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Failure of Slopes and Embankments Under Static and Seismic Loading

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Summary

The stability of slopes and embankments under the influence of static and seismic loads has been the subject of study for many researchers. This paper presents the mechanisms and causes of landslides as well as the forms of failure of slopes and embankments under static and seismic loading, with examples of failures from both Greek and international space. There is also mention to measures to protect and stabilize landslides, categories of slope stability analysis, and methods of seismic impact analysis**.** What follows is the determination of tolerable movements based on the caused damage on natural slopes, dams and embankments and an attempt is made to connect them with the vulnerability curves that are one of the key elements of stochastic seismic hazard**.** Particular importance is given to the statistical parameters of the mechanical characteristics of the sloping soil mass and to the simulation of random fields necessary for solving complex geotechnical works. Finally, we compare the simulation and description of random fields and the L.A.S. method is observed to be the most accurate of all simulation methods. The L.A.S. algorithm in conjunction with finite difference models can demonstrate the large fluctuations in the factor of safety values and the permanent seismic displacements of the slopes under the effect of seismic charges whose time histories are known.

Keywords: slope; embankment; failure; seismic action; tolerable movements; vulnerability curves; simulation; random fields; L.A.S. algorithm; fluctuations; permanent seismic displacements.

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1. Introductory Notions

The term slope stability describes the stability of an inclined soil formation or an artificial soil structure with a sloping, free surface. On each slope, the difference in level and the slopes in combination with the gravitational forces and the possible presence of water on the ground, create shear stresses inside the slopes, which are countered by the shear resistance of the soil. When growing tensions go beyond shear resistance, they lead to fracturing in the slope and a landslide, such as the landslide of Figure 1 on Taiwan's No3 motorway in 2010.

Figure 1: Failure of the slopes on motorway No. 3 of Taiwan (2010). For the removal of the soil, 50 excavators, 100 trucks and 1000 workers had to be used for 20 days [1]

The volatility of slopes leads to the displacement of soil mass downstream (loss of stability), known as landslide and constitutes a significant risk to human activities and is often accompanied by the destruction of property, injury and loss of life. The need therefore to assess stability has led to the development of analytical methods pertaining to either two or three dimensions. Some of the basic causes that can trigger the failure of a slope are the extreme sloping angle of the free surface, the low shear strength of soil or of one layer of soil, the reduction of soil shear strength due to an increase in pore water pressure, the imposition of external forces, digging at the base of the slope or some earthquake. Types of failure typically encountered in loss of stability are shown in Figure 2, including failures due to (a) erosion of slopes (b) erosion due to river (c) filling of cracks with rainwater (d) gravity and earthquake (e) weak layers within the soil formation (f) stresses on the upper surface of the slopes and (g) excavation at the base of the slopes.

In any place where instability has appeared in the past, previous soil masses have created a "first slip", which results in significant deformations, usually along a slip surface. Initially, the greater strength, which is triggered in parts of the slip surface, is the maximum, but after a certain amount of deformation, it changes to the critical state strength. As the available strength decreases in these sections, the maximum strength develops in other parts of the sliding surface until the strength across the entire surface is at a critical state. This process is called progressive failure. We can see that relying on maximum strength is a dangerous assumption, and to assume a critical moment resistance is perhaps a relatively conservative approach. For this reason, it is important to know the types of landslides, as well as the failure mechanisms they present to proceed with the analysis of stability and the calculation of a satisfactory safety factor.

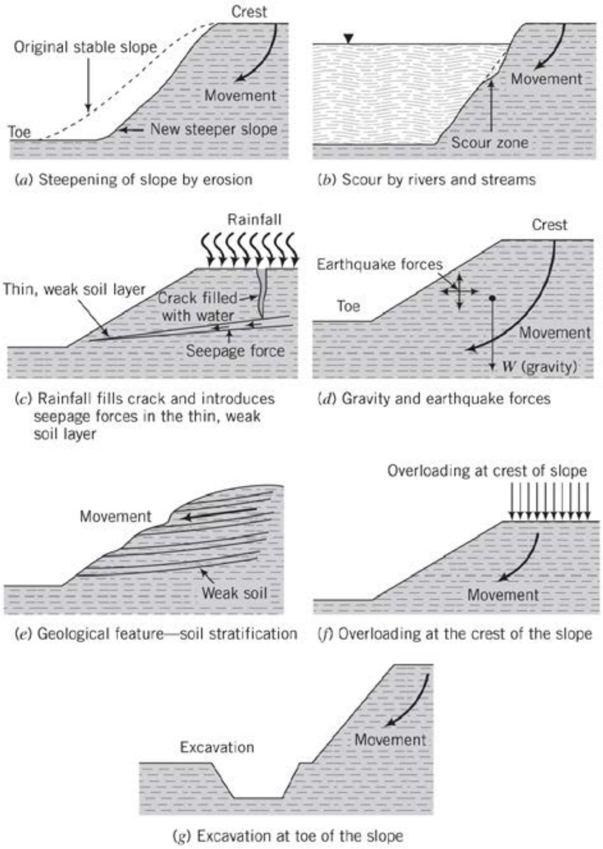

Figure 2: Possible forms of slope failure due to: (a) slope erosion (b) erosion of the slopes due to river (c) filling of cracks with rainwater (d) gravity and earthquake (e) weak thin layers (f) stress stress (g) digging at the base [2]

2. Landslides

2.1 Definition of a landslide

By sliding we refer to the downward slow or rapid movement of a soil mass due to gravity. A landslide is triggered when the shear stresses developed inside the soil exceed those that the soil can resist. Landslides can be caused by the fluidization of fine grain sandwich layers, or due to a general failure, in combination with increased loads due to an earthquake, increasing the pore pressure and reducing the available shear strength of the soil. The latter is a fairly common situation in many powerful earthquakes. It should be noted that in slopes, as with the gravity retaining walls, the vertical component of the seismic oscillation, by reducing the active weight, acts negatively on the stability and must, therefore, be examined with particular care. Figures 3 and 4 show typical examples of general failure in two of the major earthquakes that have struck our country over the past 50 years [3].

