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Probabilistic Evaluations of Prescribed Safety Margins in Eurocode 7 for Spread Foundations

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Abstract

According to the design code Eurocode 7, analysis procedures require reaching a prescribed safety margin, based on the conditions of levelling design action and design resistance. Such semi-probabilistic procedures do not result in a consistent equivalent value of the Overall Factor of Safety (OFS), neither in individual analysis nor in different tasks in geotechnical engineering. Furthermore, the implementation of different calculation approaches in Eurocode 7 also does not guarantee an equal probability of the occurrence of a relevant limit state. A comparative analysis is conducted for an example of a centrically loaded spread foundation on homogenous, isotropic, and coarse-grained soil, according to procedures in Eurocode 7, Design Approach 3. An algorithm is developed to estimate failure probability, taking into consideration the relevant statistical characteristics of each calculation parameter. A significant influence of the statistical characteristics of the relevant sample is emphasized. The degree of required modification of the equivalent Overdesign Factor (ODF) and the Overall Factor of Safety (OFS), based on the criterion of the required reliability index β and failure probability p_f, is quantified.

Keywords

geotechnical engineering, Eurocode 7, shallow foundations, probability of failure, reliability analysis

1 Introduction

The geotechnical limit state analysis, as given Eurocode 7 design code [1], is based on the use of recommended, individual values of partial safety factors (PSF). However, respecting the right to determine values related to regulatory safety matters at a national level, the code offers the possibility of implementing different values through the national annexes [2]. The survey conducted by Sousa et al. [3] found that the Eurocode part with the lowest percentage of recommended PSF values acceptance, within the national annexes, is exactly Eurocode 7, with an average of only 50% of the recommended values been accepted without any change. This is explained by the very different national practices in the field of geotechnical engineering design.

In geotechnical design, specific values of PSFs depend on the relevant limit state (five possible states, taking into consideration the soil and the structure), the position of their implementation (action or resistivity) and the type of the analyzed engineering task (spread foundation, piles, geotechnical anchors, retaining structures, slopes and overall stability). Depending on the adopted calculation approach for a specific design situation, a combination of implementing PSFs for action and resistance (material) is defined. However, the current version of Eurocode 7 prescribes values of PSFs for ultimate limit states for persistent, transient and accidental situations. During the design process, a geotechnical engineer must select a set of characteristic values and the corresponding PSFs, hoping to obtain in the end a design that satisfies a prescribed reliability level [4]. The implementation of PSFs in the field of structural engineering is based on a probabilistic basis of analyzing an infinite number of relevant samples. Geotechnical engineering, however, fosters a different approach. It is based on defining the PSFs value, which will ensure a uniform level of safety, in accordance with the to-date engineering practice. As Kovačević et al. [5] note, the practical geotechnical engineers appear reluctant to adopt probability-based methods, which are perceived as too complex and impractical for use, mostly due to due

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to a common concern on insufficient number of samples. This, along with the unknown statistical properties of soil materials [6], leads to a prevailing misconception that these methods require considerably more effort in comparison to traditional design methods. However, when it comes to the analyses of spread foundations, the geotechnical community has been more progressive in the implementation of different probability - based methods [4, 7–15].

The concern of an insufficient number of samples to perform the probabilistic analysis is the motive for a more comprehensive study which includes the assessment and quantification of the reached, or prescribed, safety margins. This can be achieved by further development of the procedures for evaluation of the adopted design approaches, through acquisition of a relevant data, evaluated from a probabilistic perspective, such as the type and quantity of field and laboratory investigation works, empirical correlations, computation approaches, model factors, etc., further subjected to the statistical analysis. Phoon and Kulhawy [16] identify three main sources of statistical dispersion: inherent variability, measurement error and transformation uncertainty. While the inherent soil variability, parameterized by aleatory uncertainty, describes the variation of properties from one spatial location to another, as noted by the Fell et al. [17], the measurement error implies the scatter of measurements on presumably homogeneous soil volumes, and is conceptually related to precision. In order to relate the on-site and laboratory test results to the design parameters, a transformation models are used, where in the process of model characterization, some degree of uncertainty is introduced. In addition to these sources, a statistical uncertainty can also influence statistical dispersion [18]. To obtain a more comprehensive insight into the formal aspects of statistical approaches in the context of geotechnical engineering, the reader is referred to a several relevant literatures, starting from the early works of Lumb [19] to the most recent papers on the topic [20].

