4th International Conference on
Earthquake Geotechnical Engineering
June 25-28, 2007
Paper No. 1310

# PROBABILISTIC ANALYSIS OF EMBANKMENTS USING MONTE CARLO SIMULATION

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## ABSTRACT

The majority of earthquake engineering applications and especially the geotechnical ones, can be considered as “imprecise” scientific problems, due to various (geotechnical, seismic, mechanical and geometrical) inherent uncertainties that characterize them. Nevertheless, geotechnical engineering practice compromises with the use of deterministic simplifications due to their low computational cost and their minimal complexity. However, nowadays, probabilistic methods are gaining popularity due to the advances in computational resources and in numerical methods, since they offer a more precise way to determine the seismic vulnerability of structures and/or geostructures. This paper presents an efficient methodology for establishing fragility curves, as an outcome of the probabilistic slope stability analysis, of large-scale embankments under seismic loading conditions.

Keywords: Seismic probabilistic analysis, slope stability, fragility curves, Monte Carlo simulation

## INTRODUCTION

In earthquake engineering applications and moreover geotechnical problems serious uncertainties exist, for instance those related to mechanical and geometrical parameters, seismic hazard intensity, etc. However, deterministic simplifications are usually the viable solution in engineering practice due to the various difficulties to quantify the aforementioned uncertainties. On the other hand, despite their increased computational complexity and cost, probabilistic techniques are gaining popularity recently (due to the advances both in computational resources and numerical methods) (e.g. HAZUS, 1999), since they offer a more precise way to determine both the hazard level and the structure’s or the geo-structure’s performance and thus the evaluation of its seismic vulnerability, in the framework of the state-of-the-art Performance-based Earthquake Engineering (PBEE).

A very useful tool for interpreting more realistically (compared to the standard safety factor calculation) the seismic risk and the vulnerability assessment of an embankment (or any type of geo-structure, or structure in general) over a large range of seismic hazard intensity levels is the construction of its fragility curves. A fragility curve represents the probability that the embankment exceeds a given damage level (for example, by exceeding its strength capacity, or a predefined displacement limit along its surface) as a function of a certain measure of the severity of the earthquake (e.g. peak ground acceleration). There are two basic approaches to create a fragility curve:

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either empirically, by using damage data available from past earthquakes, or numerically. The latter approach is most frequently (and also in this study) performed via the Monte Carlo simulation technique, which is a very simple and generally applicable statistical method that involves multiple analyses of the employed numerical model.

This paper presents efficient empirical and numerical methodologies for creating fragility curves, as an outcome of the probabilistic pseudostatic slope stability analysis, of large-scale embankments under seismic loading conditions. The proposed methodology involves the consideration of random variability of soil mechanical properties and the embankment's geometry, as well as the seismic intensity levels (in terms of peak ground acceleration).

## MONTE CARLO SIMULATION TECHNIQUE IN GEOTECHNICAL PRACTICE

Monte Carlo simulation (MCS) technique is a popular method for handling various reliability analysis problems. It is very frequently the most viable solution when the system to be analyzed becomes too complex and it can't be solved using simpler methods of reliability analysis, such as various reliability index methods. However, the latter methods should be used when the problems are relatively small and simple, as the MCS approach is usually much more time consuming. In Monte Carlo simulations each random variable is represented by a probability density function and repeated "standard" system analyses are performed by changing the values of the random variables using a random number generator. In order to obtain an accurate MCS solution a rather big sample of simulations, i.e. many thousands of these conventional analyses must be performed. In order to improve the computational efficiency of MCS a lot of sampling techniques, also called variance reduction techniques, have been developed. Among them, the most popular are the importance sampling, adaptive sampling technique, stratified sampling, latin hypercube sampling, antithetic variate technique, conditional expectation technique.

As it was aforementioned, when performing Monte Carlo simulations each random variable must be represented by a probability density function, while on the other hand the deterministic variables are represented as constants. In geotechnical engineering there are four probability density functions that are most commonly used: uniform distribution, triangular distribution, normal distribution, and lognormal distribution. Other probability density functions could be used provided that there were test data that matched those functions. Table 1 lists the most important probabilistic parameters that are used in geotechnical engineering and the probability density functions that typically best represent those parameters. The four probability density functions typically used in geotechnical engineering are briefly discussed below (USACE, 2006):

a. *Uniform Distribution*. The uniform distribution is the simplest type of distributions. All that is needed to describe it is the high and the low value of the random parameter. The uniform distribution gives the probability that observation will occur within a particular interval when the probability of occurrence within that interval is directly proportional to the interval length. If there is inadequate available information for a probabilistic variable, the principle of insufficient reason states that "the uniform distribution should be used". In addition, the uniform distribution is used for the generation of random numbers.

b. *Triangular Distribution*. A triangular distribution is used when the smallest value (most pessimistic outcome), the largest value (most optimistic outcome), and the most likely value are known. The triangular distribution is the most commonly used distribution for modeling expert opinion.

c. *Normal Distribution*. The normal distribution is the basic distribution in statistics. The central limits theorem states "the sum of many variables tends to be normally distributed". Most things in nature tend to be normally distributed. Consequently, the normal distribution is an appropriate model for many but not all physical phenomena. Furthermore, this distribution is easy to use because of the many standardized tables available.

d. *Lognormal Distribution*. The lognormal distribution is the logarithm of the normal distribution. As such it best represents processes which are the product of many random variables. This distribution is used when the value of the variable cannot be less than zero, since the extreme values of a normal distribution can be negative.