Figure 3: Slope Failure and traffic obstruction (Lefkada 2003 earthquake)

Figure 4: Pathway slipping of the provincial road (Kozani earthquake 1995)

2.2 Causes of landslides

The main causes of landslides are as follows:

- The weight of the material itself, as well as its lithological composition
- Dynamic loading (earthquakes)
- Flow strengths (forces)
- Change in the mechanical properties of the soil material
- Change of slope geometry
- Climatic changes
- Charging with embankments - constructions - technical works
- Effects of groundwater - surface water, frost
- Effect of vegetation
- Combination of the above

2.3 Basic types of slope failure

From time to time, various geological and engineering systems have been proposed by geologists and engineers, based on a variety of parameters, one of which is the Barnes (1978) slope movement. The main mechanisms of slope failure are shown in Figure 5.

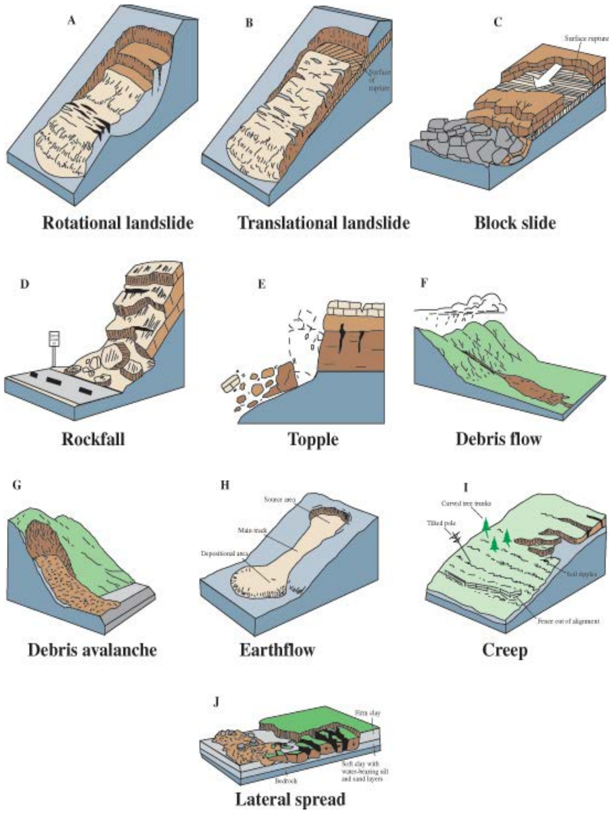

Figure 5: Mechanisms of slope failure **[4]**

2.4 Examples of failures

Below a few representative faults of road embankments and railway tracks are shown.

A) Malakasa landslide: During the first hours of 18/02/1995, this massive landslide manifested itself in the 36th km of the Athens - Lamia road. This movement of the slopes caused very significant material damage and cut off both the road and rail communications of the capital with Northern Greece.

Figure 6: The damaged road of the Athens-Thessaloniki National Road from the landslide of Malakasa.

Figure 7: Deformation of the railway line from the landslide of Malakasa.

B) Niigata-Ken Chuetsu (Japan): On October 23, 2004, a 6.6 magnitude on the Richter scale earthquake was recorded in the above-mentioned region of Japan. Much destruction had been recorded in transport networks due to soil failures because of slope slides.

Figure 8: Road damage during the Niigata - Ken Chuetsu earthquake (Japan)

Figure 9: Road failure during the Niigata –Chuetsu earthquake (Japan, 2004)

C) Kocaeli (Turkey): An earthquake measuring M=7.4 on the Richter scale was recorded on 17/08/1999. Major disasters were observed in the transportation and infrastructure networks in the Kocaeli and Sakarya regions, mainly due to ground fractures. Also, many bridges had small to medium damage, while two bridges collapsed and serious damage was recorded in high-rise buildings.

Figure 10: Failure of a national highway in Turkey (Kocaeli, 1999)

3. Examples of land and embankment failures due to earthquakes

The probability of landslides under the influence of earthquake inertia forces depends on the combination of seismic loading and the pre-existing geological conditions. Of course, it is difficult to predict, identify and categorize such slippages and failures. The reason for this lies with the difficulties associated with the determination of reliable parameters of the materials on the slip surface, the exact and inadequate characterization of the behaviour of the materials under irregular circular loads, and the uncertainty associated with estimating the seismic charges which are never clearly known. Reference [5] Some real cases of landslides and embankments under seismic loading are described below.

Earthquake of Lefkada 2003

On August 14, 2003, a seismic vibration of M=6.2 magnitude and a focal depth of h = 10 km was recorded in the Ionian Sea west of the island of Lefkada at the height of Ag. Nikitas. The intensity of the earthquake in the city of Lefkada was VIII (EMS). In the same area, the acceleration measured was α = 0.42g. The earthquake caused extensive damage throughout the island of Lefkada. A significant number of rockfalls and landslides occurred along both the western axis of the island and the eastern part, albeit at a lesser degree. Figures 5.2 and 5.3 show images of landslides and rocks in the road network. Also, significant failures were observed in the ports of the island.

