Probability Analysis of Embankments Stability Constructed on Stone Columns under Seismic Load

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Abstract

Geotechnical engineers are usually interested in slope stability problems. To increase the factor of safety (FOS) for slopes, different ways could be used such as retaining walls, stone column, and soil reinforcement. Using stone columns may be considered one of the alternative solutions for slope stabilization. Analysis and design of slope are mainly depend on deterministic method. The FOS is usually chosen after understanding and knowledge soil parameters, problem geometry, method of analysis which causes a differ in FOS in view of designers. The inherent variability of soil characteristic considers that a slope stability problem is a probabilistic problem rather than deterministic problem. The objective of this research is to predict probabilistic approach for uncertainty in the slope stability analysis. This research is carried out on a hypothetical problem which includes a sensitive variability analysis. The computer program commercially named SLOPE/W is used in this research which adopted Monte Carlo method for probability simulation. The output results are presented as a form of a probability of slope failure. These results were found to be a butter index for slope stability compare to FOS because it provide a range of values of FOS rather than one. Also it is found that a reduction in the probability of failure in the order of about (41-100) % can be obtained when using two rows of stone columns in the embankment with two limits of standard deviation for static slope stability analysis, While the effect of seismic load on the probability failure reduction is in the order of about (26-56) % for the same conditions above of static case.

Keywords: stone column, slope stability, probability, reliability index, seismic analysis.

1- Introduction

Soil is a naturally formed material which it have physical properties changing from point to point in other meaning the properties of any soil vary spatially within the deposit, both vertically and horizontally. The total variability in the evaluation of values of the soil properties is a major effect for uncertainty of the slope stability. Experimental results on natural soils reveal that the random variations in soil material are based on the normal distribution function. This approach provided a rational basis for making decision when choosing design parameter values. Thereby it also became possible to determine the probability that the value was less or more than the value meaning it is possible to determine risk. (Lumb [1] and Tan et al. [2]).

The traditional slope stability analyses evaluate the FOS based on constant conditions and material parameters. The slope is considered to be stable when the FOS is more than one. While, if the FOS is less than one, the slope is considered to be unstable. Deterministic analyses had limitations for calculating slope stability like the variability of the input parameters.

Generally, a FOS is an index for the slopes stability. As the input parameters are vary, the FOS does not give the real risk level of the slope because the uncertainties in analysis parameters are not taken into account during the calculation. While, probabilistic analysis have two major indices (the probability of failure and the reliability index) considered for estimating the stability and uncertainties in soil properties. Thus a probabilistic analysis is superior to a deterministic analysis.

2- Methods For Seismic Slope Stability Analyses

Surveys on behavior of embankments during seismic load found that embankments constructed from materials (good compacted clayey, unsaturated sand, some dense saturated sands, gravels and silts) are not vulnerable to degradation in strength due to earthquake shaking. These materials are generally having good performance during earthquakes (Seed et. al. [3]). However, the embankment may have some amount of permanent deformation due to earthquake excitation. With efficient-constructed of earth dams, the value of permanent seismic deformations should be small. Otherwise, even stable earth dams that subjected to major earthquakes may have large deformations that could hazarded the structures safety. For evaluating the seismic instability and seismically induced permanent deformations, simplified methods were developed for this purpose (Seed [4] and Seed et. al. [3]). For the development of the seismic stability of natural slopes in clayey materials using various modifications of the following two methods (Duncan and Wright [5]):

1. Pseudo-static method.
2. Sliding block method.

2-1- Pseudo Static Analyses

The pseudo static method is one of the oldest methods for analysis of seismic stability. This method is assumed that the seismic loading could be simulated by equivalent horizontal static force that obtained by multiplying the self-weight (the weight of potential failure mass) with seismic coefficient, k_h. The FOS for seismic slope stability analysis is computed according conventional limit equilibrium by integrate the pseudo static force in the limit equilibrium. The seismic coefficient (k) could be expressed as a fraction of the acceleration gravity, g, however, the pseudo static force having one directional action, while the seismic acceleration could acts in different directions with very short period, in other meaning the horizontal pseudo-static force has a larger effect on the FOS than the vertical pseudo-static force

This method considers the vertical component of the earthquake accelerations is negligible. While, the horizontal component is considers and taken as horizontal force. The application of this technique in the limit equilibrium of slope stability analysis is relatively clear from the view of the applied mechanics theories. The pseudo static force in an infinite slope is assumed to be a known force that included in equilibrium equations by expressing shear strength in terms of total stresses as shown in Fig. (1).

Resolving forces perpendicular to slip plane:

$$N = W\cos\beta - kW\sin\beta$$

Resolving force parallel to slip plane:

$$T = W\sin\beta + kW\cos\beta$$

Weight of sliding block:

$$W = \gamma \ell z\cos\beta$$

Substituting (3) into (1) and (2):

$$N = \gamma \ell z\cos^2\beta - k\gamma \ell z\cos\beta\sin\beta$$

$$T = \gamma \ell z\cos\beta\sin\beta + k\gamma \ell z\cos^2\beta$$

For the stresses on the slip plane:

$$\sigma = \frac{N}{\ell} = \gamma z\cos^2\beta - k\gamma z\cos\beta\sin\beta$$

$$\tau = \gamma z\cos\beta\sin\beta + k\gamma z\cos^2\beta$$

Finally, for the factor of safety (total stresses):

$$F = \frac{s}{\tau} = \frac{c+\sigma\tan\phi}{\tau} = \frac{c+\left(\gamma z\cos^2\beta - k\gamma z\cos\beta\sin\beta\right)\tan\phi}{\gamma z\cos\beta\sin\beta + k\gamma z\cos^2\beta}$$

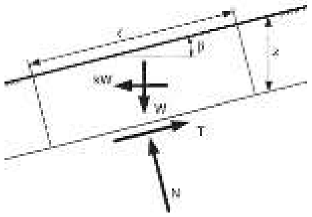

Fig. 1. Derivation equation for (FOS) of an infinite slope with a seismic force (kW)—total stress analyses, after (Duncan and Wright [5]).

2-2- Sliding Block Analyses

This method is based on the concept which suggest by Newmark [6]. In this method the analysis of soil body under earthquake loading modeled as a relatively simple plastic rigid frictional block resisting on an inclined plane to calculates the cumulative permanent displacements of a sliding mass as shown in Fig. (2). In the Newmark displacement method the relation between acceleration and time history is applied as follow, when accelerations higher than the critical acceleration, a_y cause the block to move, which is the minimum acceleration required for stability. The movement of block continues after the acceleration take places below the yield acceleration also, when velocity between block and soil under slip surface became zero, the movement continue, after that the block continue to move when the acceleration exceed the yield acceleration as shown in Fig. (3). In otherwise if the acceleration does not exceed the yield acceleration, there is no computed sliding block displacement. To evaluate the displacements, the accelerations that exceed the yield acceleration are integrated for computing the velocities. Integration is adopted to compute the displacements as shown in Fig. (3).

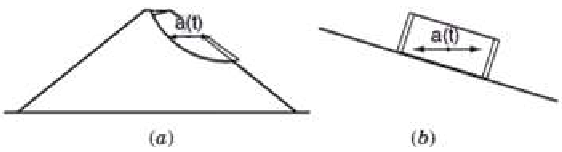

Fig. 2 (a) Actual slope; (b) sliding block representation used to compute permanent soil displacements in a slope subjected to earthquake shaking, after (Duncan and Wright [5]).