**Table 1. Soil parameters and corresponding probability density functions (USACE, 2006)**

| PARAMETER | PROBABILITY DENSITY FUNCTION | REFERENCE |
|---|---|---|
| Variables that do not take negative values | LN | Lacasse & Nadim (1996) |
| Unit Weights | N | |
| Cone Resistance<br>Sand<br>Clay | <br>LN<br>N / LN | <br>Lacasse & Nadim (1996)<br>Lacasse & Nadim (1996) |
| Undrained Shear Strength ($S_u$)<br>Clay<br>Clayey Silt | <br>LN<br>N | Lacasse & Nadim (1996)<br>Lacasse & Nadim (1996) |
| Ratio $S_u/\sigma'_u$<br>Clay | <br>N / LN | <br>Lacasse & Nadim (1996) |
| Liquid and Plastic Limits | N | Lacasse & Nadim (1996) |
| Void Ratio & Porosity | N | Lacasse & Nadim (1996) |
| Overconsolidation Ratio | N / LN | Lacasse & Nadim (1996) |
| Friction Angle<br>Sand | <br>N | <br>Lacasse & Nadim (1996) |
| Limited Information<br>High & Low Only<br>High, Low & Most Likely<br>No Information | <br>U<br>T<br>U | <br>Mosher (1997)<br>Mosher (1997)<br>Hobbs (1997) |
| Construction Costs | N | Mosher (1997) |
| Distributions Resulting From<br>Summation<br>Multiplication | <br>N<br>LN | |
| Remarks :<br>N : normal distribution<br>LN : lognormal distribution<br>N / LN : normal and lognormal distribution<br>U : uniform distribution<br>T : triangular distribution | | |

The current paper presents efficient methodologies for the generation of fragility curves via MCS for two different types of embankments, one with fixed geometry (constant dimensions) and one with “varying” geometry (dimensions are altered, since they considered as probabilistic variables that are dependant to normal distribution) as it is presented in Figure 1. As it was previously mentioned MCS uses a random number generator to select the value of each random variable using its probability density function. For each simulation (embankment’s pseudostatic analysis) a factor of safety (FOS) is calculated either numerically (using an “in-house” Fortran program or a commercial geotechnical package), or empirically. Each factor of safety is used to develop a probability density function, while the number of unsatisfactory performance events is also determined. The iterations are repeated thousands of times until the process converges. From the calculated probability density function of the factor of safety, or simply from the ratio of “failure simulations” (i.e., when FOS is less than one) over the total number of simulations defines the probability of geostructure’s unsatisfactory performance $p_{exc}$ (i.e to exceed the target FOS) is given by:

$$p_{exc} = \frac{N_H}{N_{SIM}} \tag{1}$$

where $N_H$ and $N_{SIM}$ are the number of “failure” and total simulations, respectively.

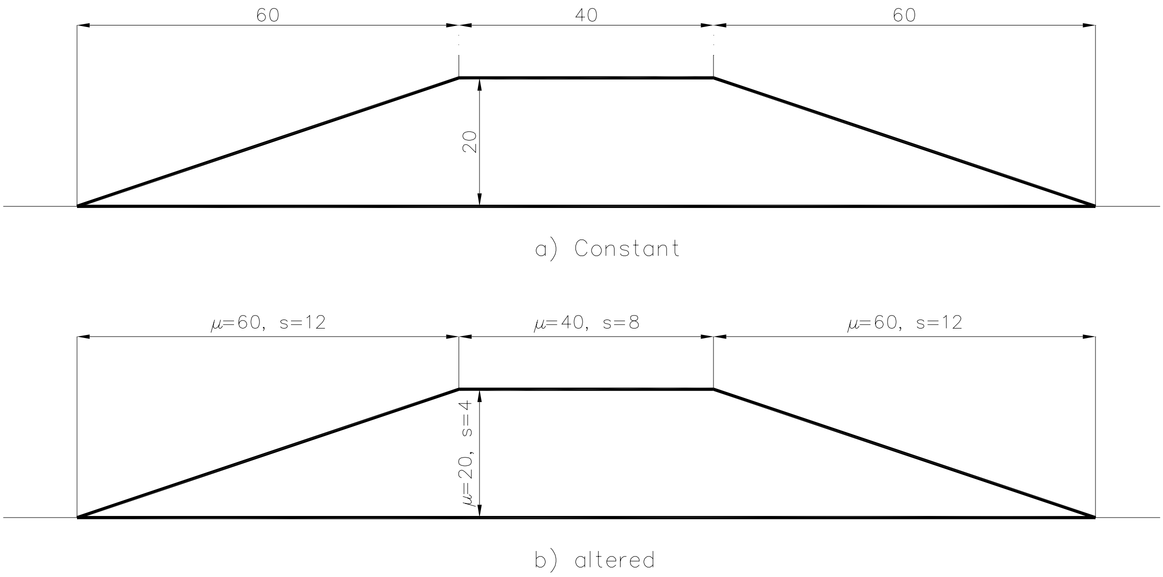


**Figure 1. The two types of embankments that were used for the generation of the fragility curves, a) with fixed geometry (*constant*) and b) with changeable geometry (*altered*)**

