Case of Ultimate Limit State Design and Eurocode 7-1

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ABSTRACT

The history of the Limit State Design (LSD) in geotechnics is rather long. The first attempt to implement the semi-probabilistic design method in geotechnics was published probably by Brinch Hansen in 1953. This theory was implemented formally in Czech practice in 1966 but it was opposed by most professionals. The theory was contrary to the former successful Safety Factor Design and objections were targeted especially against the Ultimate Limit State Design (ULSD). The development of Eurocode (EC) 7-1 began at the end of 1970 and met with similar opposition. However, the same problem was solved in a different way with the Czech standardization which had implemented the LSD with another definition of characteristic input values. The European standardization retained the classical LSD including geotechnical ULSD although design problems were not solved satisfactorily. Now, EC 7-1 has come into force in the European Union (also in the Czech Republic) and it is in a period of calibration. The most serious problem is the ULSD application for geotechnical *(non-linear)* tasks using derived material inputs which appear to be very inadequate. It appears to be it necessary to check the base of the ULSD theory. The paper presents results and conclusions of the problem analyses.

INTRODUCTION

An idea of the Limit State Design (LSD) in geotechnics first appeared about 60 years ago. Brinch Hansen published in 1953 what was probably the first concept implementing the semi-probabilistic design theory and design method in geotechnics according to a general concept of structure theory. The general concept of LSD is composed of three groups of limit states of a given structure: Ultimate Limit States, Serviceability Limit States and Durability Limit States. A structure had to be designed according all relevant limit states and the most unfavorable one is decisive. Designs are not based on the most probable input data but they apply small probable unfavorable values, such as, “design values.” An approach to design value derivation is rather complicated. Firstly, it has to be a derived “characteristic value” which, secondly, is divided or multiplied by one or more partial factors to be an obtained design value. The theory and codes distinguish a higher number of partial factors. It is obvious the concept was created for linear tasks of elastic structure states.

A development of the Eurocode 7, Geotechnical Design - Part 1: General Rules (EC 7-1) began at the end of 1970 and its design concept was founded on the LSD theory from the very beginning. The developed code encountered numerous difficulties and problems for which solutions have been found, Even a problem with the Ultimate Limit State Design (ULSD) was solved at the beginning but it has not been worked out as yet. The European standardization (EUROCODES) kept LSD, including ULSD in the code for all geotechnical tasks and it has done so thus far although the design problem of the ultimate states has not been solved satisfactorily.

However, the same problem in the Czech standardization has been solved in another way. The theory was implemented formally in Czech geotechnical practice in 1966 but it was not accepted by most professionals. The theory was in opposition to the former successful and simpler Safety Factor Design. Both at that time and currently objections have been focused in particular against the ULSD and its statistical definition of material characteristic values and definition of material design values. Consequently, Czech standardization has implemented LSD with one very substantial exception only for soils (geotechnics): soil property characteristic values have been considered as cautious statistical mean values. Adequate standards have come into force for shallow and pile foundations and earth pressure only, but LSD has been used for foundation design only, not in other geotechnical tasks.

The last draft of EC 7-1 which is still in force (e.g., in the Czech Republic as ČSN EN 1997-1, 2009), presents four permitted derivations of the characteristic values in Section 2 “Basis of Geotechnical Design,” par. 2.4.5.2, “Characteristic values of geotechnical parameters” according to following definitions (three for ULSD, one for Serviceability Limit State Design (SLSD):

a) Cautious estimate of the value affecting the occurrence of the limit state (according to clause (2)) - for the *Ultimate* Limit States
b) Such value that the calculated probability of a worse value governing the occurrence of the limit state under consideration is not greater than 5 % (according to clause (11)) - original “statistical definition” for the *Ultimate* Limit States.
c) Value selected as a very cautious value using standard tables of characteristic values related to soil investigation parameters (according to clause (12)) - for the *Ultimate* Limit States
d) Cautious estimate of mean value (according to clause (7)) - for the *Serviceability* Limit States.

The statistical definition (ad b) is original for ULSD. The other two definitions (ad a) and c)) were completed later after discussions and objections by some national committees. In effect, these two later definitions leave the whole risk and responsibility (for both danger and efficiency) on the designers.