By recognizing the weaknesses of semi-probabilistic approach in providing a consistent equivalent value of the Overall Factor of Safety (OFS), the European geotechnical engineering community is currently making significant efforts towards the revision and improvement of Eurocode 7, which will encompass a wide range of modifications. The current version of Eurocode 7 allows the application of full probability methods as an alternative to the semi-probabilistic approach, however the procedures for doing this are described rather vaguely. Clause 2.4.5.2 (10) of Eurocode 7 states that statistical methods may be used when selecting characteristic values of geotechnical parameters [21], but these methods are not mandatory. Low and Phoon [22] note that, although uncertainties have been considered in Eurocode 7 in an approximate way, both spatial correlation between the same parameter at different sampling points, and cross-correlation between different parameters at the same sampling point, are not considered. While it seems that the semi-probabilistic approach will remain in focus of the revised version of the standard, the new version will acknowledge the probabilistic analysis in more proper manner, eventually allowing designers to estimate and quantify the influence of a particular source of statistical dispersion on the complete design process in order to achieve the required safety margins. One significant modification [23] will include implementation of three Consequences Classes (CC), associated with the three Reliability Classes (RC) related to the different target reliability indexes (β).

This paper contributes to the aforementioned efforts through development of an algorithm to estimate spread foundation failure probability, taking into consideration the relevant statistical characteristics of each calculation parameter. This will provide an improvement of the current approach, where means and methodology of measuring the reached safety margins are not entirely clear, nor are they explicitly defined. When dealing with different assignments in geotechnical engineering, it is necessary to research the factor of their deviation in terms of failure probability, using a prism of the established values of the Overdesign Factor - $ODF = R_d / E_d$ (R_d – design resistance, E_d – design value of the effect of actions), and the equivalent Overall Factor of Safety *OFS*. Unlike some previous works dealing with *ODF* – *OFS* relation and with β – coefficient of variation relation, such as Orr and Breysse [24], this paper aims to show the interdependence of the probability of failure with the prescribed safety margins according to EC7 (*ODF*) and the classical approach (*OFS*), by establishing a β–*ODF*–*OFS* correlation.

2 Practices prior to Eurocode 7 adoption

The analysis of approaches used in engineering practice prior to the adoption of Eurocode 7 is motivated primarily by a need to directly compare the reached solutions from an aspect of quantifying the reached safety levels. The classical approach is methodologically based on the concept of analyzing the actual stress state, q. The value of this stress is the one necessary to reach an equilibrium in the limit analysis of individual cases. The corresponding limit resistance is divided by the Overall Factor of Safety, or the parameters of soil shear strength are factored. In the case of computations using the Overall Factor of Safety, the evidence format for a spread foundation is as follows:

$$q \leq q_a = q_u / OFS \ , \tag{1}$$

where:

q_u - ultimate value of the foundation contact stress, defined from non-factored parameters of the soil's shear strength,

OFS - Overall Factor of Safety, usually having the value of 2–3,

q_a - allowable foundation contact stress,

q - actual value of foundation contact stress, i.e., the value required for balancing the external actions and resistance.

This approach provides immediate and intuitive insight into the reached safety margin since it is possible to directly verify the allowed value deviation in respect to the actual stress state. The Overdesign factor *ODF* in this case is represented as follows:

$$ODF = q_u / q \ . \tag{2}$$

If *ODF* = *OFS*, there is an optimal value, i.e., achieving of the prescribed safety margin. If *ODF* < *OFS*, a reduction of the prescribed margin is present, while *ODF* > *OFS* yields a conservative solution. In addition to the immediate insight into the degree of reaching the required safety margins, it is also possible to clearly identify how far is the reached solution from failure, which occurs when *ODF* = 1, i.e., the equivalent *OFS* = 1.

The procedures of calculation the *OFS* are generally not probabilistic. Consequently, there is no strict definition of the means to reach the calculated values of soil strength parameters. The usual approach is to define it as the mean value of a specific sample.

3 The semi-probabilistic approach adopted by Eurocode 7

The prescribed design approaches in Eurocode 7 are based on a direct analysis of limit cases in the relation of resistance and action, where the concept of the Overall Factor of Safety is dismissed. Three design approaches are used (DA1 (DA1_1, DA 1_2), DA2 (DA2*), DA3). Factoring is implemented onto actions (A - Action), materials (M - Material) and resistances (R - resistance). Table 1 outlines a schematic overview of PSF for each the design approach. Double underlined corresponding letters indicate cases in which the values of PSFs are not unit values (≠ 1).