## FRAGILITY CURVES

For a given structure, it is possible to predict via deterministic methodologies the level of ground shaking which is necessary to achieve a target level of response and/or damage state. In addition, it is possible to assume in a similar manner the material properties and certain other structural attributes that affect the overall capacity of the system Such a deterministic assessment requires that certain assumptions should be made about the ground motion and site conditions, since both factors affect seismic demand. Nevertheless, values of these parameters are not exact; they undoubtedly posses various types of randomness and uncertainties. An increasingly popular way of characterizing the probabilistic nature of the seismic phenomena is through the use of the so-called *fragility curves*. Fragility curves provide the failure probability of a system (in this case embankments’ slopes) as a function of peak ground acceleration (PGA).

The construction of fragility curves is one of the most useful computational tools for determining the dynamic behavior of a slope (or any type of structure) over a large range of seismic levels. Specifically, a fragility curve provides the probability that the slope exceeds a given damage level as a function of some measurement of the severity of the earthquake (e.g. the peak ground acceleration). There are two basic approaches to establish a fragility curve for a slope: empirically by using damage data from past earthquakes, and numerically by Monte Carlo simulation involving multiple numerical analyses of the slope (Tantalla et al., 2001).

In this study both approaches have been used. The general assumption that the empirical fragility curves can be expressed in the form of a two-parameter lognormal distribution function and can be developed as functions of PGA in order to represent the intensity of the seismic ground motion has been used. Use of PGA for this purpose is considered reasonable since in the present study the pseudostatic slope stability approach is used. On the other hand, the procedure for the numerical fragility curves that was employed in the present investigation can be summarized as follows:

- Model the uncertainty in soil properties. For that purpose, soil properties values were generated and randomly assigned (according to a uniform distribution) to each simulation.
- Use different levels of peak ground accelerations to perform a pseudostatic slope stability analysis.
- Construct the fragility curves.

The scope of the present study was to examine in more detail how the factor of safety is “shaped” in probabilistic slope stability analysis using the pseudostatic method and then establish fragility curves for the safety factor correlated with the imposed peak ground acceleration. The models are studied under different PGAs (following the principles of Performance-based Earthquake Engineering) in order to examine more thoroughly the influence of the seismic hazard severity. Therefore, in order to cover a sufficient range of seismic intensity levels, five different cases (ranging from PGA 0.01g to 0.5g) were examined:

- Case I (minimum level): 0.01g
- Case II (low level): 0.05g
- Case III (low-to- medium-level): 0.10g
- Case IV (medium level): 0.20g
- Case V (severe level): 0.50g

For the aforementioned five seismic load cases, five Damage States (DSs) were considered for the embankments. Those DSs were established by taking them into account as limits for the probability of failure that exceed four different Factors’ of Safety for the embankments slope, namely FOS=1.0, 1.3, 1.4 and 2.0. To obtain discrete Damage States, according to the above perspective, a range of the Damage Index must be specified.

**Table 2. Correlation of FOS and the embankment’s Damage State**

| Damage State | Range of Damage Index |
|---|---|
| None | FOS > 2.0 |
| Slight | 1.4 < FOS <2.0 |
| Moderate | 1.3 < FOS < 1.4 |
| Extensive | 1.0 < FOS < 1.3 |
| Complete | FOS < 1.0 |

The correlation of the above FOS and the DSs of the slopes is a crucial factor for the construction of the fragility curves. Therefore the association presented in Table 2 is proposed and used in the present study. This classification follows the principles of the Greek Seismic Code (EAK, 2000) where the specific DSs are described as:

- FOS = 1.0 is the acceptable factor of safety for the stability of a slope for pseudostatic conditions,
- FOS = 1.3 is the acceptable factor of safety for the stability of a slope for static conditions when considering the existence of water,
- FOS = 1.4 is the acceptable factor of safety for dry static conditions, and
- FOS = 2.0 is the factor of safety for total stability of the slope without the appearance of any kind of damage.

Brief descriptions regarding the creation of fragility curves, both empirical and numerical, are presented in the subsequent sections.