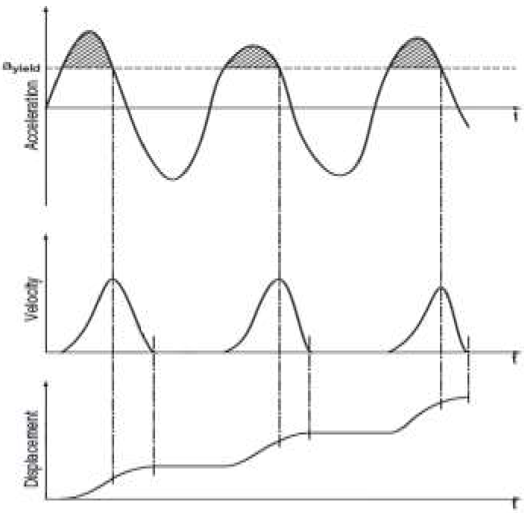

Fig. (3) Double integration of acceleration–time history to compute permanent displacements, after (Duncan and Wright [5]).

3- Embankments Stabilized With Stone Columns

A number of factors and parameters such as soil properties, pore water pressure resume, slope geometry, earthquake, and vibration can influence the slope stability. Engineering slope stabilization is generally referred to stop or decrease the possible of instability process of slopes. Preventing the movement of a slope or increasing the safety factor (SF) is possible by using structural or geotechnical methods. Stone columns are method for slope stabilization. This method was used since 1950 normally for cohesive soil improvement. It is a hole with circular section which is filled by gravel, rubble and etc and is an effective method to increase the shear strength on the slip surface of clayey slopes. The most important for using stone columns (Barksdale and Bachus [7]) are:

1. Improve embankment stability constructed on soft ground.
2. Increase the carrying capacity of shallow foundations on soft cohesive soil.
3. Accelerate the consolidation rate of the soft cohesive soil and decrease the total and differential settlements.
4. Mitigation of hazards induced earthquake liquefaction of sandy soils.

4- Reliability and Probability of Failure

The probability analysis can give answer to the probability that a failure in a slope will occur also the parameters in the input data how much the total uncertainties are affected by the each parameter.

The probability of failure could be computed in two ways (Mostyn and Li [8]):

The first way could be contributed in projects to find the percentage of slops that would fail when the same slope is regenerated many times. While the second way is more contributed in projects where a given design is modeled one time only and it either fails or not. Nevertheless, the probability of failure is a good index revealing the risk level of the stability of slope.

There is no straightforward relevant between the FOS and the probability of failure. In other words, a slope with a lower FOS than a slope with a higher FOS may be stable (Harr [9]). For example, a slope having FOS and standard deviation of 1.25 and 0.5 respectively may have a higher probability of failure than a slope with FOS and standard deviation of 1.0 and 0.1 respectively.

In this sense the FOS is not a sufficient indicator of safety because the uncertainties in material can significantly influence the probability of failure.

The reliability of a slope (R) is defined as uncertainty of stability analyses and is given by Eq. (1) (Duncan and Wright [5]):

$$R = 1 - P_f \qquad (1)$$

P_f: is the probability of failure and **R** is the reliability or probability of no failure.

The reliability index (β) gives a more realizable value of stability than the FOS and given by Eq. (2) (Christian et al. [10]).

$$\beta = \frac{\mu - 1}{\sigma} \qquad (2)$$

5- Statistical Analysis

5-1- Probability Density Function

A normal distribution function or it called the Gaussian distribution function is most common function that is used to represent soil parameters, such as the friction angle and the cohesion, as random variables in probabilistic analyses. The normal distribution is closely approximate to a normal curve because many measurements give frequency distribution. A normal distribution function can be given by Eq. (3):

$$f(x)=\frac{1}{\sqrt{2\pi\sigma}}e^{-\frac{(\chi-\mu)^2}{2\sigma^2}} \quad -\infty\langle\chi\rangle\infty$$

(3)

Where:

$f(x)$ = relative frequency

σ = standard deviation

μ = mean value

A normal distribution curve is like a bell shape. The properties of the normal distribution that it is symmetric about the mean value, μ, therefore the median is equal to the mean. This curve represents the relation between the mean value, μ and the standard deviation,σ. It can be seen that, maintaining the mean value μ constant, the standard deviation σ governs the spread of the curves. A probability density function (PDF) shown in Fig. (4) describes the relative likelihood that a random variable will assume a particular value. In this case the random variable is continuously distributed.

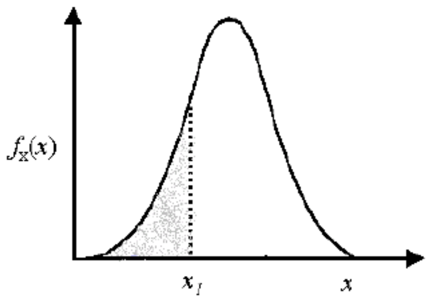

Fig. 4. Probability density function

5-2- Random Number Generation

The random number is generated from the uniformly distributed function having values ranging between 0 and 1.0. Transformation of the uniform random to a normally distributed number should be considered for the evaluation of the normally distributed input parameters in order to use the above uniformly generated random number. This "normalization" process is given by Eq. (4) which it is suggested by SLOPE/W manual [11]:

$$N=\sqrt{(-2\ln R_1)}*(2\pi R_2) \qquad (4)$$

Where:

N = normalized random number

$\mathbf{R_1}$ = uniform random number 1

$\mathbf{R_2}$ = uniform random number 2

To transform the equation it is required to generate two uniform random numbers. The normalized random number can be obtained when the mean value and the standard deviation are 0 and 1 respectively.

5-3- Correlation Coefficient

A correlation coefficient is an expression which represents a relation between two parameters. Lumb, in 1970 was conducted laboratory tests on different types of soil. He concluded that the correlation coefficient ranges between -0.72 and 0.35 are often negatively correlated for the shear strength parameters c and ϕ. The probability distribution of slops may be affected by correlation between strength parameters. SLOPE/W program allows correlating the specification of coefficients c and ϕ correlation coefficients for all soil models.

Correlation coefficients always are ranging between -1 and 1. When the correlation coefficient is positive, c and ϕ are positively correlated using the larger value of the correlation coefficient. While, for negative correlation coefficient, c and ϕ are negatively correlated using the larger value of the correlation coefficient for c and the smallest one for ϕ . When c and ϕ are independent parameters, no correlation coefficient will occur.

In SLOPE/W, when estimating new trial values of ϕ and ϕ2, the normalized random number is adjusted to consider the effect of correlation. Eq. (5) is used as follows:

$$N_A = N_1 k + (1 - |k|) N_2 \qquad (5)$$

Where:

k = correlation coefficient between the first and second parameters

$\mathbf{N_1}$ = normalized random number for the first parameter

$\mathbf{N_2}$ = normalized random number for the second parameter

$\mathbf{N_A}$ = adjusted normalized random number for the second parameter.

6- Method of Probabilistic Analysis

6-1- Monte Carlo method

The Monte Carlo method is a simple computational method. In general, this method has properties listed below (Yang et al. [12] :

- Having the same solution procedure, adopted for the finite element stress method or Spencer's method.
- The input parameters are to be modeled probabilistically and the variability results in terms of a normal distribution model are depend on the mean value and the standard deviation.
- The FOS will be calculated many times depending on new input parameters.

