfrac{T}{3\theta} & T \le \theta \\ \left(\dfrac{\theta}{T}\right)\left[1 - \dfrac{\theta}{3T}\right] & T \ge \theta \end{cases} \tag{3}$$

relying on the hypothesis of triangular correlation function:

$$\rho(\tau) = \begin{cases} 1 - \dfrac{|\tau|}{\theta} & |\tau| \le \theta \\ 0 & |\tau| \ge \theta \end{cases} \tag{4}$$

where θ is the scale of fluctuation of each parameter and τ is the distance at which the correlation is calculated. Such expression is used for stochastic broadband models which are characterized only by the scale of fluctuation, as in this case. For the slope studied below, the scale of fluctuation values used are those shown in the eqn. (2) and accounted for distances involved into the sliding mechanism, the variance reduction function results equal to $\gamma(T\approx250\text{m})=0.19$.

4 A case study in Pomarico town

The slope under study is part of the eastern side of the hilly relief on which is built Pomarico town, in Matera district. The relief is made up of the typical units belonging to the characteristic succession of plio-pleistocenic formation which fills the Bradanic Forethrough. From the bottom to the top it can be distinguished the following units: Blue Clays and Monte Marano subappennine sands. The first unit is made up of clayey silt and silty clays. The second unit is constituted by yellowish sands, often laminated, with soft rock inclusion. The hilly relief is within a system of slopes sometimes steep and sometimes smoothed by the erosion; the most of them corresponds to edges of landslides. The stability of the zone was accurately studied by a deterministic point of view Cherubini [10] because of its estimated importance in the unstable side evolution of the town. In such geomorphologic environment, the steepest section has been selected in order to study its dynamic stability. Such section (fig.1) is characterized, in the highest part of the slope by Monte Marano sands from 428 m to 400 m while the underlying stratum is made up of blue clays. The results of physical-mechanical characterization tests on the materials and the statistical calculations are synthesized in table1. As regards the seismic activity of Pomarico town, only a parametric study was led in terms of the awaited PHA (peak horizontal ground acceleration). Italian seismic building code, that is OPCM 3274 of March 20th, 2003, classifies all of the Italian towns in terms of the awaited PHA (peak horizontal acceleration) corresponding to earthquakes with return period of 475 years. Pomarico town is inserted in the zone 3 characterized by PHA values included between 0.05 g and 0.15 g, where g represents the gravity acceleration. Italian code, for the analyses of dynamic stability of the slopes, suggests the use of pseudo-static methods without however fixing lower bound values for the Dynamic Safety Factor. Therefore the stability of Pomarico slope has been studied taking as minimum allowable dynamic safety factor value equal to 1. The intrinsic variability of soil strength parameters have been taken into account by means of the Montecarlo method in order to evaluate the probability of occurrence of the calculated dynamic safety factor. The conditions investigated for the Blue Clays within the slope (fig.1) are two: drained and undrained conditions. The drained conditions were investigated to calculate the upper bound values of dynamic safety of factor. However the undrained conditions commonly represent the most critical condition in respect of stability and also referring to the variability of the undrained resistance parameter which is the undrained cohesion.