## EMPIRICAL CALCULATION OF FRAGILITY CURVES

For the empirical evaluation of the fragility curves, a number of methodologies have been proposed, mostly for concrete frame structures and bridges. In the present study, an attempt has been made to use the methodology for the empirical fragility curves developed for the concrete structures in geotechnical applications. The probability of reaching or exceeding a given Damage State can be modeled as a cumulative lognormal distribution. For a specific Damage State of the geostructures, given the Factor of Safety, the probability of reaching or exceeding a DS, is modeled as (Shinozuka et al., 2000; UTCB, 2006):

$$F(DS \mid PGA) = \Phi\left[\frac{1}{\beta_{tot}} \ln\left(\frac{PGA}{PGA_{DS}}\right)\right] \tag{2}$$

where:
F( ) = is the probability that the DS will be equal or exceed the target range of Damage Index (see Table 2),
$\Phi$ = is the standard normal cumulative distribution function,
$\beta_{tot}$ = is the standard deviation of the natural logarithm of factor of safety of each DS,
PGA = the peak ground acceleration, and
$PGA_{DS}$= the median value of PGA at which the geostructure reaches the threshold of the Damage State

Median values of fragility curves are developed for each DS: None, Slight, Moderate, Extensive and Complete). Structural or geostructural vulnerability is characterized in terms of Factor of Safety. The factor $\beta_{tot}$ is modeled as the combination of three contributors to damage variability: $\beta_C$, $\beta_D$ and $\beta_{M(Sds)}$, as described in the following equation:

$$\beta_{tot} = \sqrt{\beta_C^2 + \beta_D^2 + \left(\beta_{M(Sds)}\right)^2} \tag{3}$$

where:
$\beta_{tot}$ = is the lognormal standard deviation that describes the total variability for each DS,
$\beta_C$ = is the lognormal standard deviation parameter that describes the variability of the capacity curve,
$\beta_D$ = is the lognormal standard deviation parameter that describes the variability of the demand spectrum,
$\beta_{M(Sds)}$ = is the lognormal standard deviation parameter that describes the uncertainty in the estimation of the median value of the threshold of each DS.

The value of $\beta_{tot}$ was initially proposed by Dutta & Mander (1998), as an outcome of a theoretical approach which was confirmed afterwards via empirical fragility curves. When there is no accurate data to determine the specific value of $\beta_{tot}$, it usually takes values in the range of 0,30 to 0,70. The lower value, 0,30, is used for the nuclear structures and components because of the greater uncertainties that exist and mostly the increased safety demands that are imposed in such structures (NRC, 2006), while the upper limit, 0,70, is used for concrete structures. Due to the major uncertainties that are present in a geostructure, the value of $\beta_{tot}$ = 0,30 has been used in this study.

**Table 3. Median factor of Safety for each PGA level for the two examined embankment types**

| PGA LEVEL | *CONSTANT* | *ALTERED* |
|---|---|---|
| 0.01g | 1.9747 | 2.0445 |
| 0.05g | 1.7348 | 1.7896 |
| 0.10g | 1.5248 | 1.5464 |
| 0.20g | 1.1955 | 1.2368 |
| 0.50g | 0.9854 | 0.9934 |

The median value of the FOS for the slope stability of the investigated embankments for each intensity level, as it evaluated through a rough analysis with few simulations for each PGA level (10 simulations), are presented in Table 3. The empirical fragility curves that were estimated for the chosen Damage States are presented in Figures 2 and 3, for the embankment with the fixed geometry (*constant* model of Fig. (1a)), and for the embankment with the variable geometry (*altered* model of Fig. (1b)), respectively. By observing these figures, it is obvious that the empirical fragility curves for the both types of embankments have very small differences in their lower and upper intervals, apart from the region of moderate PGAs and medium DSs.