Table 1 Design approaches PSF combinations

DA	DA1_1	DA1_2	DA2 (DA2*)	DA3
Comb.	A1+M1+R1	A1+M2+R1	A1+M1+R2	A1(A2)+M2+R1

In DA3, A1 stands for structural action, while A2 stands for geotechnical action. The general condition format is as follows:

$$E_d \leq R_d \ , \tag{3}$$

where

E_d - design value of the effect of actions,

R_d - design value of the resistance to an action.

The Overdesign Factor *ODF* can now formally be defined as the relation between design resistance and design action:

$$ODF = R_d / E_d \ . \tag{4}$$

If *ODF* = 1 the prescribed margin of safety is achieved, whereas if *ODF* < 1 the reached level is lower than prescribed, while *ODF* > 1 follows conservative solution.

Since the limit state analysis is conducted with design values of action and resistance, there is no direct insight into the achieved safety margin relating to the actual limit values. In the case of *ODF* < 1, the prescribed margin is not achieved, but it is not straightforward how much the value is to be reduced to achieve the appropriate limit state. Specifically, *ODF* values lesser than one ensure safety that is less than prescribed, but without immediate failure. Failure occurs only when the equivalent value of the *OFS* is equal to one.

The basis of the Eurocode design approach comes from the probabilistic choice of PSF values for unfavorable, favorable, permanent, or variable action effects, as well as shear strength parameters, or characteristic resistance. As rigorous approach to the choice of PSF values is conducted, with no possibility of their adjustment, the range of possible calculation results is inevitably linked with the procedures of choosing representative, characteristic, values of relevant soil shear strength parameters, which in case of spread foundations affect the achieved resistivity. The selection of soil characteristics is done via an adopted concept of characteristic values (X_k), with a prescribed probability of not being exceeded in a hypothetically unlimited test series [25]. This approach is not directly applicable to the geotechnical engineering due to the uncertainty which is the consequence of natural randomness, inaccuracies in a prediction from tests and interpretation of their results.

4 The full-probabilistic approach

Probabilistic analysis is based on the estimated probabilistic distribution of all relevant parameters of actions and resistance. The probability of failure occurrence is determined for a specific limit state, based on the adopted distributions of individual parameters. The means of implementing the described procedure depends on the adopted computation model (analytic, semi-probabilistic, Monte Carlo etc.).

It is necessary to determine the appropriate parameters for an analyzed task in geotechnical engineering. The computation task comes down to the determination of the reliability index value β in relation to the prescribed value of failure occurrence probability pf. Table 2 indicates the relation between β and p_f. The recommended values of the target index of reliability according to [25], for reference periods of structure exploitation, are β = 4.7 for one-year period, and β = 3.8 for 50 year period.

Based on to date practices in geotechnical engineering according to Phoon and Ching [26], the target value β is usually chosen for failure probability p_f = 0.1%, thus β = 3.1. For highly redundant geotechnical structures, such as pile group or reinforced soil, the target value is within β = 2.0 – 2.5. The choice of the final target reliability index value in this case dominantly depends on the possibility of resistance redistribution from one element to another.

The relationship between these two values, i.e., the analytic expression is given in the following formula:

$$\beta = -\Phi^{-1}\left(P_f\right), \tag{5}$$

where Φ^{-1} (P_f) is Inverse Gaussian Distribution.

Fig. 1 shows the relation between the reliability index β and probability of failure p [%]. The graph of the relation between the reliability index β and failure probability p_f [%] shows target values within the unsatisfactory and hazard (unacceptable) values, i.e., dependent on the consequences of reaching a limit state for spread foundation (high, medium and low). By cancelling out one of the system's elements, there is no resistance redistribution to adjoining elements.

Table 2 The relations between the reliability index and failure probability, and the expected reliability levels

Reliability index β	Failure probability p_f [%]	Expected reliability level
0.5	31	
1.0	16	Hazard
1.5	6.77	Unsatisfactory
2.0	2.3	Bad
2.5	0.62	Below average
3.0	0.13	Above average
4.0	0.003	Good
5.0	0.00003	High

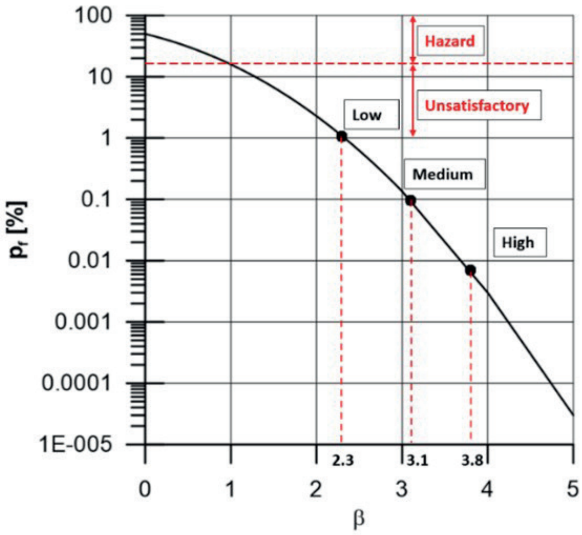

Fig. 1 The relation between the reliability index β and probability of failure p_f [%]