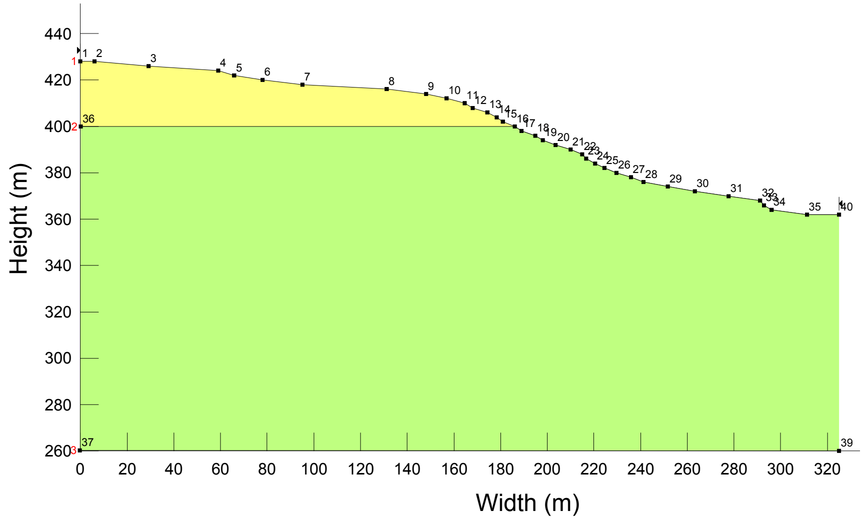

Figure 1: Pomarico's slope geometry: first layer is made up of sandy soil; second one is made up of blue clays.

Table 1: Coefficients of variation and mean values of physical-mechanical properties of the soils within Pomarico slope.

Soils	Wet unit weight γ (kN/m^3)	CV (%)	Effective Cohesion c'(kPa)	CV (%)	Undrained cohesion c_u (kPa)	CV (%)	Effective shear resistance angle ϕ' (°)	CV (%)	γ(T)
Sand	20	2					35	20	0.19
Clay	20	2	20	30	178.5	30	28	20	0.19

In fact this parameter, calculated for static conditions, turns out to have a coefficient of variation quite high, almost 30% as suggested by Phoon & Kulhawy [11]. Moreover its mean value is difficult to assess in dynamic conditions. Accordingly the Italian seismic code suggests, whether it is necessary, the resistance of the cohesive soils is to be characterized by the c_u static parameter modified in order to take into account the degradation effects due to the cyclic load application or the high velocity dynamic loading. Such studies require careful experimental laboratory investigations which are affected by uncertainties that are beyond the scope of the probabilistic study of this paper. The dynamic safety factors were calculated using Slope/W code and the results are shown in table 2-3. The limit equilibrium methods employed in calculation of dynamic safety factor are Morgenstern & Price and Bishop method.

Table 2: Dynamic Safety Factor of Pomarico slope for undrained condition.

Undrained condition			
FS		PHA	Coefficient of Variation of FS
Morgenstern & Price	Bishop		
1.018	0.923	0.003g	6.77%
0.973	0.884	0.005g	6.77%

Table 3: Dynamic Safety Factor of Pomarico slope for drained condition.

Drained condition			
FS		PHA	Coefficient of Variation of FS
Morgenstern & Price	Bishop		
1.401	1.258	0.01g	4.21%
1.243	1.112	0.02g	4.18%
1.114	1.004	0.03g	3.85%
1.008	0.908	0.04g	4.07%

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As can be seen in table 2 and 3 the dynamic safety factor mean values calculated by the two methods present a 10% difference in all of the conditions considered. The discussion on the incidence of this error, that is a systematic error, is beyond the interest of this study. However it must be considered of about 7% (in undrained conditions) and of 4% (in drained conditions) in addition to the intrinsic variability of the mechanical and physical properties of soils. As shown in tables 2–3 the critical PHA values for the dynamic stability of the slope are well beneath the lower limit of 0.05g fixed by Italian seismic code. The results of the analysis led in terms of Probability of Failure and Reliability Index are also illustrated in table 4-5. The Reliability index measures the degree of stability in terms of performance level. The calculated Reliability index values (tables 4-5) can be compared with the ones suggested in table 6 by the U.S. Army Corps of Engineers [1]. The comparison shows that the slope under undrained conditions has always a high risk level (Performance level = hazardous) confirmed by the circumstance that the interval of possible values of FS is well beneath FS=1 (table 4, in italic). According to drained conditions, the performance level reaches the hazardous value when PHA=0.04g which is still lower than the minimum PHA value suggested by seismic code for the seismic zone=3.