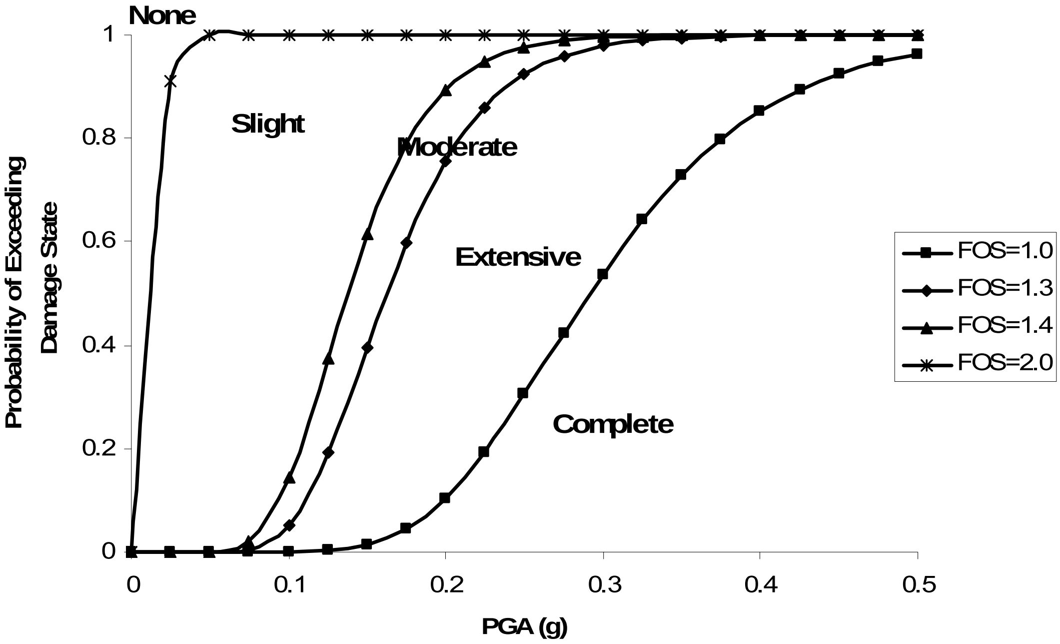


**Figure 2. Empirical fragility curves for the *constant* embankment for the five Damage States**

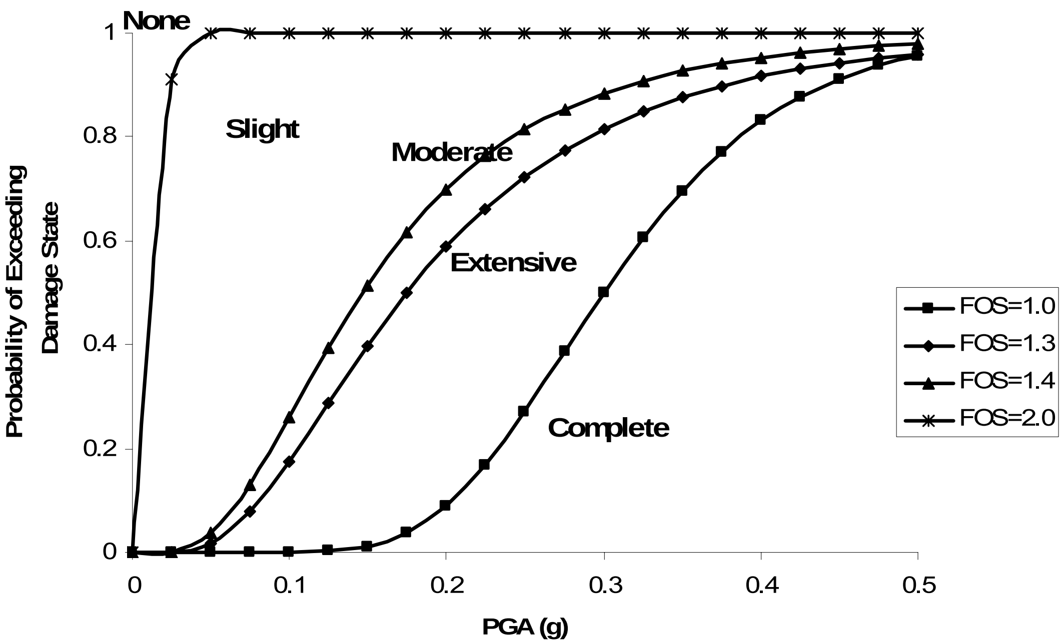


**Figure 3. Empirical fragility curves for the *altered* embankment for the five Damage States**

## NUMERICAL CALCULATION OF FRAGILITY CURVES

Any type of geo-structure, such as earth-fill dams, highway embankments, or solid waste landfills are of extremely high engineering interest due to the high risk (socio-economical, environmental, etc) related to their potential failure. As recent experience has shown, in regions with high seismicity the aforementioned risk is more intense. In order to study the seismic response of an embankment, as well as the related seismic stability issues and establish fragility curves as an outcome of the probabilistic slope stability analysis, it is important to determine the effect of the seismic excitation on the geostructure's dynamic response via its fragility curves.

Engineering practice in seismic slope stability analysis is based on three categories of methods; namely: *stress-deformation* analysis, *pseudo-static* analysis, and *permanent deformation* analysis. *Stress deformation* analyses are mainly performed utilizing the finite element method with the application of sophisticated constitutive models to describe the potential nonlinear material behavior. However, the parameters required for the implementation of the models are not easily or accurately quantified in the laboratory or in situ. On the other hand, a crude estimation of seismic slope stability is obtained through *pseudostatic analysis*. Based on limit equilibrium methods of static slope stability analysis, and including horizontal and vertical inertia forces, the results are provided in terms of the minimum *factor of safety*. The basic limitation of this method is the selection of the proper value of the seismic coefficient, as it controls the inertial forces on the soil masses. Finally, *permanent deformation* analyses are based on the calculation of seismic deformations through the simple method proposed by Newmark (1965), known as sliding block analysis.