The second basic conceptual procedure is derivation of design values from characteristic ones using partial factors. EC 7-1 distinguishes a high number (30) of partial factors for different parameters, including the following: actions, permanent actions, permanent destabilising actions, permanent stabilizing actions, *soil parameters (material properties)*, soil parameters in stratum, soil parameters also accounting for model uncertainties, variable actions, unconfined strength, resistance, uncertainty in resistance models, earth resistance, sliding resistance, bearing resistance, shaft resistance of piles, uncertainties in modelling the effects of actions, destabilizing actions causing hydraulic failures, stabilizing actions against hydraulic failures, tensile resistance of piles, total resistance of piles, permanent and temporary anchorage. The most problematic of the partial factors are soil material factors for the shear strength (effective cohesion and angle of shearing resistance, and undrained shear strength). The problems with derived design values have led to a situation where the code requires up to three approaches of input data derivation and model calculations for some geotechnical tasks (e.g., slope stability).

Particularly in geotechnics, correctness of numerical models and calculations depend mostly on input data, i.e., according to LSD on design values. The theory application has brought in a geotechnical design according to ULSD hard problems. These problems have been discussed and solved in the European Union (EU) and in the East Central Europe long decades since 80s (in Czech from 60s) without a satisfactorily result. Some research in the 1990s and after 2000 [Koudelka 2002, 2003] has shown that the problems were caused by applying both of the value definitions (characteristic and design values) and for soil material properties, especially for shear strength. Also a draft of the Japanese geotechnical standard [Fukui et al. 2003] contains similar results. The paper deals with just the problem of design material properties which is the matter of the problems of the theory and EC 7-1.

EC 7-1 has now come into force in the European Union (also in the Czech Republic) and is in a period of calibration. A number of reasons exist for a verification of the ULSD theory. The most serious problem is an ULSD application of the derived soil design values for geotechnical non-linear tasks which appears to be very inadequate (especially in the statistical definition of material characteristic values and partial material factors). Also, in addition to others, EC 7-1 does not solve design using such advanced numerical methods as FEM and BEM. It appears to be necessary to check the base of the ULSD theory and to turn attention to reliability-based design [e.g. Akbas-Kulhawy 2011].

RESEARCH OF ULTIMATE LIMIT STATE THEORY

Long-term research on the theory of the ULSD has been in progress in the EU simultaneously with a draft and acceptation process of EC 7-1. The research targets four basic geotechnical tasks: shallow and pile foundations, slope stability and earth pressure. The first analyses of earth pressure research showed the problem of matter was not entirely with the LSD theory but just in a theory of earth pressure itself. As a result, this problem was solved separately, being supported by special grant projects and applying physical and numerical experiments. Some results on earth pressure research are presented (Koudelka 2000, Koudelka p./Koudelka T. 2004a, 2004b] and also in a second paper at the Conference [Koudelka 2012].

Research of the other tasks has been in steady progress. The slope stability problem was analyzed first and practically for total parameter scales using theory of model similarity. The slope stability problem was solved by a wide analysis of the three Code approaches and a classical design according to safety factor [Koudelka P. 2002]. Results of the analysis made other analyses practically unnecessary.

An analysis of designs of shallow foundations calculated according to ultimate limit state designs and respective models of EC 7-1 and ČSN 73 1001 was carried out as the second research step, also in wide parameters scales [Koudelka 2007 – compare also to Scarpelli-Fruzzetti 2005]. Results were compared not only between both models but also to tabular values of the Czech standard ČSN 73 1001. Results of the analysis led to simplifying adjustments of the standard or the code.

The third analysis according to the Czech standards ČSN 73 1002 [1987] and ČSN 73 1004 [1981] was related to pile foundations applying a standard numerical model [Koudelka 2008]. The code presents no numerical model for calculations and it targets on pile load tests only

ULTIMATE LIMIT STATE DESIGN IN GEOTECHNICS

Non-linear behavior of soil and rock masses in the Ultimate Limit States (ULS) has different manifestations in various geotechnical structures and systems. In its purest form, it probably appears in slopes and embankments in which the soil mass is usually not combined with man-made structures and the stability is highly sensitive to the changes of properties. A striking example of this is that it is possible to show the influence of non-linear soil mass behavior on slope stability when using partial safety factors of materials for ground properties γ_m and the statistical definition of the characteristic value according to EC 7-1.