Figure 11: Lefkada: Landslides on the road network

Kobe Earthquake 1995

In January 1995, the Nikawa city of Japan suffered a great landslide from the Hyogo-ken - Nambu earthquake. The magnitude of the earthquake was 7.2 on the Richter scale, and the distance at which sliding from the tectonic fault occurred was less than 10km. Estimates indicate that the magnitude of the maximum acceleration in the area was about 0.5g. The slope consisted of sand and clay, while two-thirds of the slip surface were below the aquifer level. The type of slip was superficial and the volume of the material affected was about 110,000 to 120,000m^3. The movement distance was more than 80 m. Figure 12 below shows the area in which the slippage occurred

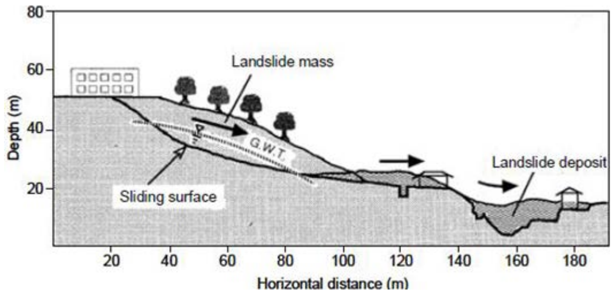

Figure 12: Nikawa slippage - cross section (Sassa and his colleagues 1996)

Kalamata Earthquake 1986

This is the earthquake that occurred in the city of Kalamata on September 13, 1986. The earthquake, albeit moderately strong, was of a M=6.2 magnitude and caused severe damage, while 20 people were killed. Reference [6] A site of characteristic failure (which occurred within a radius of 9 km from the epicentre) is shown in the following figure:

Figure 13: Landslides near the village of Ladas [7]

Northridge (Rinaldi) 1994 and Chile (1964) earthquakes

The Northridge earthquake took place on January 17, 1994, and was centered on Reseda, a settlement in the northern / central San Fernando Valley area of Los Angeles, California. It lasted about 20 seconds, had a magnitude of 6.7 on the Richter scale, and caused massive accelerations of 1.8 g (16.7 m/sec^2). The earthquake was felt in Las Vegas Nevada, about 360km from the epicentre. Figure 5.14 shows a landslide in Santa Monica, California, during the 1994 Northridge earthquake.

Figure 14: Landslide with residences in the city of Santa Monica, California, during the 1994 Northridge earthquake (Photo by P. Dakoulas 1994) [8]

Valdivia of Chile Earthquake (1960)

The earthquake in Chile on May 22, 1960 had a magnitude between M=9.4 - 9.6, a duration of 10 minutes and is the largest ever recorded. The epicenter was near Valdivia, 560 km south of Santiago. Figure 5.15 shows a landslide near Valdivia in the 1960 earthquake. The landslide shifted 30 10^6 yd^3 of ground material at a distance of 25 m vertically and 300 m horizontally. The cause of the landslide is the development of overpressure in sand "lenses" within the alluvial deposit lake.

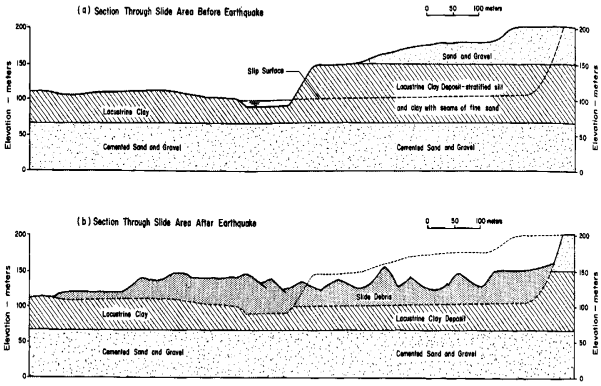

Figure 15: Landslide in Valdivia, Chile, during the earthquake of 1960. The landslide shifted 30x10^6 yd^3 of ground material at a distance of 25 m vertically and 300 m horizontally.

New Zealand Earthquake 2016

The New Zealand earthquake took place on November 15, 2016, and was M=7.8. The epicenter of the earthquake was between Christchurch and Kaikoura. In Figures 16 and 17, there are two of the many landslides that have caused major damage to the road and rail network in the country.

Figure 16: Landslides and destruction of the New Zealand rail track

Figure 17: Landslide and destruction of the New Zealand highway

4. Measures for the protection and stabilization of slope soils

Remedial measures include general works and construction of technical projects whose main purpose is the prevention of phenomena (preventive measures) or the restoration and stabilization of a soil movement (restoration or stabilization measures). Of particular interest are the measures to control and contain the landslides as shown below:

Addressing landslides

Control measures

- Surface drainage projects:
  - Crack seals
  - Surface water removal pipes
- Shallow drainage works:

- Underground drains
- Drainers
- Combinations of drainage gutters
- 'External' drainers
- Horizontal drainage drilling

- Deep drainage projects:

  - Vertical drainers
  - Horizontal drainage drilling
  - Large diameter drainage wells
  - Drainage tunnels

- Excavation
- Foot counterweights
- Coast-bed stabilization projects
- Retaining walls
- Piles
- Pivot pads
- Anchorages

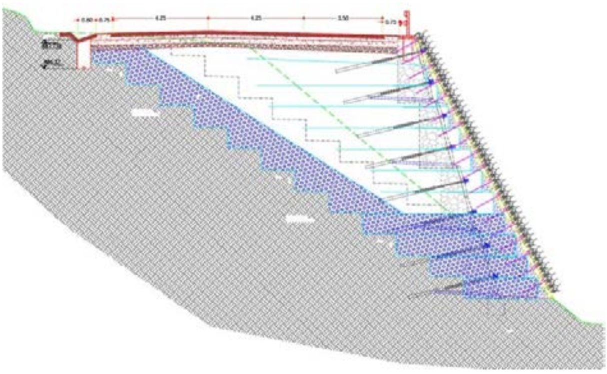

Figure 18: Combination of measures for the protection and stabilization of slopes: retaining walls, piled walls, armed embankments, torrent arrangement with gabions, trench supports.Source: OMIKRON KAPPA Consulting (Technical and earthworks, Romania-Central Europe 2005) [9]

