Applications of Computational Mechanics in Geotechnical Engineering – Sousa, Fernandes, Vargas Jr & Azevedo (eds)
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Design of spread foundations according to the EC7 and the reliability evaluation

F.F. Martins
University of Minho, Guimarães, Portugal

ABSTRACT: This paper presents the philosophy and the concepts of Eurocode 7 (EC7) applied to spread foundations. The five ultimate limit states presented in the EC7 are defined. More emphasis is devoted to the ultimate limit state GEO. It is presented an overview of the reliability methods and the three levels corresponding to the probabilistic methods are defined. It is analysed a spread foundation submitted to vertical loading using all the design approaches of the EC7, the traditional Portuguese approach based on global safety factors and the Spanish codes. The results obtained are compared. Using the Excel's solver it is calculated the Hasofer-Lind second moment reliability index β for three foundation dimensions obtained in the EC7 design and with the other approaches. Several coefficients of variation for soil parameters are used to evaluate the reliability index β.

1 INTRODUCTION

The Eurocode 7 is applied to geotechnical design of buildings and civil engineering works. It is composed by two main parts. The first part (EN 1997-1 2004) is related to general rules of geotechnical design and describes the general principles and requirements that ensure the safety, the serviceability and the durability of the supported structures. The second part includes the geotechnical investigations and field and laboratory testing. The Eurocode 7 must be used in combination with the Eurocode 0 related to bases of structural design, with Eurocode 1 related with actions on structures and with other eurocodes of design of materials. The Eurocode 8 devoted to structural and geotechnical design in seismic regions includes in its part 5 the foundations, the retaining structures and other geotechnical aspects.

According to the system of eurocodes, the design fulfils the requirements of the ultimate limit states when the design value of the actions or the effect of the actions E_d is lower than or equal to the design value of the resistance of the ground and/or the structure, R_d:

$$E_d \leq R_d \tag{1}$$

The Eurocode 7 allows the evaluation of R_d and E_d using three different design approaches. This design approaches are related to different partial factors applied to the representative values of the actions, the characteristic values of the material properties and the resistances. However the choice of the design approach and the partial factors must be defined by the countries through the National Annex.

The EC7 (EN 1997-1 2004) requires that, where relevant, five ultimate limit states shall be considered:

- Loss of equilibrium of the structure or the ground, considered as rigid body, in which the strengths of structural materials and the ground are insignificant in providing resistance (EQU);
- Internal failure or excessive deformation of the structure or structural elements, including e.g. footings, piles or basement walls, in which the strength of structural materials is significant in providing resistance (STR);

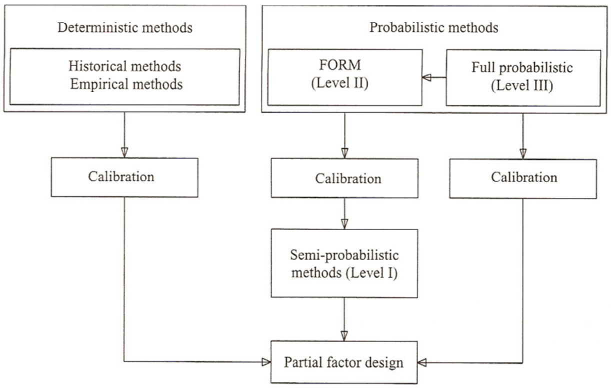

Figure 1. Overview of reliability methods (EN 1990 2002).

Failure or excessive deformation of the ground, in which the strength of soil or rock is significant in providing resistance (GEO);

Loss of equilibrium of the structure or the ground due to uplift by water pressure (buoyancy) or other vertical actions (UPL);

Hydraulic heave, internal erosion and piping in the ground caused by hydraulic gradients (HYD).

The safety in relation to ultimate limits states is applied mainly to persistent and transient situations, and the factors given in Annex A of Eurocode 7 are only valid for these situations.

In this paper it is only considered the ultimate limit state GEO.

The check against the serviceability limit states can be performed in two ways. One way is to require that the design values of the effect of the actions E_d, such as the deformations and settlements, are lower than or equal to the limit values, C_d. Another way is to use a simplified method based on the comparable experience.

2 RELIABILITY METHODS

According to the EN 1990 (2002) the partial factors and the combination factors ψ used in Equation 1 can be evaluated either on the basis of calibration related to a long experience tradition or on the basis of statistical evaluation of experimental data and field observations. These two ways can be used separately or combined. However, most of the factors proposed in the eurocodes are based on the calibration based on long experience tradition.