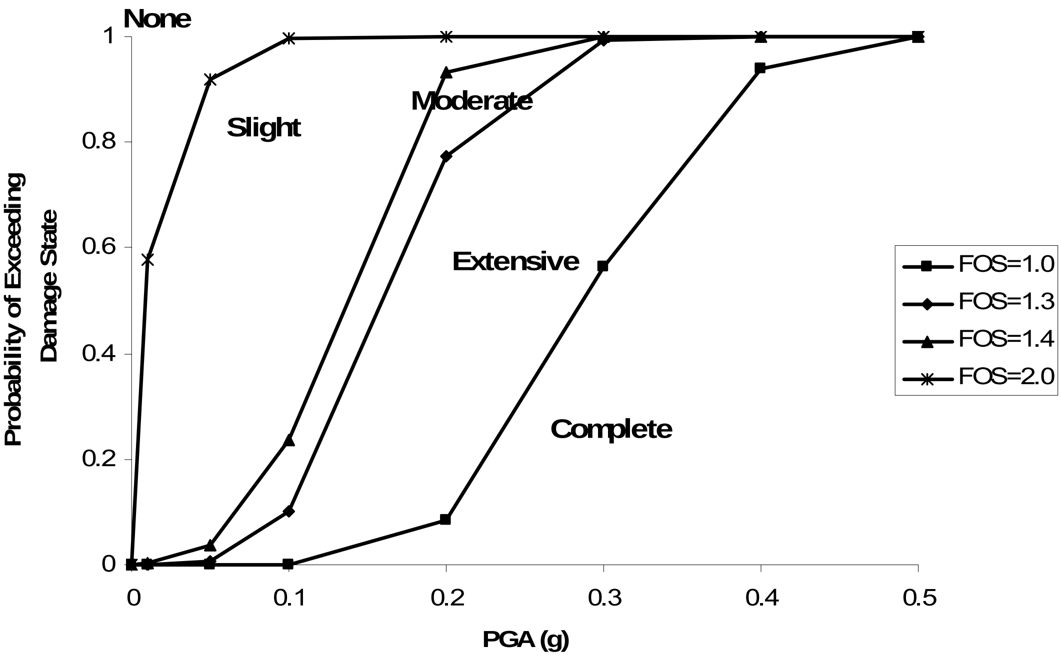


**Figure 4. Analytical fragility curves for the embankment with the constant dimensions for all Damage States**

In modern seismic codes (e.g. Eurocode 8 (EC8, 2004); Greek Seismic Code (EAK 2000)), seismic stability analysis is usually performed via the pseudostatic method, which is conducted using a value of seismic coefficient equal to a specific portion of the design peak ground acceleration at the site of interest. In this way, the dynamic response of the structure is not taken into account, resulting to incapability of predicting the actual response and stability of the geo-structure during a moderate or severe seismic event.

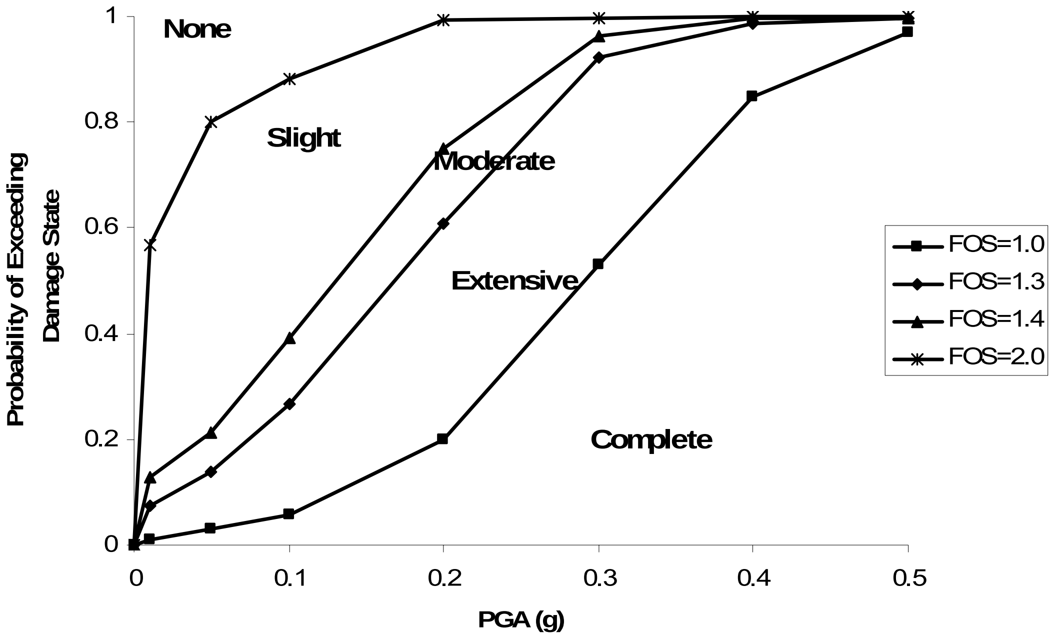


**Figure 5. Analytical fragility curves for the embankment with the *altered* dimensions for all Damage States**

In the current investigation numerical analyses were initially performed using an "in-house" Fortran program that utilized Bishop's method of slope stability analysis in conjunction with MCS technique. It was capable of modifying the geometrical and the mechanical characteristics of the embankment as well as the acting PGA. The normal distributions used for the aforementioned random variables are presented in Tables 4 and 5.