6-2- Number of Monte Carlo Trials

Many trial runs would be adopt in the analysis of Probabilistic slope stability by the Monte Carlo method. Theoretically, the more trial runs lead to get accurate results. Harr [9] drive an expression for estimating the number of the required Monte Carlo trials in the probabilistic analysis as given by Eq. (6). This equation is depending on the desired level of confidence in the analysis and the number of variables.

$$N_{mc} = \left[\frac{(d^2)}{(4(1-\varepsilon)^2)} \right]^m \qquad (6)$$

where :

N_{mc} = number of Monte Carlo trials,

ε = the desired level of confidence (0 to 100%) expressed in decimal form,

d = the normal standard deviate corresponding to the level of confidence, and

m = number of variables.

6-3- Measure of Random Variables

The trial factors of safety are assumed to be normally distributed in SLOPE/W program. So that, statistical analysis can be carried out to determine the PDF, standard deviation, mean, and the probability distribution function of problem of the stability of slopes. Eq. (7) and (8) is used in this statistical analysis (Lapin [13]):

Mean factor of safety, μ:

$$\mu = \left(\frac{\sum_{i=0}^{n} F_i}{n} \right) \quad (7)$$

Standard deviation, σ:

$$\sigma = \sqrt{\left(\frac{\sum_{i=0}^{n} (F_i - \mu)^2}{n} \right)} \quad (8)$$

7- Parametric Study

The parametric study contains the analysis of embankment constructed on soft clays. The material of the embankment body is the same as that of its foundation but strengthened with stone columns. In this section, a one row or two rows (at distance 1.7m from first row) of stone columns are used to reinforce the slope and parametric study has been performed to determine the effect of uncertainties in the geotechnical properties of the slope soil materials and stone column material on the slope stability. The embankment to be analyzed is shown in Fig. (5). The height of embankment is 10m with 30^0 side slopes and 10m crest width.

The geotechnical properties of the clayey soil and stone column are shown in Tables (1) and (2).

Typically, the strength parameters (C and ϕ) and the unit weight could be treated as variables. Table (3) shows a summary of typical reported values of coefficient parameters.

In this section, a study is to be carried out on embankment constructed using different conditions (with and without stone columns). Reliability is studied and different states of standard division are discussed.

7-1 Case (1)

Four soil parameters are considered as variables, the strength of the embankment and its foundation, angle of internal friction of the stone column and saturated unit weight of the soil and stone column as shown in Table (4) by making use of the data of Table (3)

The results obtained from analysis of case (1) where the standard deviation with lower limit are shown in Tables (5) and (6) for static and seismic conditions, respectively. In general, the mean FOS increases as compared to the FOS obtained from state without using stone columns analysis. The probability of failure decreases or the reliability index increases when the stone column of one or two rows is used.

The density function and cumulative distribution function of the FOS for this case as obtained by the program Slope/W are shown in Fig.s (6) to (17) for static and seismic analysis respectively.

7-2- Case (2)

In this case the soil is analyzed with a maximum limit of standard division for the strength, angle of internal friction and unit weight of soil as shown in Table (7)

Tables (8) and (9) show the result of analysis where the standard deviation is calculated with upper limit for static and seismic analysis. The effect of increasing the standard deviation on the PDF and cumulative distribution function of FOS are demonstrated in Fig.s (18) to (29).The reliability index obtained for this case is much less than the reliability index obtained from case (1).

The density function and cumulative distribution function of the FOS for this case as obtained by the program Slope/W are shown in Fig.s (18) to (29) for static and seismic analysis, respectively.

Form static slope stability analysis, it can be noticed from the results based on lower limit and upper limit of standard deviation that the use of one row of stone columns increases the reliability index by about (93) % and (58) %, respectively. An increase in the reliability index to about (94) % and (61) % is obtained when using two rows of stone columns, while when adopting seismic load in slope stability analysis, the increase in reliability index is about (90) % and (83) for one raw of stone column and increase in the reliability index is about (94) % and (91) % for two rows of stone columns. This means that the best improvement in stability is obtained when using one row, then limited benefit is obtained when increasing the number of rows.

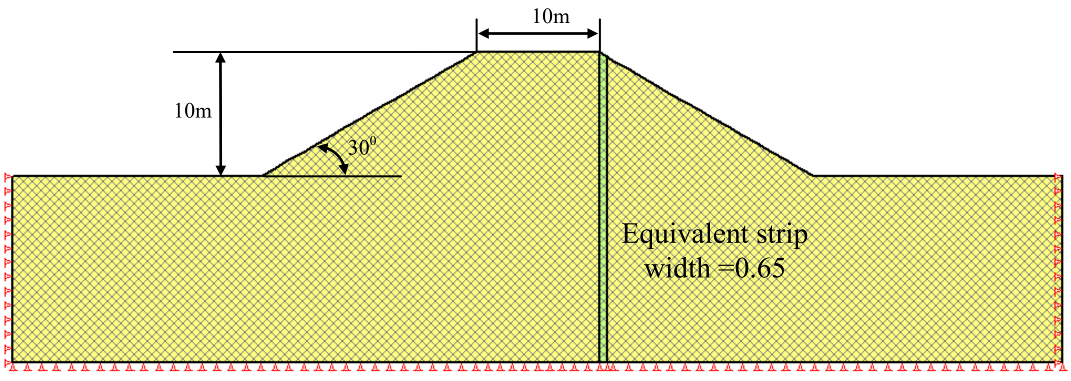

Fig. 5. Geometrical specification of slope with stone column (after Ghazavi and Shahmandi [14]).

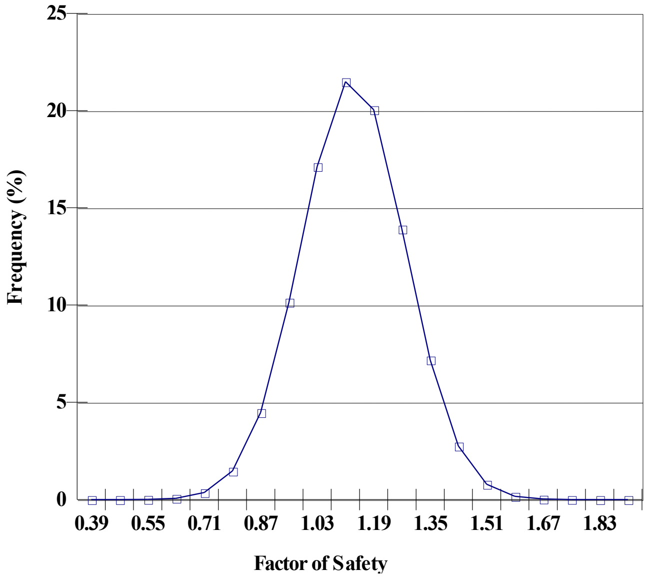

Fig. 6. Probability density function without stone columns for static analysis

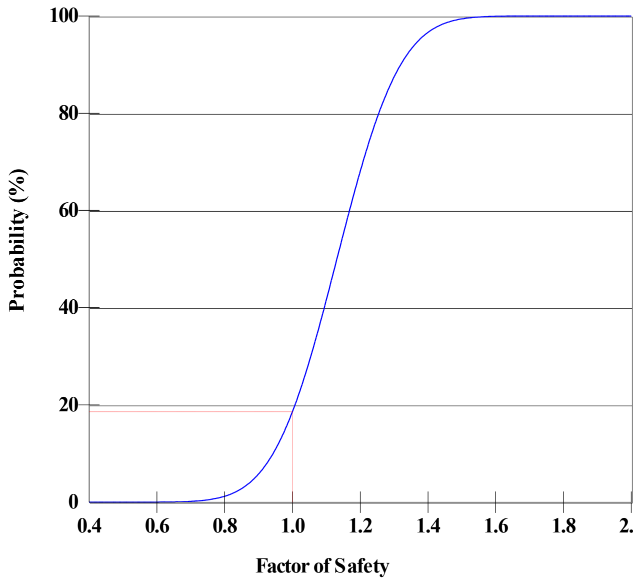

Fig.7. Probability distribution function without stone columns for static analysis