The available methods for calibration of factors and the relation between them are presented schematically in Figure 1 (EN 1990 2002). The abbreviate designation FORM means first order reliability methods.

It can be seen in Figure 1 that the probabilistic methods are divided in three levels. In relation to the level I, semi-probabilistic methods are used to define the partial factors. In both levels II and III the measure of reliability should be identified with the survival probability $P_s = 1\text{-}P_f$, where P_f is the failure probability for the considered failure mode and within an appropriate reference period. The structure should be considered unsafe when P_f is larger than a pre-set target value P_0. It must be underlined that the probability of failure is only a reference value that does not necessarily represent the actual failure. It is used for code calibration purposes and comparison of reliability levels of structures.

The level III uses exact probabilistic methods and the evaluation of the probability of failure or reliability of the structure is based on the statistical distributions of all basic variables.

In the level II approximated probabilistic methods are used and the reliability is measured using the reliability index β instead of the probability of failure. The reliability index β is related to P_f by:

$$P_f = \Phi(-\beta) \tag{2}$$

where Φ is the cumulative distribution function of the standardised normal distribution. The increasing of the β values corresponds to the decreasing of the probability of failure P_f.

The probability of failure can be expressed by:

$$P_f = \text{Prob}(g \leq 0) \tag{3}$$

where g is a performance function given by:

$$g = R - E \tag{4}$$

where R is the resistance and E the effect of actions. R, E and g are random variables.

The structure is considered to survive if $g > 0$ and to fail if $g \leq 0$.

According to Favre (2004), Cornel, in 1969, was the first that proposed a measure of safety under the form of the index β:

$$\beta = \frac{\mu_g}{\sigma_g} \tag{5}$$

where μ_g is the mean value of g and σ_g is the standard deviation of g.

This index β is not invariable in relation to the formulation of the performance function.

To overcome this situation, five years later, Hasofer and Lind proposed a new definition for β (Favre 2004):

"The shortest Euclidian distance, in the reduced Gaussian space, from the origin to the performance equation $\Sigma(y) = 0$"

where $\Sigma(y)$ is the transformed performance function in the reduced Gaussian space.

The theory related with the evaluation of β can be found in the specialized bibliography as, for example, in Favre (2004).

More recently Low & Tang (1997) proposed an efficient method using spreadsheet software for calculating the Hasofer-Lind second moment reliability index. The method is based on the perspective of an ellipse that is tangent to the failure surface in the original space of variables. Iterative searching and numerical partial differentiation are performed automatically by a spreadsheet's optimization tool.

The matrix formulation is the following:

$$\beta_{HL} = \min \sqrt{\left(X - \mu_X^N\right)^T C^{-1} \left(X - \mu_X^N\right)} \tag{6}$$

Restrained to g(X)=0

where X is a vector representing the set of random basic variables which include the effect of actions E and resistances R; μ_X^N is the vector of the mean values of the basic variables X with the upper index N meaning normal or equivalent normal distribution; and C is the covariance matrix.

In the case of variables of non-normal distribution it is necessary to establish relationships between non-normal distribution and its equivalent normal distribution. This can be obtained by equating the cumulative probability and the probability density ordinate of the equivalent normal distribution with those of the corresponding non-normal distribution at the design point X*. This leads to the following equations:

$$\sigma_{X_i}^{N}\left(x_i^*\right)=\frac{\phi\left\{\Phi^{-1}\left[F_{X_i}\left(x_i^*\right)\right]\right\}}{f_{X_i}\left(x_i^*\right)} \tag{7}$$

$$\mu_{X_i}^{N}\left(x_i^*\right)=x_i^*-\sigma_{X_i}^{N}\left(x_i^*\right)\Phi^{-1}\left[F_{X_i}\left(x_i^*\right)\right] \tag{8}$$

where $\Phi^{-1}[.]$ is the inverse of the cumulative probability of a standard normal distribution; F_{Xi} (x_i*) is the original cumulative probability evaluated at x_i*; $\phi\{.\}$ is the probability density function of the standard normal distribution; and $f(x_i*)$ is the original probability density ordinates at x_i*.

This method was implemented in this paper using the Excel's solver which is invoked to minimize β, by changing the values of the X vector, subject to $g(X) = 0$.