The main soil parameter incorporated in the proposed probability calculation procedure is the soil internal friction angle. The effect of a range of standard deviations on the required width of the foundation, and finally on the values of the reliability index β and failure probability p_f will be evaluated. Other parameters of the probabilistic analysis (soil unit weight, concrete unit weight, permanent and variable action) were defined as the constant values of standard deviation obtained from the relevant literature.

The Taylor series method is used in the analyses for the calculation of the reliability index, according to [27–30]. The method is part of the FOSM (First Order Second Moment) method class since the analysis uses the first two "moments" (expectation and standard deviation) of the function. A linearization of the function in the point of greatest probability, i.e., its expected value, is conducted. The utilization of the Taylor series method is shown in [28, 29, 31, 32]. The method comprises of two parts. The first part determines the variation coefficient value for a specific variable, e.g., the safety factor, while the second part determines the probability of failure occurrence based on the estimated normal or lognormal distribution of the obtained results.

Foundation soil is coarse-grained, homogeneous, and isotropic without groundwater. The geometry parameters are introduced into the calculation as deterministic variables [33, 34].

ODF value (Eq. (6)) is a function of the parameters design values ($X_{d,i}$). In the case of shallow foundation, there are five parameters which are considered as random variables: the angle of internal friction $\phi'_{(di)j}$, soil unit weight γ_{sd}, concrete unit weight γ_{cd} permanent action G_d and variable action Q_d.

$$DF = \frac{R_{di,j}\left(\phi'_{(di),j}, \gamma_{sd}, \gamma_{cd}, G_d, Q_d\right)}{E_{di,j}\left(\phi'_{(di),j}, \gamma_{sd}, \gamma_{cd}, G_d, Q_d\right)} \qquad (6)$$

OFS was obtained using characteristic values of random variables:

$$OFS = \frac{R_{ki,j}\left(\phi'_{(ki),j}, \gamma_{sk}, \gamma_{ck}, G_k, Q_k\right)}{E_{ki,j}\left(\phi'_{(ki),j}, \gamma_{sk}, \gamma_{ck}, G_k, Q_k\right)}. \qquad (7)$$

Design values were calculated using the characteristic values in accordance with Eurocode 7, DA3 [1]. Characteristic value of a geotechnical parameter (X_k) can be calculated based on its mean and standard deviations as follows [35]:

$$X_k = X_m - 0.5 \times \sigma_X , \qquad (8)$$

where

X_m - geotechnical parameters mean value,

σ_X - standard deviation of X.

For external actions, the characteristic value is assumed to correspond to the mean value [25].

Contrary to Eurocode 7, which prescribes constant values of the PSF, the implementation of probability approach would offer advantage of providing different values of these factors depending on the soil variability. While Eurocode 7 has purposely not been prescriptive concerning how soil variability and correlation uncertainty should be considered, and hence how cautious the characteristic value should be, Orr et al. [36] conclude that more clarification on this should be given in the revised version of Eurocode 7 or in guideline documents.

5 Evaluation of the probability approach – a centrically loaded foundation

A full-probability analysis is conducted for five random variables: soil internal friction angle and unit weight, footing concrete unit weight, permanent and variable unfavorable action.

An Eq. (6) is used for the calculation of the soil failure limit state, foundation bearing capacity, according to Eurocode 7 Annex D [1], for coarse-grained soil for the special case of a centrically loaded foundation in homogenous and isotropic soil characterized the by soil internal friction angle ϕ and unit weight γ.

$$\frac{R}{A'} = q' N_q s_q + 0.5 \gamma' B' N_\gamma s_\gamma , \qquad (9)$$

with:

$$N_q = e^{\pi \operatorname{Tan} f'} \tan^2\left(45° + \frac{f'}{2}\right), \qquad (10)$$

$$N_\gamma = 2\left(N_q - 1\right) \tan f' , \qquad (11)$$

$$s_q = 1 + \left(\frac{B'}{L'}\right) \sin f' , \qquad (12)$$

$$s_\gamma = 1 - 0.3\left(\frac{B'}{L'}\right). \qquad (13)$$

The task is to determine the foundation width value, in this case $B' = B$ from the condition of the prescribed safety margin, for a given foundation length value of L = 3 m, t_f = 0.8 m, t = 1.0 m, Fig. 2.