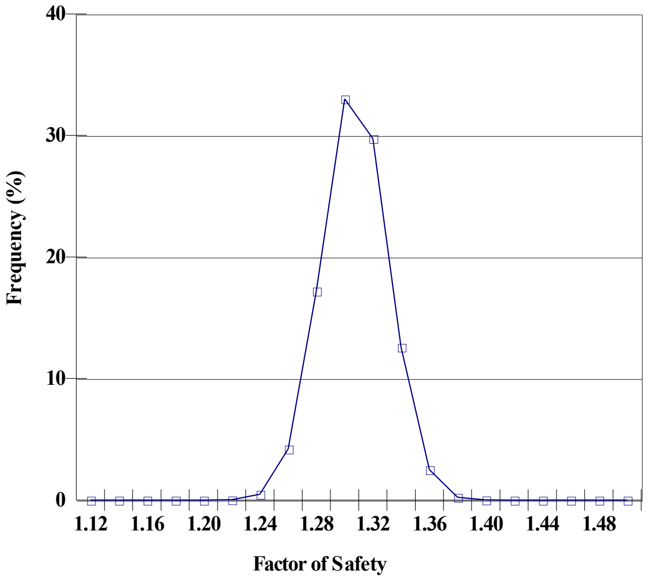

Fig. 8. Probability density function with one stone column for static analysis

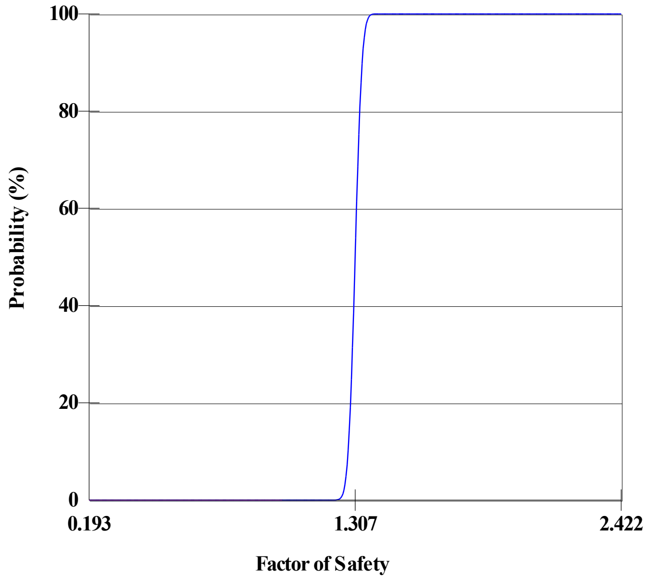

Fig. 9. Probability distribution function with one stone column for static analysis

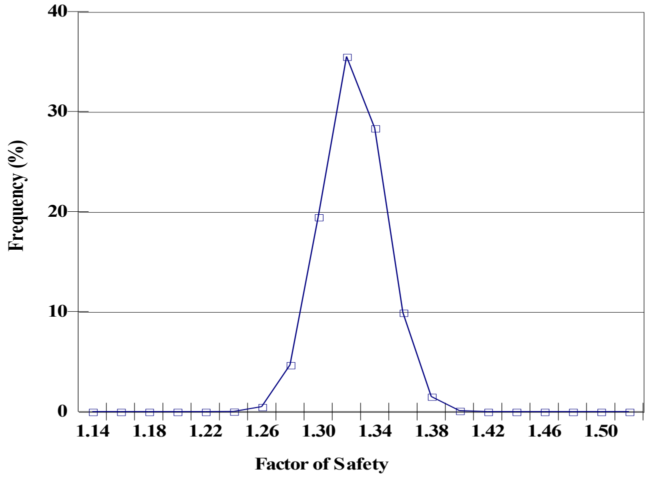

Fig. 10. Probability density function
with two stone columns for static analysis

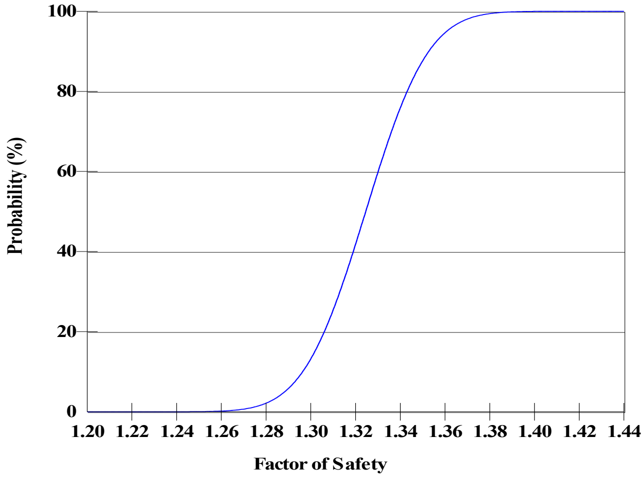

Fig. 11. Probability distribution function
with two stone columns for static analysis

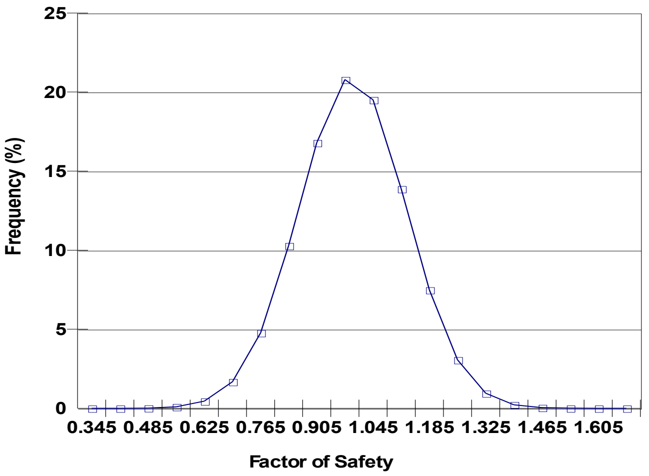

Fig.12. Probability density function
without stone columns for seismic analysis

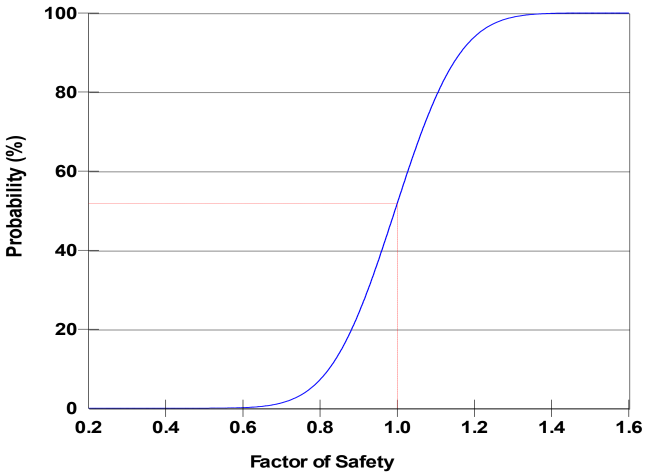

Fig.13. Probability distribution function
without stone columns for seismic analysis