3 DESIGN METHODS OF SPREAD FOUNDATIONS

The methods used to verify the design of foundations are the direct method, the indirect method and the prescriptive method.

In relation to the direct method it is necessary to perform two separate verifications. One for ultimate limit states and other for serviceability limit states. For both limit states it is necessary to use a calculation model that may be numerical, analytical or semi-empirical. The last model is based on in situ test results.

The indirect method is based on comparable experience and uses the results of field or laboratory tests or other observations. This method covers both the ultimate limit states and the serviceability limit states and in the calculations may be used analytical and semi-empirical models.

In the prescriptive method the design is evaluated on the basis of comparable experience. The calculation model may include charts or tables.

4 BEARING RESISTENCE USING THE DIRECT METHOD

In relation to the bearing resistance of a spread foundation the following inequality shall be satisfied:

$$V_d \leq R_d \tag{9}$$

where V_d is the design value of vertical load or component of the total action acting normal to the foundation base and R_d is the design value of the resistance.

The design value of any component F_d of V_d shall be derived from representative values using the following equation:

$$F_d = \gamma_F \times F_{rep} \tag{10}$$

with

$$F_{rep} = \psi \times F_k \tag{11}$$

Values of ψ are given by EN 1990 (2002) and values of partial factor γ_F are given in Table 1.

R_d can be calculated through the analytical expressions presented in the sample given in Annex D of EC7 (Part 1). The design values of the strength parameters of the ground used in these expressions are obtained by dividing its characteristic values by the partial factors presented in Table 2. The resistance is also divided by a partial factor given in Table 3.

Table 1. Partial factors on actions (γ_F) or the effects of actions (γ_E).

Action		Symbol	Set A1	Set A2
Permanent	Unfavourable	γ_G	1.35	1.0
	Favourable		1.0	1.0
Variable	Unfavourable	γ_Q	1.5	1.3
	Favourable		0.0	0.0

Table 2. Partial factors for soil parameters (γ_M).

Soil parameter	Symbol	Set M1	Set M2
Angle of shearing resistance*	$\gamma_{\varphi'}$	1.0	1.25
Effective cohesion	$\gamma_{c'}$	1.0	1.25
Undrained shear strength	γ_{cu}	1.0	1.4
Unconfined strength	γ_{qu}	1.0	1.4
Weight density	γ_{γ}	1.0	1.0

*This factor is applied to tan φ'.

Table 3. Partial resistance factors (γ_M) for spread foundations.

Resistance	Symbol	Set R1	R2	R3
Bearing	$\gamma_{R,v}$	1.0	1.4	1.0
Sliding	$\gamma_{R,h}$	1.0	1.1	1.0

Table 4. Combinations for the different design approaches.

Design approach	Combination
1	A1′ + M1′ + R1
	A2′ + M2′ + R1
2	A1′ + M1′ + R2
3	A1 or A2′ + M2′ + R3

The manner in which the partial factors are applied shall be determined using one of three design approaches given in Table 4.

In relation to Design Approach 3 the partial factor A1 is applied on structural actions and A2 is applied on geotechnical actions.

As was mentioned before the Annex D of EC7 presents a sample analytical method for bearing resistance calculation of a spread foundation both on drained and undrained conditions. As the example presented in this paper is related to drained conditions it is presented below the analytical equation used in these conditions:

$$R/A' = c'N_c b_c s_c i_c + q'N_q b_q s_q i_q + 0{,}5\gamma' B' N_\gamma b_\gamma s_\gamma i_\gamma \tag{12}$$

Table 5. Characteristics of the analysed foundation and settlements.

B m	L m	D m	p kPa	V kN	q_c kPa	Es kPa	Settlements (cm) Computed	Measured
2.6	22.8	2.0	179	10611.12	3924	14842.9	3.68	3.89

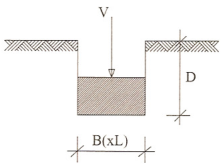

Figure 2. Geometrical characteristics and applied load on foundation.

where c′ is the effective cohesion, q′ is the overburden pressure at the level of the foundation base, γ' is the effective weight density of the soil below the foundation level, B′ is the effective foundation width, N are the bearing capacity factors, b are the factors for the inclination of the base and i are the factors for the inclination of the load. The subscripts c, q, γ used with b, s and i are related to cohesion c, overburden pressure q and weight density γ.