5. Slope Stability Analysis

5.1 Categories of slope stability analysis methods

Ways to analyze slopes can be divided into two main categories:

a. Calculation of stability: The stability of the slopes is usually analyzed by the use of limit equilibrium methods. This analysis requires knowledge of the shear strength parameters of the geological material, but not knowledge of the stress-strain relation. As a measure of stability, in principle, the safety factor is calculated.

b. Calculation of movement: The status and magnitude of the movement of the sloping material is usually analyzed by the finite-element method and the finite-difference method. This method calculates the movements and the acting stresses in the slope body, taking into account the stress-strain relation with the use of appropriate elastoplastic soil simulations.

5.2 Stages of deformation of the slopes under the influence of seismic loading

Deformations of the slopes (both natural and artificial) follow the following three stages:

1. During the quake, gravity and seismic forces can cause instability and create a failure surface or even activate a pre-existing sliding surface, causing permanent displacements to the slope. The displacements at this stage are usually small and are controlled by: a) the magnitude and duration of the earthquake inertia forces; b) the sloping geometry; and c) the undrained resistance of the material moving during the earthquake. An exception is the case in which the remaining strength of the material to which the seismic forces are applied, is greatly reduced. Then the earthquake induced shifts can be large and uncontrollable.
2. Immediately after the earthquake, the second stage follows if the residual undrained shear resistance on the slip surface created by the earthquake is less than the resistance required to maintain the static equilibrium (e.g. if the geometry safety factor at the end of the earthquake Is less than one). The reduction of resistance forces results in an acceleration of the ground mass downward. Downward movements of the slope which were triggered by the earthquake will continue with a sliding foot outward movement and the speed of movement will provide the kinetic energy necessary to further transport the slope material. At this stage, the force that causes motion is gravity, while the resistance forces will continue to depend on the remaining undrained strength available, created during the first stage of motion. Note that the movements are great if the strength is significantly reduced or if the foot of the slope is minimally or none at all supported, due to unfavorable topographical conditions or great loss of strength.
3. The latter stage includes further shifts that may occur as a result of sliding and joining processes as well as destabilizing hydrostatic forces in the event of the creation of deep open cracks during the earthquake, which are then filled with surface soil or water. Additional movements may occur but are slow and correlate with progressive failure and soil strength under drainage conditions [10].

5.3 Methods of analysis of seismic effects

In the case of areas with increased seismicity, it is necessary to analyze the stability of the slopes including the seismic stress dynamic response. As a basis for the determination of the dynamic parameters (usually acceleration) of the seismic motion introduced in the analysis, the (statistically expected) design quake is included, which corresponds to the individual seismic ruptures of the influence area.

The most often used methods in the technical design are:

1. The pseudostatic method of analysis. It is a very conservative method and often gives safety factor values lower than those calculated by the static analysis.
2. The Newmark analysis method (1965), [11] which evaluates permanent displacements of the mass of the slopes based on the sloping cube slip.
3. Numerical simulation with FE or FD: Full numerical simulation based on the statute law of soil behaviour during an earthquake, while for the dynamic analysis the Finite Element or Finite Difference method is used to calculate the deformation of the slopes under seismic loading [12].

For this reason, the following procedure is proposed by Komodromos 2008 [13]:

- Solutions using numerical methods
- Reverse analysis - identification of vulnerable areas
- Select type and location of support measures
- Calculation of intensive support methods
- Determination of intensive-kinematic condition
- Design of projects

5.4 Maximum tolerable shifts of the slopes under seismic excitation

For the seismic design of natural slopes and terraces such as dams and embankments, the geotechnical engineer designs using the methodology of acceptable land movement that has prevailed in recent years as the basic design philosophy.

The prediction and control of the deformations and consequently of the movements of the geotechnical works under seismic loading is one of the main objects of geotechnical engineering. The calculation of the remaining soil movements is more complex, but it has the significant advantage of being directly linked to the damage caused to buildings and ground structures when they receive a seismic vibration.

It is obvious that almost all of the geotechnical structures have a certain limit on the movements that they can accept without failing, without, at the same time, altering their functionality. These movements are so-called *"maximum permissible movements"* which should be the largest that the geotechnical structure or the natural slope (calculated by use of the Newmark method, finite element analysis, etc.) can assume to be considered safe. Tolerable movements from seismic actions should be considered in conjunction with movements expected from other factors, such as soil settling from static loads, creep, etc. In addition, account should be taken of the importance of construction, any overlying structures, the needs arising immediately after the earthquake, the possibility of inspecting the ground-construction system or the slope (e.g. to determine the appearance of cracks) as well as the financial costs and the time required to repair the damage. For this reason, tolerable movements are not easy to delimit and quantify in a more general context of geotechnical constructions and to be included in regulations.

Matasovic (1991) performed a natural slope stability analysis (using the flysch strength parameters) with the static method, the pseudostatic method, the Newmark sliding solid upon an incline method, and the simplified Ishihara method. In this study, he pointed out the important problem of considering a level of tolerable movement because the behaviour of a slope during and after the seismic vibration is associated with the choice of the shear strength parameters of the material and the exact calculation of the seismic load. He adopted the limits of tolerable movements for natural slopes proposed by the State of Alaska's Geotechnical Evaluation Criteria Committee based on 2 very strong earthquakes, that of Alaska in 1964 and Mexico in 1985 as presented in the following Table:

Table 1: Determination of tolerable movements on the basis of the damage caused on natural slopes (Matasovic 1991) **[5]**

EFFECTS / DEFECTS	Tolerable movements (cm)
I. Destructive	300
II. Serious	90
III. Medium	30
IV. Small	15
V. Negligible	<3

The international experience presented in the above table showed the following:

- Displacements based on the 10 cm sliding solids on an inclined plane analysis **[11[** are considered to be unlikely to lead to landslides and destruction.
- Larger displacements of 10 to 100 centimeters can cause ground breakage or decrease in strength, resulting in the failure of the project.
- Finally, estimated movements of more than 100 centimeters should characterize the work as unstable.