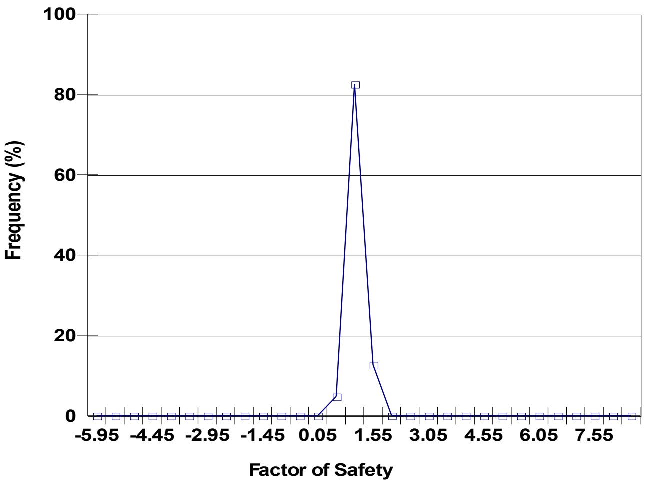

Fig.14. Probability density function
with one stone column for seismic analysis

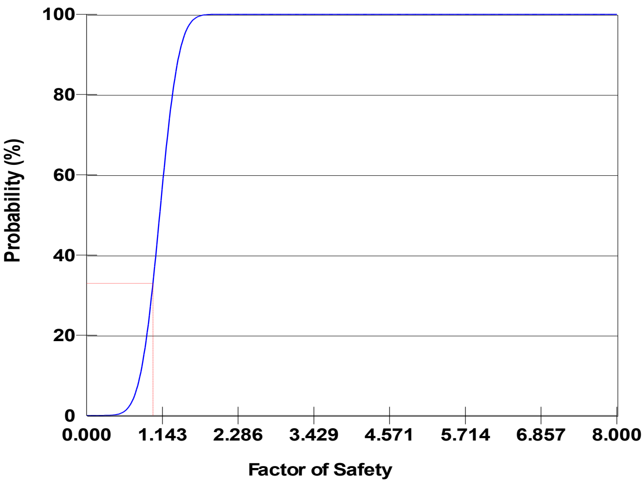

Fig. 15. Probability distribution function
with one stone column for static analysis

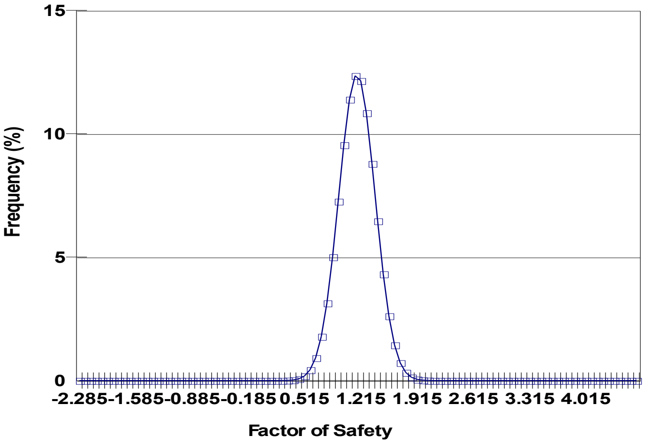

Fig. 16. Probability density function
with two stone columns for seismic analysis

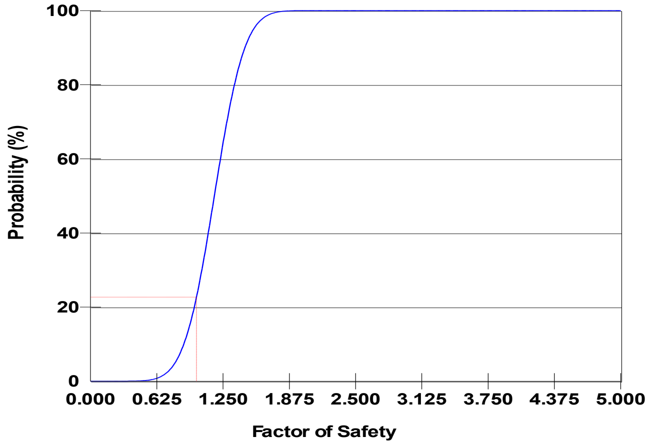

Fig. 17. Probability distribution function
with two stone columns for seismic analysis

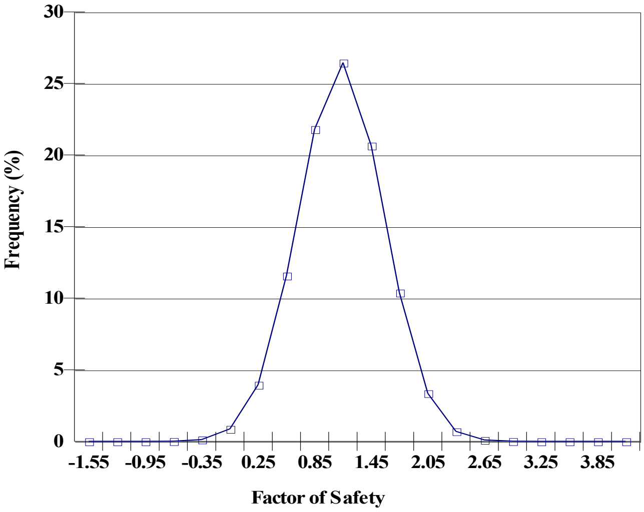

Fig. 18. Probability density function
without stone columns for static analysis

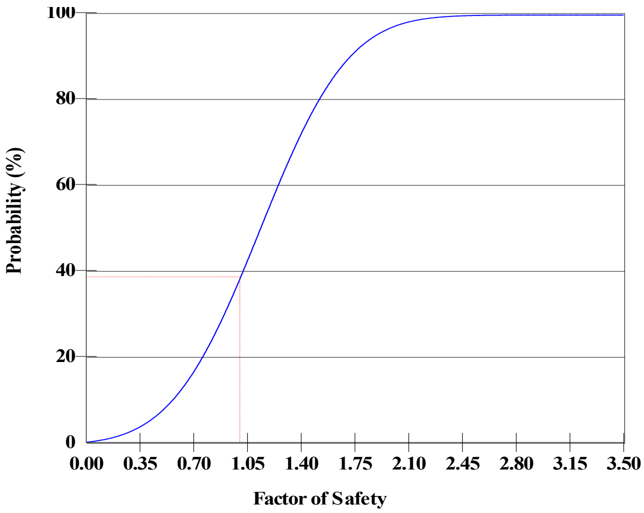

Fig. 19. Probability distribution function
without stone columns for static analysis