**Table 4. Normal distribution of the soil properties that were used for the analysis**

| SOIL PROPERTY | DISTRIBUTION | MEAN VALUE | STANDARD DEVIATION |
|---|---|---|---|
| Cohesion (kPa) | Normal | 5 | 10% |
| Friction angle (°) | | 30 | |
| Specific weight (kN/m$^3$) | | 22 | |

**Table 5. Normal distribution of the PGAs that were used for the analysis**

| CASE | DISTRIBUTION | MEAN VALUE | STANDARD DEVIATION |
|---|---|---|---|
| I | Normal | 0,01g | 10% |
| II | | 0,05g | |
| III | | 0,10g | |
| IV | | 0,20g | |
| V | | 0,50g | |

From the two different models examined (see Fig. 1), the one with the constant embankment's dimensions and the one with the varying dimensions (distributed according to the normal distribution), fragility curves were produced, as depicted in Figures 4 and 5. The slight differences in the fragility curves between the two different models can be attributed to the variation of the embankment's dimensions. In other words, the embankment may become higher with steeper angle thus more vulnerable, or it can become smaller and/or wider with milder inclination and consequently to become more stable. However, it has to be noted that the variation of fragility curves is again in the medium intervals, and in any case is much less than the variation of the geometry.

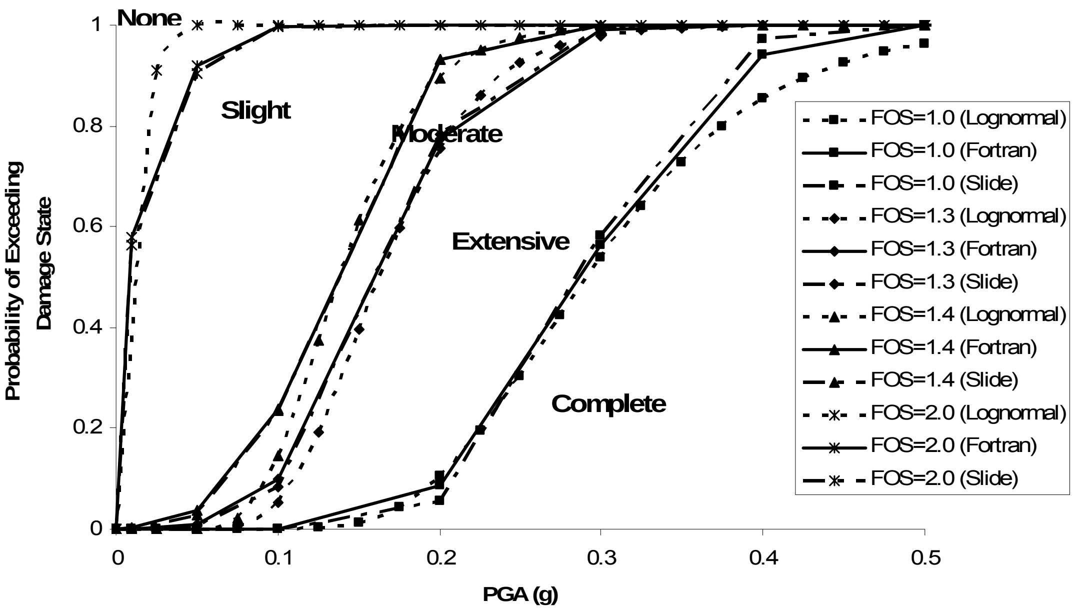


**Figure 6. Comparison between the empirical (Lognormal) and the analytical (Fortran and Slide) fragility curves for the embankment with the *constant* dimensions for all Damage States**

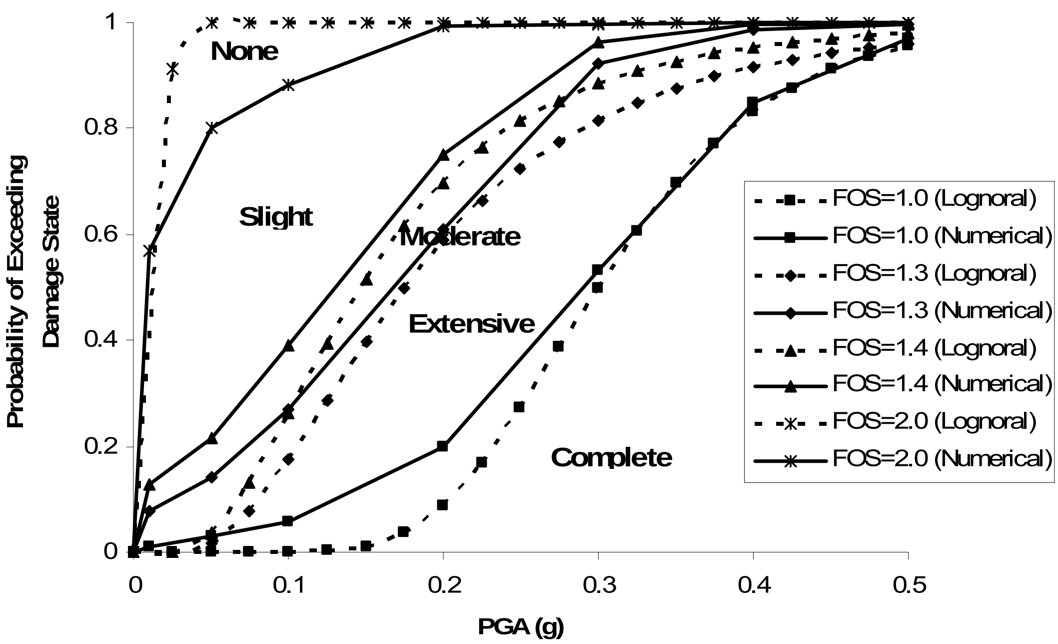


**Figure 7. Comparison between the Lognormal (empirical) and the numerical fragility curves for the embankment with the *altered* dimensions for all Damage States**