In natural slopes, tolerable movement depends on the structures that are grounded on the slope or on the foot of the slope. If there are buildings, permissible movement is equivalent to that of the foundations, and if there are no structures, the allowed movement may be greater.

In small dams and embankments, movements of a few centimeters or even a few tens of centimeters may be tolerable, provided the continuity of the dam filter is not interrupted.

In road and highway embankments, the horizontal ground movement should be no more than about 5 cm, above which unacceptable deformation of the road surface is caused. In provincial road embankments, the permissible movement may be greater e.g. in the order of 10 centimeters, because: (a) the risk of an accident is lower due to the less frequent passage of vehicles and (b) traffic disturbance is of lesser importance.

Table 2 is particularly interesting in the research work of Gazettas and Dakoulas (1991) **[14]** concerning the safety of loose stone layered dams under the influence of seismic forces.

Table 2: Recommended slopes by Seed and his colleagues [152] [15] for CFR dams, depending on the seismicity of the area.

Earthquake Magnitude	Peak Crest Acceleration	Average DS Slope for Displacements of 2ft or more	Average DS Slope for Dispacements of 1 ft or less	Seismicity of the Area
6·5	<0·25g	1·35	1·4	
6·5	≈0·45g	1·4	1·4	Low
7·5	≈0·45g	1·4	1·4	to
8·5	≈0·45g	1·45	1·45	Moderate
6·5	≈0·75g	1·5	1·5	
7·5	≈0·75g	1·55	1·6	
8·5	≈0·75g	1·65	1·7	High
6·5	≈1·0g	1·55	1·55	
7·5	≈1·0g	1·6	1·65	Very
8·5	>1·0g	1·8	1·8	High

There are some significant attempts made towards a probabilistic approximation of the above calculations.

Travassarou (2006), Reference [16] after proposing a new empirical relationship for the calculation of permanent seismic displacements in slopes, stresses that the significant uncertainties surrounding the problem of permanent seismic displacements point to the usefulness of probabilistic calculation methods which take into account dispersion in the relevant parameters.

6. Vulnerability curves

Fragility curves have emerged in recent years as an indispensable tool for a number of purposes related to seismic risk management, such as calculating expected losses in future earthquakes, setting priorities for building and utility network reinforcement, seismic safety, etc.

"Vulnerability" refers to the behaviour of a compromised item, which is due to a variable intensity phenomenon. Especially in the case of an earthquake, a phenomenon quite common in our country, experience has shown that in addition to buildings, utility networks are also quite vulnerable to strong earthquakes. Failures that occur after an earthquake are due to the following phenomena caused by an earthquake:

- Terrestrial oscillation
- Intersections with fractures
- Subsidence in transition zones from better to worse soil
- Landslides, i.e. mass movement of the slopes due to failure of the slope soil because of fracturing along a surface
- Failures in public utility networks due to landslides are mainly due to a drop of rocks or landslides that entrains the network
- Liquefaction, i.e. the conversion of saturated, non-coherent soil from a solid to a liquid state

Vulnerability curves are one of the key elements of stochastic seismic hazard. They connect seismic intensity with the probability of approaching a level of failure or destruction (small, moderate, widespread, catastrophic) for each hazard element.

Significance is linked to functional, economic, social, and other criteria. Seismic hazard expresses the possibility of seismic vibration of a certain intensity at a specific time in the area under consideration. Seismic risk is calculated using the following mathematical formula:

[Risk] = [Hazard] x [Vulnerability] x [Importance] [17]

The development of vulnerability curves for road slopes was proposed in the European Safeland project by Pitilakis and his colleagues (2010**)** [3]. In this European program the Hazus vulnerability curves (NIBS, 2004) [18] were modified as a function of the maximum land acceleration taking into account the slope characteristics and using the Bray and Travasarou (2007) [19] model already mentioned. In the following table and in the following figures, the vulnerability curves for different seismic acceleration values and for an M=7.0 magnitude, as well as four different failure levels, are presented.

Table 3: Proposed parameters of vulnerability curves for road slopes (Pitilakis and his colleagues 2010) [3].

	Peak Ground Acceleration							
	ky=0.05		ky=0.1		ky=0.2		ky=0.3	
Damage states	Median (g)	β	Median (g)	β	Median (g)	β	Median (g)	β
slight/minor	0.16	0.40	0.30	0.35	0.55	0.35	0.80	0.30
moderate	0.28		0.48		0.85		1.20	
extensive	0.40		0.68		1.18		1.64	
complete	0.66		1.08		1.82		2.40	

where:

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Peak Ground Acceleration: maximum ground acceleration.

Median: mean PGA values for each vulnerability curve.

k_y: Yield coefficient: coefficient of seismic acceleration efficiency (i.e., seismic factor that brings to the slope a safety factor similar to that of pseudostatic analysis).

B: standard deviation parameter proposed in the Safeland urban road design.

Below are the slopes of road slopes for different levels of failure and different seismic accelerations.