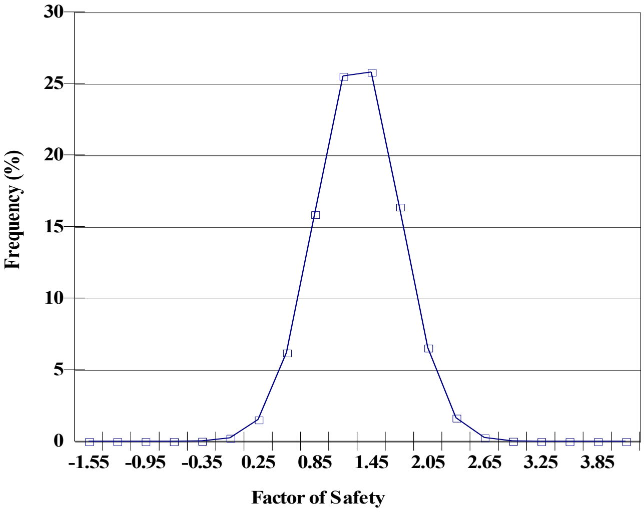

Fig. 20. Probability density function
with one stone column for static analysis

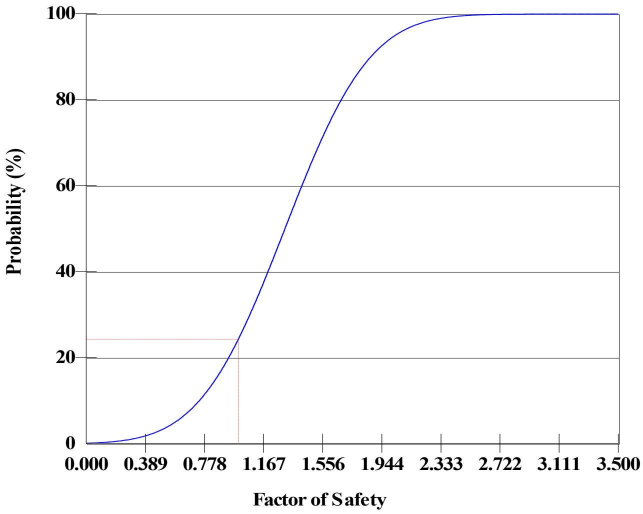

Fig. 21. Probability distribution function
with one stone column for static analysis

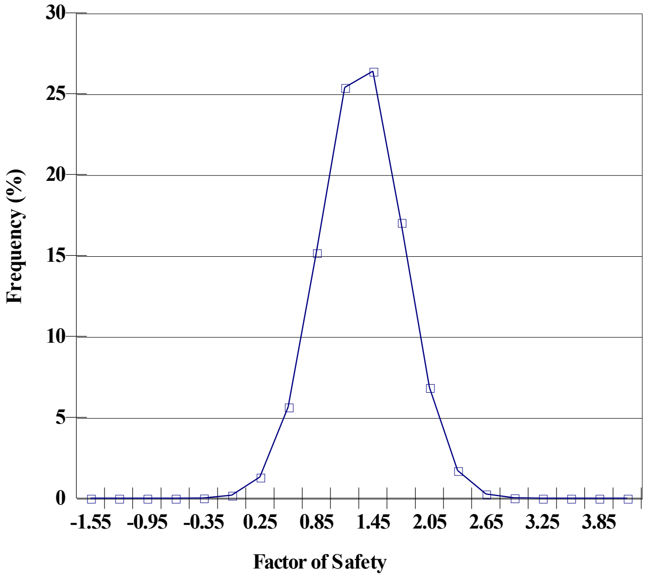

Fig. 22. Probability density function with two stone columns for static analysis

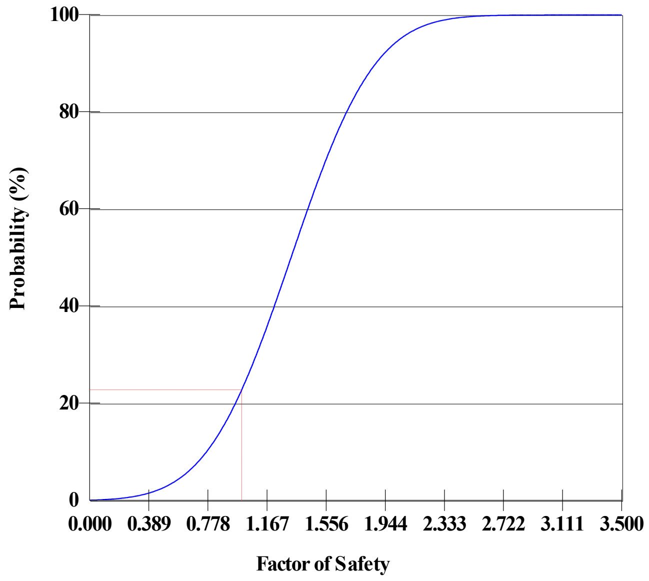

Fig. 23. Probability distribution function with two stone columns for static analysis

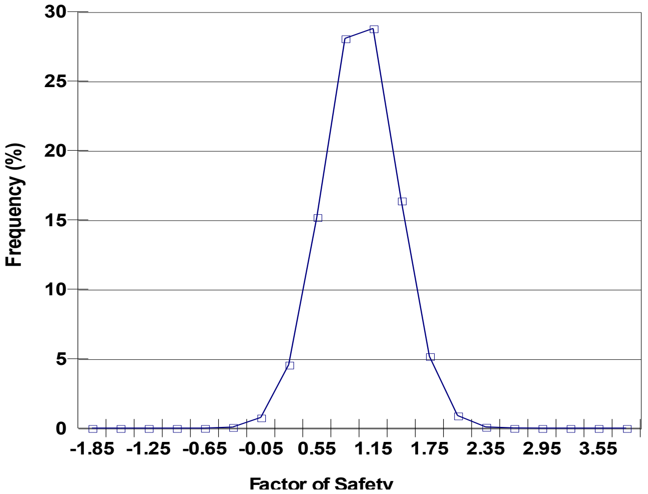

Fig. 24. Probability density function without stone columns for seismic analysis

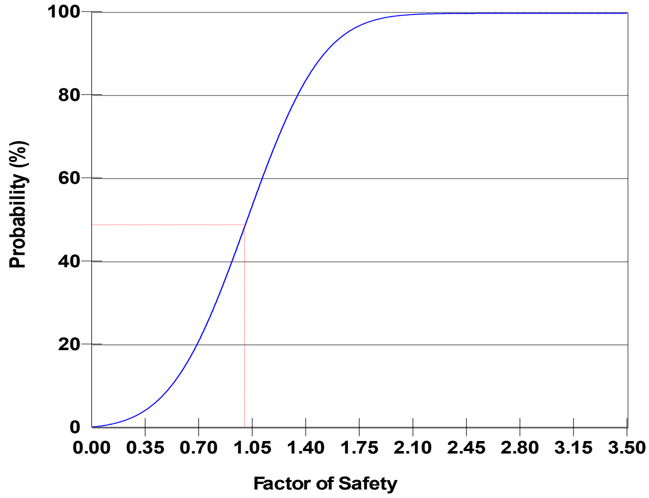

Fig. 25. Probability distribution function without stone columns for seismic analysis