The analysis was conducted for the mean values of two soil internal friction angles: ϕ'_m = 28° and 40°, for the following expected range of standard deviation, i.e., coefficient of variation [27], COVϕ'_m = 2.5–12.5 %.

The variables from Fig. 2 are:

G_m - mean value of the permanent action,

Q_m - mean value of the variable action,

γ_{cm} - mean value of the unit weight of the concrete foundation,

ϕ'_m - mean value of the soil internal friction angle,

γ_{sm} - mean value of the soil unit weight.

The coefficient of variation of four parameters used in analysis have chosen value as shown in Table 3.

For predefined actions, as well as for shear strength parameters of the foundation soil, it is necessary to determine the foundation width B. The criterion for selecting the required foundation width is defined by achieving the given (optimal) safety margin according to Design Approach 3, i.e., in the case $ODF = R_d/E_d = 1$. Fig. 3. shows

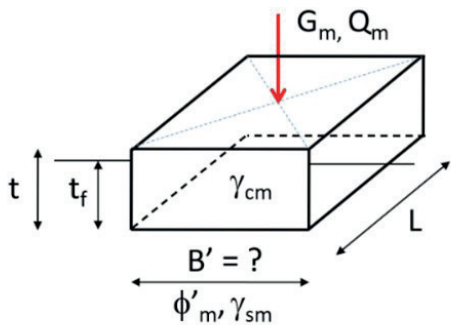

Fig. 2 An example of centrically loaded spread foundation

Table 3 Statistic characteristics of random variables

Parameter	Mean value	Coefficient of variation COV [%]
Permanent action	G_m = 700 kN	5
Variable action	Q_m = 700 kN	15
Unit weight, foundation soil	γ_{sm} = 19 kN/m^3	10
Unit weight, foundation concrete	γ_{cm} = 25 kN/m^3	5

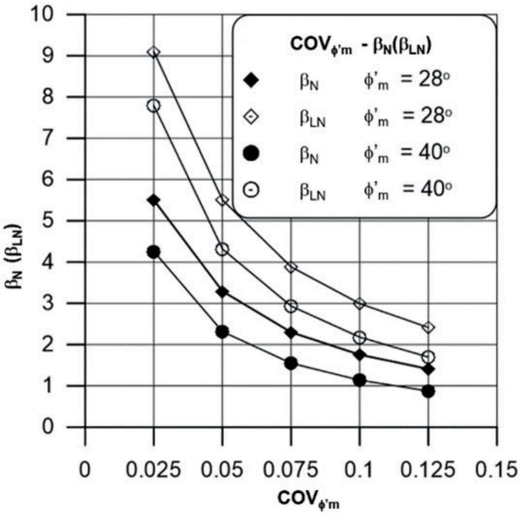

Fig. 3 Reliability index β values as the function of the coefficient of variation for the soil internal friction angle

the value range of the reliability index β for different values of soil internal friction angle coefficient of variation, considering the ϕ'_m = 28° and 40°.

In general, the normal distribution of safety factors yields more conservative values, that is, it provides smaller values of the reliability index β, i.e., a greater failure occurrence probability, for the same observed value of the correlation coefficient. This relation is expected, considering some earlier observations [25].

Likewise, there is an apparent decreasing trend of the values of the reliability index for both distributions with the increase of the coefficient of variations. Therefore, to surpass the target values of the reliability index, it is necessary to search for a greater safety margin by the criterion of the relation between calculated resistances and calculated actions. This trend becomes even more apparent for larger soil internal friction angles. In the observed distribution of the COVϕ'_m, a larger soil internal friction angle results in more conservative (lower) values β, and larger p_f. For example, COVϕ'_m = 0.1 in a normal distribution equals $\beta_{\phi'=28}$ = 1.756 and $\beta_{\phi'=40}$ = 1.138. The explanation for this trend can be found in the relation of relative increment increases FSEV (expected factor of safety), σ_F i V_F (standard deviation and coefficient of variation for FS_{EV}) for normal and lognormal distribution FS_{EV}, given by the following equations:

$$\beta_N = \frac{FS_{EV} - 1}{\sigma_F}, \tag{14}$$

$$\beta_{LN} = \frac{\ln\left(\frac{FS_{EV}}{\sqrt{1+\left(COV_F\right)^2}}\right)}{\sqrt{\ln[1+\left(COV_F\right)^2]}}. \tag{15}$$

Based on the values outlined in Table 4, there is no single relation between Overdesign factor $ODF = R_d/E_d$ and the equivalent overall factor of safety OFS. The deviation occurs due to two reasons.