In the example presented here are performed analyses based on EC7, traditional Portuguese analyses using the global safety factors and the Hansen and Vesić methods (Bowles 1996) and analyses based on Spanish geotechnical codes (Recomendaciones Geotécnicas para el Proyecto de Obras Marítimas y Portuarias, ROM 05-94, Documento Básico DB-4 "Cimentaciones" do Código Técnico de La Edificación e Guia de Cimentaciones de Obras de Carretera (Perucho & Estaire 2005)).

5 EXAMPLE

The example presented here is related to a case studied by Schmertmann and presented by Bowles (1996). It was already presented by the author in the part related to the EC7 (Martins 2006) which is presented here. However the part related to the reliability evaluation is only introduced in this paper. It is a shallow foundation of a bridge pier settled on silty sand. Table 5 presents the geometric characteristics of the foundation (Fig. 2), the value of contact pressure, p, the corresponding vertical action, V, the tip resistance from CPT-test, q_c, the Young's modulus, E_s, the computed settlements by a method presented by Bowles (1996) as well as the measured values presented by the same author.

For the EC7 calculations the values presented in Table 5 are considered as characteristics.

Bowles (1996) says neither the percentages of V corresponding to the permanent and the variable actions nor the angle of shearing resistance. In relation to the actions it is considered here 60% for the permanent actions and 40% for the variable actions. The value of the angle of shearing resistance was established on the base of the Table 6 presented in a provisory version of EC7-Part 3 (ENV 1997-3 1995) which related the tip resistance from CPT-test, q_c, with the angle of shearing resistance, φ', and with the Young's modulus, E_s. Whereas the soil is silty it was considered a value slightly less than those obtained in Table 6. Therefore, it was considered $\varphi' = 32°$.

Table 6. Angle of shearing resistance φ' and Young's modulus E_s for sands from cone resistance q_c.

Relative density	q_c MPa	φ' °	Es MPa
Very low	0.0–2.5	29–32	<10
Low	2.5–5.0	32–35	10–20
Medium	5.0–10	35–37	20–30
High	10.0–20.0	37–40	30–60
Very high	>20.0	40–42	60–90

Table 7. Information related to random basic variables.

	G_v kN	Q_v kN	$\tan\varphi'$	γ' kN/m^3
X_k	6366.67	4244.45	0.6249	10
V_x%	1	10	5–10–15	5–7.5–10

To evaluate the reliability it is used the following vector X representing the set of basic random variables for this situation:

$$X^T = \{G_v, Q_v, \tan\varphi', \gamma'\} \tag{13}$$

where G_v is the vertical permanent action; Q_v is the vertical variable action; $\tan\varphi'$ is the tangent of the angle of shearing resistance of the soil and γ' is the effective weight density of the soil. All the other variables were considered as constant.

According to Schneider (1997) cited by Orr and Farrel (1999) the coefficient of variation of $\tan\varphi'$ ranges from 0.05 and 0.15 and of γ ranges from 0.01 to 0.10. Based on these limits and the values presented by Serra & Caldeira (2005), the values presented in Table 7 were considered for the random basic variables.

According to the EN 1990 (2002) normal distributions have usually been used for self-weight and extreme values are more appropriated for variable actions. However, lognormal and Weibull distributions have usually been used for material and structural resistance parameters and model uncertainties. Based on these considerations it was assumed a normal distribution for G_v and γ' and, for sake of simplicity, a lognormal distribution for $\tan\varphi'$ and Q_v.

The equations used to evaluate the mean of these variables are the following (Serra & Caldeira 2005):

$$\mu_{G_v} = \frac{G_{vk}}{1 \pm 1.645 V_{Gv}} \tag{14}$$

$$\mu_{Q_v} = e^{\ln(Q_{vk}) \mp 1.645 V_{Q_v} + 0.5\ln\left(V_{Q_v}^2 + 1\right)} \tag{15}$$

$$\mu_{\tan\varphi} = e^{\ln(\tan\varphi_k) + 0.67 V_{\tan\varphi} + 0.5\ln\left(V_{\tan\varphi}^2 + 1\right)} \tag{16}$$

$$\mu_\gamma = \frac{\gamma_k}{1 - 0.67 V_\gamma} \tag{17}$$

The sign $\pm$ in Equation 14 and $\mp$ in Equation 15 allows considering the vertical action as favourable (upper sign) and unfavourable (lower sign) for the foundation safety.