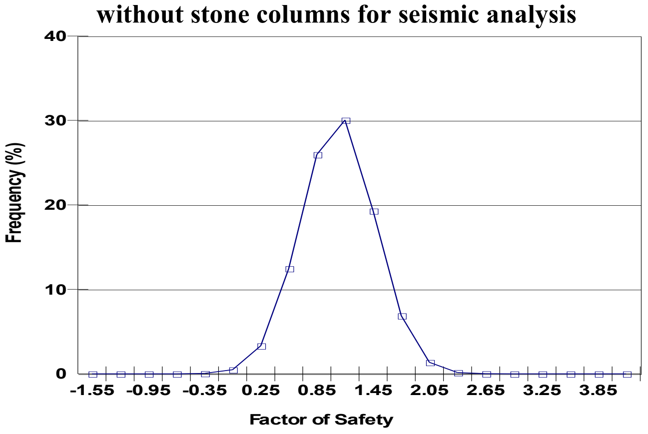

Fig. 26. Probability density function with one stone column for seismic analysis

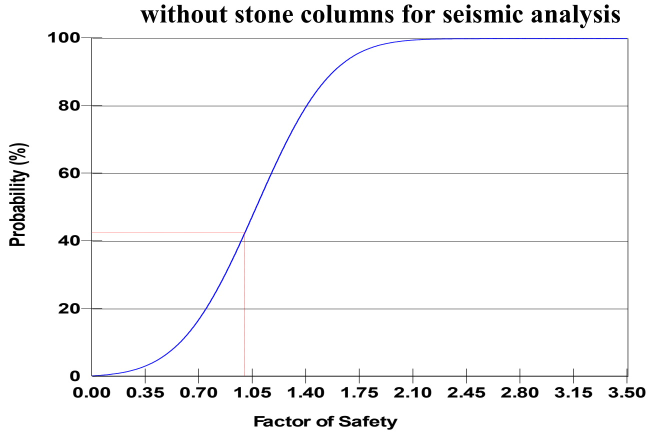

Fig. 27. Probability distribution function with one stone column for seismic analysis

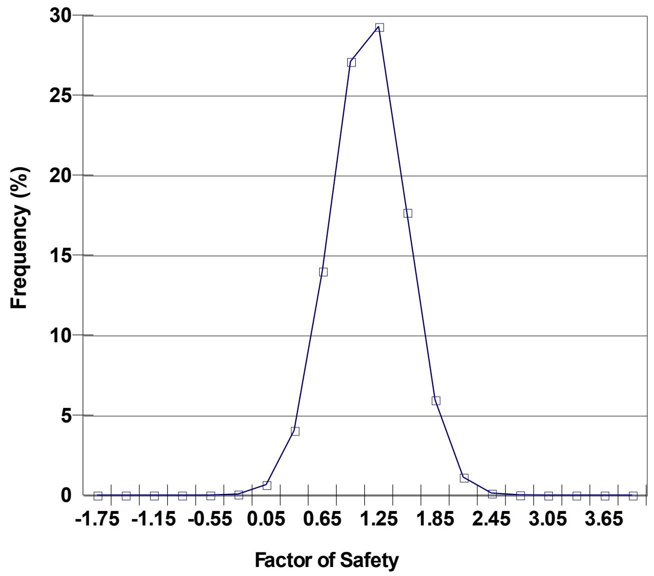

Fig .28. Probability density function with two stone columns for seismic analysis

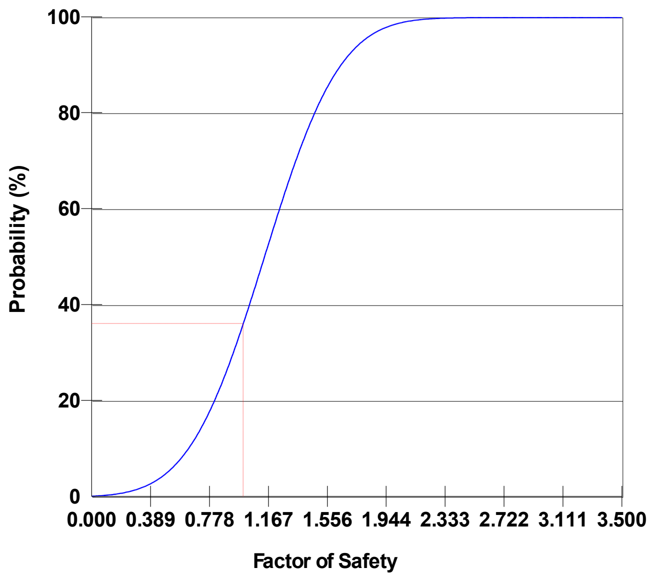

Fig. 29. Probability distribution function with two stone columns for seismic analysis

Table 1. Geotechnical properties of clayey soil. after Ghazavi and Shahmandi [14])

Modulus of elasticity [kN/m²]	Poisson's ratio	Undrained cohesion [kN/m²]	Friction angle [degree]	Saturated unit weight [kN/m³]
5000	0.48	25	0	17

Table 2. Geotechnical and geometrical properties of stone column materials after Ghazavi and Shahmandi [14].

Modulus of elasticity [kN/m²]	Poisson's ratio	Undrained cohesion [kN/m²]	Friction angle [degree]	Saturated unit weight [kN/m³]	equivalent strip width [m]
50000	0.3	0	45	22	0.65

Table 3. Values of coefficient of Variation for geotechnical properties and in situ tests after Duncan and Honorary [15].

Property or in situ test result	Coefficient of variation (%)	Source
Unit weight (γ)	3-7%	Harr (1984), Kulhawy (1992)
Buoyant unit weight (γ_b)	0-10%	Lacasse and Nadim (1997), Duncan (2000)
Effective stress friction angle (ϕ')	2-13%	Harr (1984), Kulhawy (1992)
Undrained shear strength (*Su*)	13-40%	Harr (1984), Kulhawy (1992), Lacasse and Nadim (1997), Duncan (2000)
Un drained strength ratio *($Su/\sigma'v$)*	5-15%	Lacasse and Nadim (1997), Duncan (2000)

Compression index (*Cc*)	10-37%	Harr (1984), Kulhawy, (1992), Duncan (2000)
Preconsolidation pressure (*Pp*)	10-35%	Harr (1984), Lacasse and Nadim (1997), Duncan (2000)
Coefficient of permeability saturated clay (k)	68-90%	Harr (1984), Duncan(2000)
Coefficient of permeability of partly saturated clay (k)	130-240%	Harr (1984), Benson et al. (1999)
Coefficient of consolidation (C_v)	33-68%	Duncan (2000)
Standard penetration test blow count (N)	15-45%	Harr (1984), Kulhawy (1992)
Electric cone penetration test (q_c)	5-15%	Kulhawy (1992)
Mechanical cone penetration test (q_c)	15-37%	Harr (1984), Kulhawy (1992)
Dilatometer test tip resistance (q_{DTM})	5-15%	Kulhawy (1992)
Vane shear test undrained strength (*Sv*)	10-20%	Kulhawy (1992)

Table 4. Soil properties used for cases with different standard deviation

Parameter	Mean	Coefficient of variation (lower limit)/ standard deviation
Cohesion, c (kN/m^3) (soil)	25	13/3.25
Angle of Friction, ϕ (stone column)	45	2/0.9
Unit Weight, γ (kN/m^3) (soil)	17	3/0.51
Unit Weight, γ (kN/m^3) (stone column)	22	3/0.66
Horizontal and vertical seismic acceleration	0.05	-----

Table 5. Analysis results of probability for case (1) for static condition.

parameters	values		
	Without stone column	With one raw of stone column	With two row of stone column
FoS(FEM)	1.131	1.307	1.325
Mean F of S	1.131	1.307	1.325
Reliability Index	0.891	13.561	14.433
P (Failure) (%)	18.59	0.00	0.00
Standard Dev.	0.147	0.023	0.022
Min F of S	0.43149	1.2138	1.2217
Max F of S	1.7955	1.4217	1.4333

Table 6. Analysis results of probability for case (1) at seismic condition.

parameters	values		
	Without stone column	With one raw of stone column	With two row of stone column
FOS(Bishop method)	0.993	1.062	1.133
Mean F of S	0.99396	1.1016	1.168
Reliability Index	0.046	0.441	0.748

P (Failure) (%)	51.819581	32.92	22.68
Standard Dev.	0.133	0.23	0.225
Min F of S	0.44148	0.57608	0.60839
Max F of S	1.5991	7.3574	4.3708

Table 7. Soil properties used for cases with different standard deviations.