The first deviation occurs within the given value for the soil internal friction angle, for different standard deviations. For ϕ' = 28°, the value range is within OFS = 2.714 (COVϕ'_m = 0.025) – 2.649 (COVϕ'_m = 0.125), with the mean value $OFS_{m\phi'28}$ = 2.68, standard deviation $\sigma_{\phi'28}$ = 0.024, and the coefficient of variation COV$_{\phi 28}$ = 0.00895. For ϕ'_m = 40°,

Table 4 The results of the analysis of the values for the soil internal friction angle ϕ'_m = 28° and 40° for the selected range of standard deviations

ϕ_m	COVϕ'_m	ϕ'_k	ϕ'_d	B	OFS	F_{EV}	σ_{FSEV}	COV$_{FEV}$	β_N	β_{LN}
	0.025	27.65	22.739	2.340	2.714	3.050	0.373	0.122	5.502	9.101
	0.050	27.30	22.436	2.410	2.690	3.171	0.659	0.208	3.293	5.506
28.0	0.075	26.95	22.134	2.490	2.676	3.310	1.005	0.304	2.299	3.883
	0.100	26.60	21.832	2.580	2.672	3.467	1.405	0.405	1.756	2.994
	0.125	26.25	21.530	2.660	2.649	3.608	1.850	0.513	1.410	2.414
	0.025	39.5	33.404	0.805	3.401	3.933	0.689	0.175	4.259	7.793
	0.050	39.0	32.936	0.844	3.365	4.203	1.384	0.329	2.315	4.315
40.0	0.075	38.5	32.471	0.885	3.332	4.492	2.254	0.502	1.549	2.934
	0.100	38.0	32.007	0.927	3.296	4.796	3.336	0.696	1.138	2.182
	0.125	37.5	31.544	0.971	3.263	5.122	4.702	0.918	0.877	1.698

OFS = 3.401 ($COV_{\phi'm}$ = 0.025) – 3.263 ($COV_{\phi'm}$ = 0.125), with the mean value $OFS_{m\phi40}$ = 3.331, standard deviation $\sigma_{\phi28}$ = 0.054 and the coefficient of variation $COV_{\phi'28}$ = 0.0163. Therefore, the size of deviation from the given safety margin according to Eurocode 7, i.e., the deviation in the values of the global factor of safety, is within ΔOFS = 0.9 – 1.165%. The second deviation occurs between mutually different values of the soil internal friction angle. The overall factor of safety ratio, in regard to the mean values of the for the analyzed soil internal friction angles, equals ΔGFS = $OFS_{m\phi40}$ /$OFS_{m\phi28}$ = 3.401/2.714 ≈ 1.25. A more significant deviation is apparent, that is an inability to consistently maintain the equivalent values of the to-date criterion for achieving the safety margin.

Existing analyses were based on the given ODF, i.e., levelling the design action and design resistance. According to Eurocode 7, this procedure ensures reaching of the prescribed safety margin.

The reliability index target value, i.e., failure probability, can be defined as the limit value of the coefficient of variation within the observed calculation method. Generally, by increasing the coefficient of variation, the probability of failure occurrence also increases. When their limit value is determined, i.e., when further increases of the coefficient of variation enter into unacceptably high probability of failure occurrence, it becomes necessary to correct the *ODF*. This can be done by reviewing the value of the PSF on material in the case of DA3 and/or on external unfavorable action. Another possibility is to increase the foundation size, and consequently increase the equivalent OFS.

In the opposite case, when the reliability index values are higher than required, it can be concluded that the achieved safety margin is higher than necessary. Therefore, more rational solutions are possible, such as reduction of the required foundation width, along with maintaining the same external action.

Fig. 4 shows the results of analyses of the required foundation widths B, from the condition of achieving the reliability index limit value β = 3.1. The figure indicates the relation between the coefficient of variation of the soil internal friction angle and the relative ratio of the required foundation width and the required foundation width for the case of ODF = 1.

A trend of increasing the required width with the increase of the coefficient of variation values is evident, with pronounced values for larger soil internal friction angles. Further, substantial deviations for a normal distribution of the factor of safety are also apparent. Even at lower values of the coefficient of variation $COV_{\phi'm}$ > 0.05 the required foundation width increases significantly, especially with larger values of soil internal friction angles. On the other hand, this is not the case with a lognormal distribution of the factor of safety, where the observed increase was not as substantial. The approximate limit value of the coefficient of variation is $COV_{\phi'm}$ = 0.075. For the lower value $COV_{\phi'm}$ < 0.075 the obtained solutions are conservative, whereas in cases of $COV_{\phi'm}$ > 0.075 the solutions involved are excessively high from the aspect of failure occurrence.