Table 8. Synopsis of the results for the bearing resistance.

Approach	V kN	R kN	R/V
DA-1-Comb2	11884.45	24307.20	2.05
DA-1-Comb1	14961.68	49766.63	3.33
DA2	14961.68	35547.59	2.38
DA3	14961.68	24307.20	1.62
Hansen	10611.12	44427.02	4.19
Vesić	10611.12	51658.93	4.87
Código Técnico	10611.12	45886.23	4.32
Guia Cimentaciones	10611.12	50152.45	4.73
ROM 0.5-94	10611.12	44812.84	4.22

The equivalent normal parameters can be obtained by:

$$\mu_{X_i}^N = X_i\left(1 - \ln(X_i) + \lambda_{X_i}\right) \tag{18}$$

$$\sigma_{X_i}^N = X_i \zeta_{X_i} \tag{19}$$

where λ_{X_i} and ζ_{X_i} represent the mean value and the standard deviation of the normal variable $Y_i = \ln(X_i)$.

5.1 *Design according to the EC7 (Level I)*

Table 8 presents the results obtained for the three design approaches of EC7, the traditional method (Hansen and Vesić) used in Portugal and the Spanish codes. For the EC7 approaches both the actions V and R are design values whereas for the other situations those values are not affected by any safety factor.

To obtain the foundation allowable load either in the traditional Portuguese calculations or using the Spanish codes it is used a safety factor equal to 3 to lower the resistance R. In the Portuguese case this factor ensures implicitly that the maximum allowable settlement is not surpassed. That's why the settlement was not computed. In the Spanish case, according to Perucho & Estaire (2005), it is also necessary to verify the settlements.

In all the computations performed by the Hansen method, Vesić method and Spanish codes it was obtained a safety factor greater than 3, surpassing in all of them 4. In these cases the higher safety is obtained using the Vesić method (4.87) and the lower safety is obtained through the code ROM 0.5-94 (4.22). In the case of EC7 is the design approach 3 that presents lower safety ($R/V = 1.62$) and, therefore, it determines the design in this study.

In relation to the serviceability limit states, as it can be seen, the measured settlement is lower than the value considered allowable for bridge piers, which is 5 cm (Seco e Pinto 1997).

Next it will be maintained the ratio L/B and the "optimal" width of the foundation will be calculated. This width, in the case of EC7, is that that lead to the equating between the design vertical action, V_d, and the design bearing resistance, R_d. In the Hansen method, Vesić method and Spanish codes that width will correspond to a ratio $R/V = 3$. The obtained results are presented in Table 9.

As it can be seen, in the EC7 case, is the Design Approach 3 that determines the dimensions of the foundation ($B = 2.11$ m) and in the other approaches are the Hansen method and the code ROM 05-94 that lead to larger width ($B = 2.25$ m), nevertheless the other analysis lead to values very close to this.

In relation to the serviceability limit states the values obtained for the settlements using the procedure of Bowles (1996) are also presented in Table 9 and, as it can be seen, are all lower than 5 cm.

Table 9. "Optimal" foundation width maintaining the ratio L/B and corresponding settlements.

Approach	B m	Settlement cm
DA-1-Comb2	1.91	–
DA-1-Comb1	1.56	–
DA2	1.81	–
DA3	2.11	4.45
Hansen	2.25	4.20
Vesić	2.13	4.41
Código Técnico	2.22	4.26
Guia Cimentaciones	2.15	4.37
ROM 0.5-94	2.25	4.20

Table 10. Values of β_{HL} for favourable actions.

B	$V_{\gamma'} \setminus V_{\tan\varphi'}$	5%	10%	15%
1.91	5%	6.81	5.46	5.09
	7.5%	6.61	5.46	5.09
	10%	6.33	5.44	5.10
2.11	5%	8.36	6.69	6.24
	7.5%	8.06	6.68	6.25
	10%	7.67	6.65	6.24
2.25	5%	9.11	7.00	6.40
	7.5%	8.68	6.97	6.41
	10%	8.13	6.93	6.40

Therefore, in the analysed foundation, the design according to EC7 is determined by the ultimate limit states and in the traditional Portuguese approach the use of a global safety factor equal to 3 covers the serviceability limit states. The computed settlements for the Spanish codes are also below the settlements considered allowable.