PGA (g)

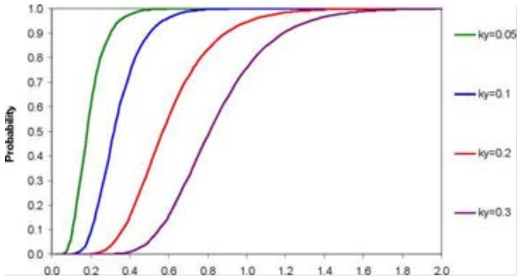

Figure 19: Road slopes- Minor damage state

PGA (g)

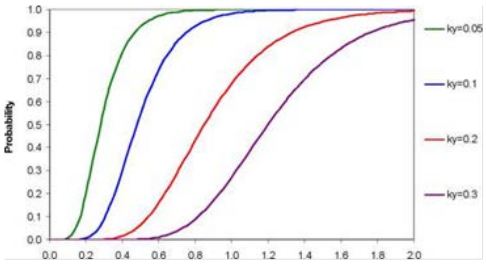

Figure 20: Road slopes-Moderate damage state

PGA (g)

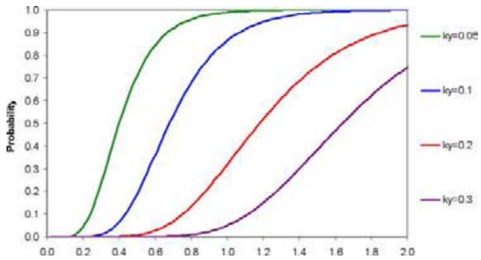

Figure 21: Road slopes- Extensive damage state

PGA (g)

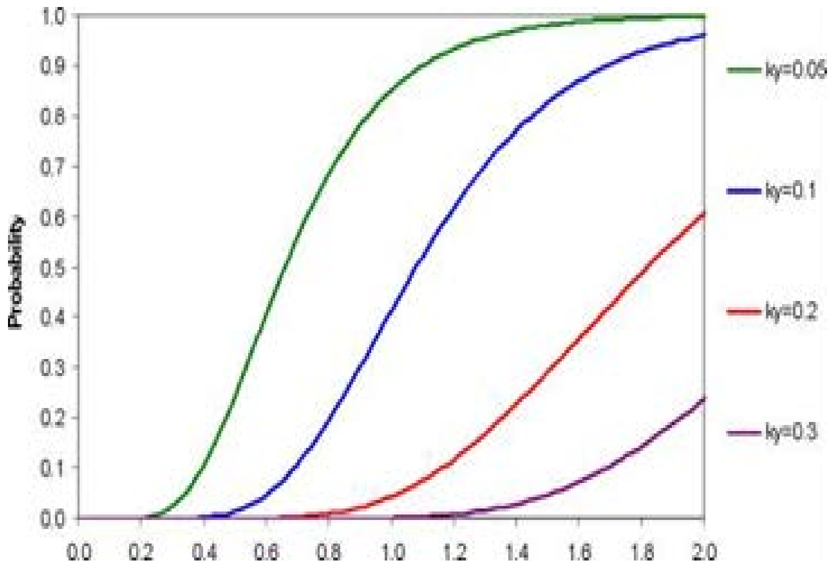

Figure 22: Road slopes- Complete damage state

7. Stochastic methods in slope stability analysis

Geo-technical design is one of the subjects of study for civil engineers, including a great deal of uncertainty because of the natural heterogeneity of soil materials and the limited extend of survey work [20].

Analysis of slope stability as a branch of the subject of geotechnical engineering is directly related to the use of stochastic methods that lead to quantification of uncertainty and has received special attention in international literature.

The assumption that the soil consists of layers with some average values for the properties of each layer does not reflect the actual conditions. Its properties are not unchanged and the range of variation can significantly affect the stability analysis of the slopes.

It is important to understand and include in the analysis as many uncertainties as possible, since each soil property affects the stability of the slope, as reported by Baecher and Christian (2003), [21] and links uncertainty to the unknown or unconfirmed.

If all the necessary spatial and point statistical parameters are available, probability analysis can be completed using various methods found in the literature.

These methods use a reliability index to calculate the probability of failure.

The reliability index is in fact the margin of safety (Margin of Safety, MS) of the difference in retention forces and destructive forces:

$$MS = FS - 1 \tag{1}$$

Therefore, based on the statistical properties of the safety factor, we can calculate the reliability of the probability of failure. The following methods contributed to this assessment:

1. Point estimation method (PEM)
2. 1st order 2nd Torque Method (FOSM)
3. 1st order reliability method (FORM)
4. Monte Carlo Simulation Method (MCM)

In particular, the significance of spatial correlation (or autocorrelation) and the local average of geotechnical properties have been recognized by many researchers [22], Reference [23] studied slope stability using the Random Finite Element Method (RFEM) that combines finite element elastoplastic analysis with random variables using the Local Average Subdivision Method (LAS) by Fenton & Vanmarcke (1990) [24].

8. Statistical Parameters of the Mechanical Characteristics of the Slope Soil Mass

8.1 Mean value, standard deviation, autocorrelation and cross-correlation

Following, the indicative fluctuation ranges of the mean μ, as well as the coefficient of variation Cov of the effective internal friction angle φ, the active cohesion c, as well as the unit weight γ (specific gravity), soil masses, are presented as found in the literature.

More specifically, the coefficient of variation for the active internal friction angle is recorded to be between 2% and 15%, as shown in the table below.

Table 4: Average mean μ values and coefficient of variation Cov for the active angle of internal friction

Researcher	Year	μ	Cov
Harr	1987[25]		2% - 13%
Kalhawy	1992[26]		2% - 13%
Phoon and his colleagues	1995[27]	20 - 40 (deg)	5% - 15%
Lacasse and his colleagues	1997[28]		2% - 5%
Suchomel	2010[29]	21 (deg)	8%
Phoon and his colleagues	1999[27]	21-40 (deg)	5% - 15%
Duncan	2000[30]		2% - 13%
Jeremic and his colleagues	2007[31]		2% - 5%
Griffiths and his colleagues	2002[23]	35 (deg)	5% - 50%
El Ramley and his colleagues	2003[32]	35 (deg)	5.60%
Schweiger	2005[33]	35 (deg)	0

Table 5: Average values of μ and coefficient of variation Cov for active cohesion.