Fig. 5 shows the relation between the equivalent *ODF*s and coefficient of variation of the soil internal friction angle, from the condition of achieving the prescribed value β = 3.1. Similar to Fig. 4, there is an evident trend of *ODF*

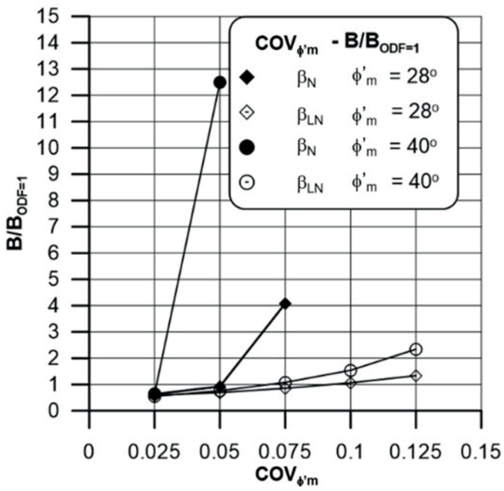

Fig. 4 The relation between relative width *B* and the coefficient of variation of the soil internal friction angle, from the condition of achieving the limit value of the reliability index β = 3.1

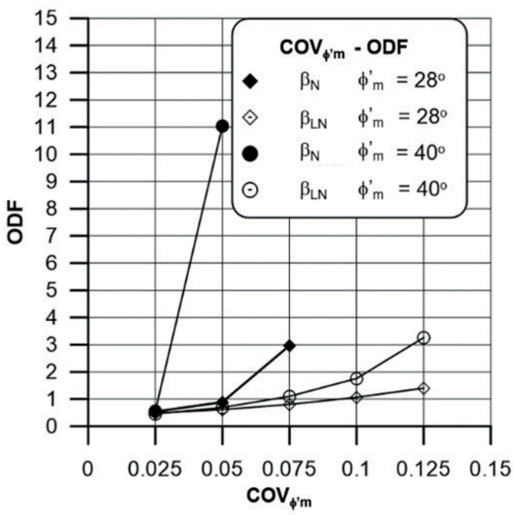

Fig. 5 The relation between the required *ODF* and the coefficient of variability of the soil internal friction angle, from the condition of reaching the limit value of the reliability index β = 3.1

increase with the increase of the coefficient of variation and the size of the soil internal friction angle. As in the previous case, normal distribution shows a significant deviation even at lower values of the coefficient of variation of the soil internal friction angle. On the contrary, the deviations are not as significant when lognormal distribution is observed. It can be concluded that, for specific case of a centrically loaded foundation on homogenous and isotropic soil, there is a combination of PSFs on permanent and variable action and on the parameter of soil shear strength or resistance, calibrated to the value of the coefficient of variation of the soil internal friction angle $COV\phi'_m = 0.075$. For coefficient of variations lower than 0.075, the deviations according to the given criterion start approximately at *ODF* = 0.5. In the opposite case, when $COV\phi'_m > 0.075$, the required *ODF* values are significantly higher, especially for larger soil internal friction angles ($\phi = 40°$, *ODF* > 3).

Fig. 6 shows the values of the equivalent *OFS* for a lognormal distribution, from the condition of reaching the prescribed value of the reliability index $\beta = 3.1$. Here, the *ODF* values from Fig. 5 merge into the required values of the overall factors of safety *OFS*. For the limit value of $COV\phi'_m = 0.075$, *OFS*s are in the expected range of cca 2–3.5. For $COV\phi'_m < 0.075$ values, absolute values reduce with their range, while the opposite applies for $COV\phi'_m > 0.075$. It becomes apparent that, with the increase in the coefficient of variation of soil internal friction angle, the equivalent value of the *OFS* significantly increases, i.e., much more conservative solution is necessary.

From an engineering aspect of optimal design, it is necessary to identify the dominant deviation. If it is the soil with its dominantly inherent deviations, more conservative solutions will be required for reaching greater values of the coefficient of variation. If, on the other hand, the deviation sources are a consequence of the measurement methods and the final interpretation of obtained values, additional efforts are necessary in the scope of research and procedure corrections, to reduce the final values of the coefficient of variation to the lower possible measure.