Parameter	Mean	Coefficient of variation (upper limit)/ standard deviation
Cohesion, c (kN/m^3) (soil)	25	40/10
Angle of Friction, φ (stone column)	45	13/5.85
Unit Weight, γ (kN/m^3) (soil)	17	7/1.19
Unit Weight, γ (kN/m^3) (stone column)	22	7/1.54
Horizontal and vertical seismic acceleration	0.05	-----

Table 8. Analysis results of probability for case (2) at static condition

parameters	values		
	Without stone column	With one row of stone column	With two row of stone column
FoS(FEM)	1.131	1.307	1.325
Mean F of S	1.1316	1.307	1.3244
Reliability Index	0.291	0.697	0.746
P (Failure) (%)	38.535780	24.232920	22.752750
Standard Dev.	0.452	0.44	0.435
Min F of S	-1.0388	-0.66468	-0.73564
Max F of S	3.3028	3.3895	3.3771

Table9. Analysis results of probability for case (2) at seismic condition

parameters	values		
	Without stone column	With one row of stone column	With two row of stone column
FoS(Bishop method)	0.993	1.062	1.133
Mean F of S	1.0128	1.0743	1.14
Reliability Index	0.033	0.19	0.357
P (Failure) (%)	48.68	42.44	36.04
Standard Dev.	0.391	0.391	0.393
Min F of S	0.10973	0.10584	0.10947
Max F of S	3.0975	3.2048	3.2034

7- Conclusions

1. A reduction in the probability of failure in the order of about (41-100) % can be obtained when using two rows of stone columns in the embankment with two limits of standard deviation for static slope stability analysis.
2. The effect of seismic load on the probability failure reduction is in the order of about (26-56) % when using two rows of stone columns in the embankment with upper and lower limits of standard deviation.
3. The safety factor values and reliability index of stone column reinforced slopes are influenced by various parameters including geotechnical properties of the stone column material and number of rows.
4. The results obtained from seismic analysis of cases 1 and 2 show that the mean FOS increases as compared to the minimum FOS obtained from deterministic analysis.
5. The mean safety factor does not change much when standard deviations are varied in the static slope stability analysis However, the probability of failure increase gradually when the standard deviation of the soil parameters increases.
6. The probability of failure and reliability index can give a complement to the calculation so that the result can be interpreted more but they shall not be used as a design value.

References

1. Lumb, P., 1966. "**The Variability of Natural Soils**", Canadian Geotechnical Journal, Vol. 3, No. 2, pp. 74-97.
2. Tan, C.P. Donald, I.B. and Melchers, R.E., 1993." **Probabilistic Slope Stability Analysis**" - State-of-Play, Proceedings of the Conference on Probabilistic Methods in Geotechnical Engineering, Canberra, Australia. pp. 89-110.
3. Seed, H. B., Makdisi, F. I., and DeAlba, P., 1978." **Performance of earth dams during earthquakes**". Geotechnical Journal., ASCE. vol. 104No.7 pp967–994.
4. Seed, H. B. 1979. "**Considerations in the earthquake-resistant design of earth and rockfill dams.**" Geotechnique. vol. 29, No.3, pp.215–263.
5. Duncan, J., M., Wright, S., G., 2005" **Soil Strength and Slope Stability**" John Wiley & Sons, Inc
6. Newmark, N. M., 1965," **Effects of earthquakes on dams and embankments**", Geotechnique, Vol. 15, No. 2, pp. 139–160.
7. Barksdale R.D., Bachus R.C., 1983. "**Design and Construction of Stone Columns**", Federal Highway Administration Office of Engineering and Highway Operations, Volume I and II, Washington, DC.
8. Mostyn, G.R. and Li, K.S., 1993. "**Probabilistic Slope Stability Analysis**" - State-of-Play, Proceedings of the Conference on Probabilistic Methods in Geotechnical Engineering, Canberra, Australia. pp. 281-290.

9. Harr, M.E., 1987. "**Reliability-Based Design in Civil Engineering**". McGraw-Hill Book Company.

10. Christian, J.T., Ladd, C.C. and Baecher, G.B., 1994. "**Reliability Applied to Slope Stability Analysis**" Journal of Geotechnical Engineering, Vol. 120, No. 12. Pp. 2180-2207.

11. Slope/W manual. www.geoslope.com

12. Yang, D., Fredlund, D.G. and Stolte, W.J., 1993. "A Probabilistic Slope Stability Analysis Using Deterministic Computer Software" Proceedings of the Conference on Probabilistic Methods in Geotechnical Engineering, Canberra, Australia. pp. 267-274.

13. Lapin, L.L., 1983. "**Probability and Statistics for Modern Engineering**." PWS Publishers.

14. Ghazavi M. and Shahmandi A., 2008." **Analytical Static Stability Analysis of Slopes Reinforced by Stone Columns** The 12th International Conference of International Association for Computer Methods and Advances in Geomechanics (IACMAG Goa, India . pp. 2530-2537.

15. Duncan, M., and Honorary, 2000. "**Factors of Safety and Reliability in Geotechnical Engineering**" Journal of Geotechnical and Geoenvironmental Engineering, Vol. 126, No. 4, pp. 307-316

الاحتمالية في تحليل الاستقرارية للسداد الترابية المشيدة على الاعمدة الحجرية تحت تاثير الهزات الارضية

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الخلاصة

ان مهندسي الجيوتكتيك غالبا ما يهتمون في دراسة مشاكل تثبيت المنحدرات ولغرض زيادة معامل الامان هنالك عدة طرق يكمن استخدامها منها الجدران الساندة اوالاعمدة الحجرية اوالتربة المسلحة. ان استخدام الاعمدة الحجرية قد تعتبر احد الطرق البديلة المحتملة لتثبيت المنحدرات .

ان معظم طرق التحليل و التصميم مبنية على اساس طرق حسابية تقريبية .ان قيمة معامل الامان غالبا ما تختار بعد فهم و معرفة متغيرات خواص المادة و الشكل الهندسي للمنحدر و طريقة التحليل و الذي يؤدي الى حصول نتائج مختلفة لمعامل الامان باختلاف المصممين ولذلك فان هذا التباين الموروث في الخواص يملي علينا اعتبار مسالة استقرارية المنحدرات هي مسالة احتمالية اكثر من كونها مسالة حسابية فقط.

ان الهدف من هذا البحث هو ايجاد تقريب احتمالي يتضمن الشكوك في تحليل السداد الترابية المسلحة الحجرية حيث ان الدراسة اجريت على مسالة افتراضية تتضمن الحساسية في تباين خواص المواد في التحليل. ان الدراسة مبنية على نموذج Monte Carlo **الموجود ضمنيا في البرنامج** SLOPE/W **.وقد وجد في هذه الدراسة ان احتمالية الفشل احسن مقياس لاستقرارية المنحدر اذا ماقورنت مع معامل الامان بسب انها توفر مجموعة من قيم معامل الامان بدلا من حصول على قيمة واحدة. ووجد من النتائج ان النقصان في الاحتمالية تتراوح بين (**41-100**) ٪ تم الحصول عليها باستخدام عمودين حجريين باستخدام قيمتين من معامل الانحراف المعياري بينما وجد ان تاثير الهزات الارضية على احتمالية نقصان الفشل يتراوح من(**26-56**)٪ لنفس الظروف الحالة السابقة.**