Researcher	Year	μ	Cov
Griffiths and his colleagues	2002[23]	24KN/m^2	30%
Suchomel	2010[29]	10KN/m^2	21%
Harr	1987[34]		20%
Cherubini	1997[35]		20%-30%
Li and his colleagues	1987[36]		40%

There is not enough data on the variation in unit weight. Smith and his colleagues (1995) [37], Hicks and his colleagues (2002) [38] and Griffiths and his colleagues (2002) [23] considered a deterministic weight unit variable of 20 KN / m^3.

Table 6: Average values μ and coefficient of variation Cov for unit weight.

Researcher	Year	μ(kN/m^3)	Cov
Harr	1987[34]		1%-10%
Phoon and his colleagues	1995[27]	13-20	<10%
Smith and his colleagues	2004[37]	20	0%
Duncan	2000[30]	14-20	<10%
Wang and his colleagues	2010[39]	20	6%
Hicks and his colleagues	2002[38]	20	0%
Griffiths and his colleagues	2002[23]	20	0%
Schweiger	2005[33]	20	0%

Finally, R. Rakwitz (2000) [40] proposes the following standard deviation values:

Table 7: Standard deviation of shear resistance parameters

Strength parameters	**Standard deviation**
Specific weight (kN/m^3)	1
Angle of internal friction	4-8
Cohesion (kPa)	6-15
Shear measure (MPa)	7-28

Vorechovsky (2007) [41] emphasizes that a change in mean value, standard deviation and correlations has an effect on both the autocorrelation coefficient and the correlation coefficient. Of particular interest are the values of the correlation coefficient ρ (cross-correlation) between the shear strength parameters of the soil (cohesion, internal friction angle) and the specific weight γ of the soil material as shown in the following tables:

Table 8: Cross-correlation of c and φ

Researcher	Year	$\rho_{c\varphi}$	Test
Orr and his colleagues	2008[42]	-0,47	
Harr	1987[34]	0,25	CU
Hara and his colleagues	2011[43]	-0,1	CD
" "		-0,81	CD
" "		-0,87	CD
" "		-0,572	
" "		-0,554	
" "		-0,49	
" "		-0,359	
" "		-0,557	
Lumb	1970[44]	-0,7	
" "		-0,37	
Matsuo and Kuroda	1974[45]	-0,412	Direct shear test
		0,316	" "
		0,369	" "
		-0,474	" "
		-0,748	" "
Wolff	1996[46]	-0,47	CD

Table 9: Cross-correlation of c and φ

Researcher	Year	$\rho_{c\gamma}$	Test
Babu and Srirastava	2007[47]	0,25	
" "		0,5	
" "		0,75	
Chowdhurry and Xu	1992[48]	0,4	
Low and Tang	1997[49]	0,5	
Matsuo and Kuroda	1974[45]	0,44	Direct shear test

Table 10: Cross - correlation of φ and γ (Xing Zheng Wu 2013) **[50]**

Researcher	Year	$\rho_{\gamma\varphi}$
Babu and Srirastava	2009[47]	0,25
" "		0,5
" "		0,75
Chowdhurry and Xu	1995[48]	0,7
Low and Tang	1997[49]	0,59
Matsuo and Kuroda	1974[45]	0,713
" "		0,656
" "		0,926
" "		0,859
" "		-0,943

The value of the correlation coefficient between the specific gravity and the internal friction angle according to Rakwitz (2000[40]) is slightly positive between 0 and 0.5, whereas the correlation coefficient between the cohesion and the internal friction angle is always negative at -0.5.

8.2 Data on spatial statistical parameters

Variable range (l) and anisotropy scale (ξ)

The isotropy and anisotropy of a soil mass depend on the ratio of the autocorrelation length in the horizontal direction to the corresponding length in the vertical direction.

If a soil mass is isotropic then the rate of change of one variable in the horizontal and vertical direction is the same and $l_v=l_h$ while if $l_v \neq l_h$ the soil mass has an anisotropic behavior in each direction. The ratio between the autocorrelation length in the horizontal and the vertical axis is calculated by equation (2) where ξ is the measure of anisotropy:

$$\xi = l_h/l_v \qquad (2)$$

The values found in the literature are in all cases given a pair of values that characterize the spatial correlation in the vertical and horizontal axes (Table 11).

This is accomplished by giving the two different values of the variance scale for the axes or one of the two values and the anisotropy scale that expresses the relation of the property values between the vertical and the horizontal plane. The values found in the literature are several and we can say that they are in broad agreement with each other [51].