Table 4: Probability of failure and Reliability Index for the Dynamic Factor of Safety in wet condition.

Undrained condition				
P_f	Reliability Index β	PHA	Minimum and Maximum of Dynamic Safety Factor values corresponding to 99% of the probability of occurrence	
38.82%	0.283	0.003g	*0.812*	1.226
64.70%	0	0.005g	*0.777*	1.173

Table 5: Probability of failure and Reliability Index for the Dynamic Factor of Safety in dry condition.

Drained condition				
P_f	Reliability Index β	PHA	Minimum and Maximum of Dynamic Safety Factor values corresponding to 1% of probability of exceedance	
0.00%	6.824	0.01g	1.224	1.578
0.0020%	4.886	0.02g	1.087	1.399
0.6477%	2.483	0.03g	0.989	1.247
43.43%	0.165	0.04g	*0.885*	1.131

Finally considering the results of the probabilistic analysis led on simple indexes of dynamic stability of a slope (that is the dynamic safety factor), it can

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be drawn that a slope can be considered unsure even if the mean values of the dynamic safety factor is ≅1. Moreover under drained conditions, have been carried out the ranges of dynamic safety factor values (table 5, in italic) according to their stochastic characteristics. It can be pointed out that for PHA=0.04g, the minimum dynamic safety factor value is less than 1 (it means it cannot be acceptable) unless its mean value can be accepted.

Table 6: Relationship between Reliability Index (β) and Probability of Failure (p_f) - U.S. Army Corps of Engineers [1].

Reliability index β	Probability of failure $p_f=\Phi(-\beta)$	Expected performance level
1.0	0.16	Hazardous
1.5	0.07	Unsatisfactory
2.0	0.023	Poor
2.5	0.006	Below average
3.0	0.001	Above average
4.0	0.00003	Good
5.0	0.0000003	High

As the second stage of the study the stability conditions of the slope have been investigated in terms of maximum permanent displacements calculated by means of the sliding block method. In table 7 the order of magnitude for critical permanent displacements as regard the triggering of sliding mechanism are reported from literature. Displacements ranking from 10cm to 100cm can, according to the type of soils, induce instability while displacements higher than 100cm certainly cause instability. The permanent displacements induced by the seismic action on the slope both under drained and undrained conditions were therefore calculated for the studied slope, by means of Ambraseys and Menu [3] expression (see eqn.1). The results of probabilistic analysis of permanent displacements are shown in tables 8-9, accounting for the lognormal distribution of the permanent displacements. The probabilistic approach evaluates permanent displacements for 50%, 10% and 5% of probability of exceedance. Under drained conditions (table 8), displacements are greater than 10cm for PHA values belonging to the Italian seismic code range assigned to the seismic zone 3.

Table 7: Limit values for permanent displacements calculated by Newmark's method to cause failure - D'Elia [9].

Displacement (cm)	Effects on slope stability
<10cm	Not relevant.
10-100cm	Relevant cracking associated with reduction in soil shear resistance causing failure during or after seismic shaking.
>100cm	Destructive movements.

Let us fix the PHA value and compare each other the displacements resulting from 50%, 10% and 5% of probability of exceedance. The displacements higher

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than 50cm relate to PHA values higher than 0.15g, which is the maximum PHA value assigned for zona 3 by law. That means even though important permanent displacements can be registered in Pomarico, they will never reach destructive values. However it's useful to notice that the mean values of the calculated maximum displacements (corresponding to the 50% of probability of exceedance) is less than the half with respect to those corresponding to 10% of probability of exceedance. Consequently to take account of mean values only is not safety at all. If we consider the case of undrained soils (table 9), we see that displacements higher than 100cm are always registered in case of PHA equal to 0.05g. Such results can be better understood by noticing that the critical accelerations of Pomarico slope for drained and undrained conditions are of different orders of magnitude:

$$k_{c(\text{undrained})} = 0.0018\text{g} \qquad k_{c(\text{drained})} = 0.0245\text{g} \tag{5}$$

Table 8: Displacements for drained condition.