This approach enables direct measurement of deviations in relation to the prescribed method of calculation of the limit analysis according to Eurocode 7. *ODF*, but also *OFS* are now changing directly in function of the observed statistical features. For example, in contrast to [24] where the given connections are: β – COV and, *ODF* – *OFS*, int the paper the interdependence of failure probability with prescribed safety margins according to EC7 (*ODF*) and classical approach is shown.

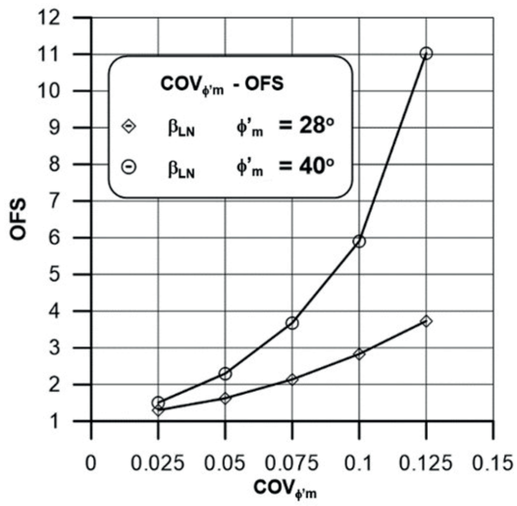

Fig. 6 The relation between equivalent *OFS* and the coefficient of variability of the soil internal friction angle, from the condition reaching the limit value of the reliability index $\beta = 3.1$ for a lognormal distribution of the factor of safety

6 Conclusions

An algorithm is developed for the optimization of the required safety margin, for the case of a centrically loaded spread foundation on coarse-grained homogenous and isotropic soil with no groundwater. An analysis is conducted for the range of standard deviations of the soil internal friction angle. A more comprehensive analysis required parallel calculations for the range of all variables (soil and concrete dead weights, permanent and variable action).

The obtained safety margin is quantified according to the current calculation approach DA3 for the limit state of soil failure beneath a spread foundation. The obtained values of the safety margins significantly depend in the estimated distribution of variables, i.e., the adopted value of standard deviation (coefficient of variation). The selected values of PSFs for action and material (resistance) do not guarantee a consistent value of the achieved level of safety. The prescribed safety margin will not be consistent within the analysis limit state for the unique design approach. In this case it will be larger for smaller values of the soil internal friction angle, as well as for the corresponding increasing values of the angle's coefficient of variation. A similar principle applies to other design approaches within Eurocode 7.

There is an approximate limit value of the coefficient of variation of the dominant parameter, in this case the soil internal friction angle. For coefficient of variation values which are lower than limit values, the solutions proposed in Eurocode 7 are too conservative. In the opposite case, when the values are larger than limit values, the obtained solutions are not appropriate from the aspect of failure probability occurrence. The greater deviation in input values, the more conservative the required solution must be. A greater deviation occurs with larger soil internal friction angles, due to the non-linear nature of the problem of limit state soil failure in the given surface. Small changes of the soil internal friction angle in the environment of its larger values, provide greater relative dispersions of the corresponding values, which leads to the occurrence of greater soil failure probability.

Based on the obtained results, it is necessary to correct the required ODF values, which will differ from individual values, as the function of the coefficient of variation of specific dominant parameters. Another possibility is maintaining the selected value of the ODF, but with the modification of PSFs for unfavorable permanent and variable actions, and respectively for the material and the resistance.

It would be necessary to develop the equivalent algorithms for analyzing limit states for a whole range of tasks in geotechnical engineering. This research would investigate reciprocal influences of change of the strength parameter values on resistance, but also on external unfavorable and favorable actions. This will enable a mutual parallel analysis of limit values of the coefficient of variation of specific parameters, but also the range of the required equivalent *ODF*. The analysis presented in the paper which utilize Taylor series method for the calculation of the reliability index, as well the other future analyses, should be conducted using other calculation procedures such as numeric integration, Monte Carlo, FORM and other FOSM methods.

The paper shows the quantification of safety margins in the mutual correlation of statistical characteristics, classical calculation method, and regulations according to EC7. Numerical values for the analyzed cases are explicitly presented, which allow engineers a direct insight into the relationships of the observed quantities. Engineers will thus be familiar with the degree of change of the default safety margin depending on the relevant statistical features.

It has been shown that relatively simple methods and procedures can lead to quality and relevant conclusions. In engineering practice, presented patterns and conclusions can be used on the whole spectrum of engineering tasks. In this way, the engineer will be familiar with the actual safety margins in the function of statistical characteristics. At the same time, the implementation of the proposed procedures can contribute to the optimization of structures, which will have a uniform safety margin.

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