Table 11: Length of spatial correlation and anisotropy scale

Researcher	Year	ξ	l_v	l_h
Phoon and his colleagues	1995[27]		2-6 (m)	10-60 (m)
Phoon and his colleagues	1999[27]		3-6 (m)	11-60 (m)
Cherubini	1999[35]		4-6 (m)	12-60 (m)
Duncan	2000[37]		2,4-7,9(m)	
Griffiths and his colleagues	2002[23]		6-6 (m)	14-60 (m)
Hicks and his colleagues	2005[38]	8	1 (m)	
Sudret and his colleagues	2002[52]	1	10-30 (m)	
Hicks and his colleagues	2002[38]	>10	0,3-3 (m)	
Suchomel	2010[29]		10-40 (m)	0,5-3 (m)

8.3 Random Fields Simulation

In order to fully understand and then solve complex geotechnical projects with complex problems, the principle of Analysis-Composition was applied. The first step in the methodology is to analyse a phenomenon or series of conjugate phenomena into simpler components, to understand their function and then to search for mathematical simulation methodologies. This attempts to reduce physics problems τo mathematics. The reconnection of the components of the general, on the one hand leads to complexity once more, but the problem is that it is now discrete and mathematically solvable. The individual parts of the problem, after decoding and decoupling coupled phenomena, can be simulated with constitutional behavioral laws (mathematical models) that are appropriate for any particular case. The subdivision of the whole area into elements is called discrimination and is governed by certain rules as well as by optimal approach rules [13]. For the simulation and description of random fields there are several available methods, the main ones being the following:

1. The D.F.T. (Discrete Fourier Transforms)
2. The F.F.T. (Fast Fourier Transform)
3. The Turning Bands Method (TBM)
4. The Random Finite Element Method (RFEM) method with the Local Average Subdivision (LAS) method.

8.4 Comparison of FFT, TBM, LAS and exact solution methods

Table 4.11 presents comparisons of FFT, TMB, LAS methods and exact mathematical solutions. The solutions of the FFT, TMB, and LAS methods are created by a set of 200 random variable fields in a 128 x 128 grid based on a Markov sequence of autocorrelation length $\theta = 2$ and physical dimensions of 5 x 5. First, averaged and dispersed fields are calculated, and then the 5% and 95% thresholds are determined, so the values between the two thresholds account for 90%.

It is noted that the LAS method is the most accurate of all simulation methods. The method of random finite elements with subdivision of the local average has significant advantages over existing approaches. It is a simple method that is ideally matched to finite element models (Fenton and his colleagues 1990).

Table 12: Upper and lower limits (5% and 95%) of the calculated mean values and dispersions for FFT, TBM and LAS based on 200 solutions (Fenton and Griffiths 2008) [24].

Method	Average value	Dispersion
FFT	(-0.06, 0.12)	(0.87, 1.19)
TBM	(-0.11, 0.06)	(0.83, 1.14)
LAS	(-0.12, 0.09)	(0.82, 1.13)
Accurate solution	(-0.12, 0.12)	(0.84, 1.17)

In conclusion, the LAS algorithm has proven to be an accurate and effective method for the local random average that produces (in a homogeneous normal distribution) real elements in one, two or three dimensions. The main advantages of the method are as follows:

1. It employs well-known local averages in a simple manner and manages to approach real conditions.
2. It produces elements that are interdependent (depending on the scale - subdivision stage) and presents the correct covariance between the local averages in any solution.
3. It is ideally suited to finite element models that use low-grade interference functions in which each local average is a separate element.
4. It avoids problems due to the symmetry of covariance at boundaries (such as in the FFT method), solved by other conventional methods.

The most important advantage, however, remains that the above LAS method works in harmony with the finite element and finite difference methods.

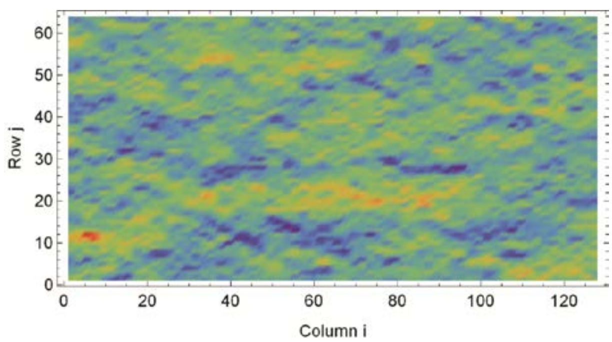

Figure 23: Field creation example with the LAS method (autocorrelation lengths $\theta_x = l_x$ = 20 m and $\theta_y = l_y$ = 2 m)

9. Conclusions

1. The classical methods of the Limit Equilibrium Method [53, 54] and the Finite Element Method [55] can not reliably respond to the slope stability calculation due to the natural heterogeneity of the geomaterials.

2. Slope failure is due to the great inclination of the free surface, the low shear strength of the slopes or an underlying soil layer, the decrease of the shear strength of the soil due to the increase of the water pressure of the pores, the imposition of unfavorable external loads, excavation or erosion at the base of the slopes as well as seismic loads.

3. From the examples of slope and embankment failure, it has been found that it is difficult to predict, identify and categorize such slides and failures. The reason for this lies with the difficulties associated with the determination of reliable parameters of the materials on the slip surface, the exact and inadequate characterization of the behaviour of the materials under irregular circular loads, and the uncertainty associated with estimating the seismic charges which are never clearly known [5]

4. The calculation of the movements requires that the slope stability analysis should include the seismic dynamic stress response.

5. The assumption of a tolerable movement level is surrounded by uncertainties as the behavior of a slope during and after the seismic vibration is associated with the choice of the shear strength parameters of the material and the precise calculation of the seismic load.

6. Vulnerability curves are one of the key elements of stochastic seismic hazard. They associate seismic intensity with the probability of approaching a failure or destruction level for each element of risk.

7. Analysis of slope stability as a branch of the subject of geotechnical engineering is directly related to the use of stochastic methods that lead to quantification of uncertainty and has received special attention in international literature. In order to fully understand and then solve complex geotechnical projects with complex problems, it is necessary to know the statistical parameters of the mechanical characteristics of the soil mass of the slopes.

8. By comparing the simulation and description of random fields, it is noted that the Local Average Subdivision method presents the highest precision of all simulation methods.

9. The L.A.S. algorithm in conjunction with finite difference models can demonstrate the large fluctuations in the coefficient of stability (factor of safety) values and the permanent seismic displacements of the slopes under the effect of seismic charges whose time histories are known. [56]

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