Drained condition			
	Displacements calculated for different PHA values		
PHA	P_f=50%	P_f=10%	P_f=5%
0.05g	0.17cm	0.42cm	0.54cm
0.083g	3.34cm	8.1cm	*10.4cm*
0.09g	4.36cm	*10.55cm*	*13.53cm*
0.125g	*10.39cm*	*25.16cm*	*32.26cm*
0.15g	*15.33cm*	*37cm*	*47.62cm*

Table 9: Displacements for undrained condition.

Undrained condition			
	Displacements calculated for different PHA values		
PHA	P_f=50%	P_f=10%	P_f=5%
0.0061g	3.35cm	8.11cm	*10.4cm*
0.0065g	4.13cm	*10cm*	*12.82cm*
0.0091g	*10.18cm*	*24.64cm*	*31.6cm*
0.02g	*43cm*	*104.02cm*	*133.62cm*
0.03g	*77.10cm*	*186.68cm*	*239.4cm*
0.05g	*150cm*	*363cm*	*465cm*

5 Concluding remarks

In the present work the dynamic stability of a slope, in Pomarico town, were carried out in terms of Dynamic Safety Factor and permanent displacements.

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Moreover the influence of inherent variability, related to dynamic resistance parameter values, on the reliability of the dynamic stability analysis has been investigated. Dynamic safety factors were evaluated by means of two limit equilibrium methods: Bishop and Morgenstern & Price. Permanent displacements were also calculated by the sliding block model. The permanent displacements corresponding to 50%, 10% and 5% of probability of exceedance were reported to draw the reliability of deterministic approach. Results have shown the differences in stability assessment according to the displacement method and the pseudo-static method. Attention must be paid to the inherent variability of resistance parameter which heavily affects the final values of displacements and dynamic factors of safety. Finally the probabilistic approach as the random field method can accomplish the risk of landslides which may be useful tool in urban planning.

References

[1] U.S. Army Corps of Engineers, *Engineering and design, introduction to probability and reliability methods for use in geotechnical engineering*, Engr. Tech. Letter No. 1110-2-547, Department of the Army, Washington D.C., 1997.

[2] Phoon K.K., Kulhawy F.H. & Grigoriu M.D., *Development of a Reliability-Based Design framework for transmission line structure foundations*, Jour. Geotech. and Geoenv. Engineering, **129(9)**, pp. 798-806, 2003.

[3] Ambraseys N.N. & Menu J.M., *Earthquake-induced ground displacements*, Earthquake Engineering and Structural Dynamics, **16**, pp. 985-1006, 1988.

[4] Newmark N.M., *Effect of Earthquakes on dams and embankments*, Geotechnique, **15(2)**, pp. 139-159, 1965.

[5] Vanmarcke E.H., *Random fields: analysis and synthesis*, Cambridge: Massachusetts Institute of Technology, 1983.

[6] Cherubini C. & Orr T.L.L., *Considerations on the applicability of semi-probabilistic bayesian methods to geotechnical design*, XX Convegno nazionale di geotecnica, Parma, 1999.

[7] Geoslope International Ltd., *Slope/W Define Version 4.21*.

[8] Babu Sivakuram G.L. & Mukesh M.D., *Risk analysis of landslides – A case study*, Geotechnical and Geological Engineering, **21**, pp. 113-127, 2003.

[9] D'Elia B., *Stabilità dei pendii in zona sismica*, Atti del Corso di Geotecnica II Ciclo: "Ingegneria geotecnica in zona sismica", Pesaro, 1998.

[10] Cherubini C., *Private communication*, 1994.

[11] Phoon K.K., Kulhawy F.H, *Evaluation of geotechnical property variability*, Can. Geotech. J., **36**, pp. 625-639, 1999.

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