5.2 *Reliability evaluation (Level II)*

Two sets of computations were performed. In the first set the actions were considered as favourable and in the second set the actions were considered as unfavourable. For each set were considered the values corresponding to the "optimal" values obtained in Design Approach 1 (1.91 m), Design Approach 3 (2.11 m) and Hansen and ROM Approaches (2.25 m).

To analyse the sensibility of β to the variation of the geotechnical parameters $\tan\varphi'$ and γ', nine combinations of the coefficient of variation of $\tan\varphi'$ and γ' were performed for each B value. Due to lack of information, it wasn't established in this paper any correlation between the soil parameters.

The obtained values are presented in Tables 10 and 11.

It can be seen in Tables 10 and 11 that the reliability index decreases more pronouncedly with the increase of $V_{\tan\varphi'}$ than with the increase of $V_{\gamma'}$. For higher values of the coefficient of variation of $\tan\varphi'$ the reliability index almost doesn't change with the change of the coefficient of variation of γ' for the same foundation width.

Generally the β values are lower for unfavourable actions and increase with the increase of the foundation area. This is in accordance with the expected because lower β values lead to greater probability of failure.

Table 11. Values of β_{HL} for unfavourable actions.

B	$V_{\gamma'} \backslash V_{\tan\varphi'}$	5%	10%	15%
1.91	5%	5.39	3.54	2.93
	7.5%	5.14	3.54	2.95
	10%	4.78	3.52	2.96
2.11	5%	7.26	5.26	3.50
	7.5%	6.93	5.27	3.51
	10%	6.51	5.27	3.51
2.25	5%	8.12	5.64	3.77
	7.5%	7.64	5.63	3.78
	10%	7.04	5.61	3.78

The design using EN 1990 (2002) with partial factors given in Tables 1 to 3 is considered generally to lead to a structure with a β value greater than 3.8 for a 50 year reference period. In the case of favourable actions all the β values are greater than this minimum recommended value. Nevertheless, in the case of unfavourable actions there are cases where the β values are greater than the minimum recommended value. This is the case corresponding to B equal to 1.91 m and the coefficient of variation of tanφ' is equal to 10% and 15% and the case corresponding to B equal to 2.11 m and 2.25 m and the coefficient of variation of tanφ' equal to 15%.

Nevertheless, the higher values of $V_{\tan\varphi'}$ lead to a very broad variation of φ' (Serra & Caldeira 2005). This can be explained by an incorrect evaluation of the soil parameters or an important heterogeneity of the soil. In the latter case the formulation of the Annex D of the EN 1997-1 is meaningless because it is only applicable to homogeneous soil.

6 CONCLUSIONS

As it can be seen, for vertical loading, in the EC7 case, is the Design Approach 3 that led to larger foundation dimensions. However, the traditional approaches (Hansen and Vesić) and the Spanish codes led to larger values of the foundation dimensions. The computed settlements obtained in all the approaches are very close. However the higher settlement is obtained with the dimensions obtained with the EC7 calculations.

The mean of resistances obtained through the Hansen and Vesić methods is about 66% of the value of the bearing resistance obtained for EC7. In relation to the Spanish codes this value is about 65%. The design bearing resistance R_d obtained through EC7 is close to the medium value of the resistance obtained with the Spanish codes dividing the resistance value by 2. The difference is of 3.4% and is similar to that obtained by Perucho & Estaire (2005) that is around 3%. Considering the mean of the resistance values obtained through the Hansen and Vesić formulae the difference is of 1.2%.

The verification in relation to the serviceability limit state doesn't govern the design for any of the design approaches whereas the total settlement is lower than the allowable settlement of 5 cm.

In the traditional Portuguese calculations it is current practice to adopt a global factor of safety of 3 for drained conditions considering that the use of this factor ensures that the allowable settlement is not exceeded. In the analysed case this allowable settlement is not exceeded.

In relation to the reliability, as the values of β were obtained based on initial simplifications and assumptions, its values less than 3.8 don't necessary mean that the minimum safety is not satisfied. It must be stressed that the results are influenced by several factors such as the assumptions related to the actions, the statistical distributions of the basic variables and the correlation between them. Therefore, the conclusions presented here shouldn't be generalized to all the situations.

ACKNOWLEDGEMENTS

This work was financed by Foundation for Science and Technology. Project POCI/ECM/57495/2004 – Geotechnical risk for tunnels in high-speed trains.

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