## COMPARISON BETWEEN EMPIRICAL AND NUMERICAL FRAGILITY CURVES

After the generation of both empirical and analytical fragility curves, the two types of curves are compared in order to verify the two methodologies. The fragility curves generated from the two types of analysis for both models are presented in Figures 6 and 7. As it is shown in Figure 6 numerical calculations have also been performed with the commercial program SLIDE (2006), so as to validate the results of the study from the empirical and the numerical calculations. The calculations executed with SLIDE were done only for the embankment with the constant dimensions due to limitations of the program (it does not have the ability to model changeable geometry). It can be observed that the proposed numerical methodology produces identical results with the ones obtained using SLIDE.

When comparing the empirical with the analytical fragility curves, for both the embankment types, it can be concluded that the estimation of the fragility curves (empirical) is accurate, especially when the embankments dimensions are fixed (*constant* model). On the contrary, they present a significant discrepancy when the dimensions of the embankments are also considered as random variables (*altered* model), following a normal distribution. The tolerance between the empirical and the analytical median probability of exceeding a specific Damage State ($p_{exc}$) for each PGA level and each FOS, is presented in Table 6. By examining the results in Table 6 it is evident that the variation between numerical and empirical fragility curves for the embankment with the altered dimensions is mainly at the "low" PGA levels (0.05g and 0.10g) and less for the medium PGA level (0.20g), except from the case when the stability of the embankment is marginal (i.e. for FOS equal to one).

**Table 6. Tolerance of "Probability of exceeding Damage State" for each PGA level and each FOS for the numerical and the empirical fragility curves for the embankment with *altered* dimensions**

| FOS | PGA | MEDIAN $p_{exc}$ NUMERICAL | MEDIAN $p_{exc}$ EMPIRICAL | % |
|---|---|---|---|---|
| 1.0 | 0.05 | 0.030 | 0.000 | 100.00 |
| | 0.10 | 0.058 | 0.000 | 99.78 |
| | 0.20 | 0.198 | 0.088 | 55.32 |
| | 0.30 | 0.532 | 0.500 | 6.02 |
| | 0.40 | 0.848 | 0.831 | 2.01 |
| | 0.50 | 0.970 | 0.955 | 1.49 |
| 1.3 | 0.05 | 0.140 | 0.018 | 86.99 |
| | 0.10 | 0.268 | 0.175 | 34.70 |
| | 0.20 | 0.608 | 0.588 | 3.25 |
| | 0.30 | 0.922 | 0.816 | 11.50 |
| | 0.40 | 0.988 | 0.916 | 7.26 |
| | 0.50 | 0.996 | 0.960 | 3.59 |
| 1.4 | 0.05 | 0.214 | 0.036 | 83.08 |
| | 0.10 | 0.392 | 0.261 | 33.30 |
| | 0.20 | 0.750 | 0.698 | 6.93 |
| | 0.30 | 0.962 | 0.884 | 8.10 |
| | 0.40 | 0.996 | 0.953 | 4.30 |
| | 0.50 | 0.998 | 0.979 | 1.83 |
| 2.0 | 0.05 | 0.800 | 0.999 | 24.98 |
| | 0.10 | 0.882 | 1.000 | 13.38 |
| | 0.20 | 0.992 | 1.000 | 0.81 |
| | 0.30 | 0.998 | 1.000 | 0.20 |
| | 0.40 | 1.000 | 1.000 | 0.00 |
| | 0.50 | 1.000 | 1.000 | 0.00 |

## CONCLUSIONS

This paper presented efficient methodologies for establishing fragility curves, as an outcome of the probabilistic slope stability analysis, of large-scale embankments under seismic loading conditions. The proposed methodology involves the consideration of random variability of soil mechanical properties and the embankment's geometry, as well as the seismic intensity levels. Both empirical and numerical methodologies were used and compared for the creation of geostructure's fragility curves. It was found that there is a good consistency between the two types of fragility curves when the geometry of the embankment remains constant. On the other hand, the empirical and the numerical fragility curves do not coincide when the dimensions of the geostructure are also considered as random variables. This is due to the fact that the degree of uncertainty becomes greater, since the number of the probabilistic variables are increased from four (c, φ, γ, PGA) for the embankment with the *constant* geometry, to eight (c, φ, γ, PGA and the embankment's dimensions) for the embankment with the *altered* geometry.

## ACKNOWLEDGEMENTS

The financial support from the European Union and the General Secretary of Research of Greece (PENED–03/454 research project) is gratefully acknowledged.

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