SLOPE STABILITY

Let us consider simple slopes of the given incline *1:n* with an angle α $(n = \tan\alpha)$ in homogenous soil masses with arbitrary combinations of statistically variable material properties (Fig. 1). There is no ground water in soil masses. Let the design of these slopes be based on average values of material properties and a classical Swedish model according to the following equation $F = (N*\tan\phi + C) / T$ where F is the stability factor on an arbitrary cylindrical slip surface, N and T are integrals of normal and shear components, respectively, of soil weight acting on the given slip surface.

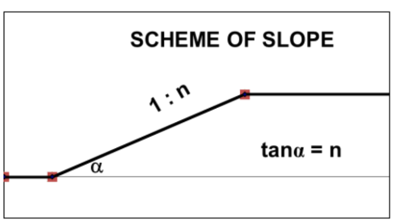

Fig. 1. Scheme of analyzed slopes and geometry of the minimalized functional.

The safety factor designs of the slopes are performed according to minimized safety factor F_o on a critical slip surface of all possible cylindrical slip surfaces (both toe and deep ones). This factor has to be minimally equal to the required standard safety factor F_s. Then an analysis is based on the relation $F_{os} = F_s$ where notation F_{os} expresses the minimize safety factor according to the standard value. If there is also a deep critical slip surface, then the analysis is the lower of the two values existing on the toe critical surface and the deep critical surface. The index *s* denominates the values appertaining to the soil mass with the required standard safety factor F_s.

For the ULSD design, let us examine the above geometrically identical homogeneous slopes with the properties changed in accordance with the provisions of EC 7-1. The characteristic shear strength values are defined according to the statistical method. The statistical variables are considered in accordance with Lumb´s results [1972] of an extensive inter-laboratory study of shear strength of soils (Ottawa sand, residual soils) with the differentiation of the tests of effective shear strength (D) and total shear strength (UU). According to this study, the variability of materials can be characterized by the standard deviation, specified as $v_c = 0.15675$ and $v_{\tan\Phi} = 0.192$, for the (D - effective) tests. The statistical variables of unit weight are considered after the Czech soils database; the average of the classes of the groups F + S (2005-160 samples) $v_\gamma = 0.044$. The statistical values of material properties were calculated according to the distribution function of the standard distribution of Pearson III type with the inclination of $\alpha=0$.

The design value of shear strength can be obtained from the characteristic value by the application/dividing of partial factors for soil parameters (material properties) γ_M, i.e. $\gamma_{\phi'}$ for the angle of shearing resistance $(\tan\phi')$ and cohesion and $\gamma_{c'}$ for effective cohesion. The density values are considered with mean values. The determination of other toe and deep critical slip surfaces in materially changed masses yields the safety factors F_{od} which have to be equal to a respective value of partial factor for sliding resistance $\gamma_{R;h}$ instead of the safety factor. The index *d* denominates the design values and values appertaining to the soil mass with the code partial factor for sliding resistance $\gamma_{R;h}$ required.

The reduction of the number of variables and a substantial limitation of the scope of the analysis can be achieved by the similarity theory. The similarity of the conventional slope model depends on the Hamilton´s similarity coefficient $\pi = c/(\gamma*h)$ and Janbu´s one $\lambda = c/(\gamma*h*\tan\phi)$ [1954] where *c* is cohesion, γ unit weight, *h* height of the slope and $\tan\phi$ shearing resistance. It can be proved analytically [Koudelka-Procházka 2001] that Hamilton´s coefficient π influences a critical safety factor value on the most dangerous slip surface according to equation (1)

$$F_o = F_{01} * \pi \qquad (1)$$

where F_o is the minimal safety factor on the most critical slip surfaces, F_{01} is the number of minimal stability for the given value of Janbu´s coefficient λ [Koudelka-Procházka 2001]. The analysis is concerned with slope declination designs beside others both according to safety factor design (F_s=1.5) using the mean properties of the soil and according to the ULSD of EC 7-1. Also analyzed are three alternatives of EC 7-1 drafts: an original in 1994 and two approaches of the final draft in 2004, i.e., Approach 2 and Approach 3 (Approach 1 is inappropriate à priori). Slope declinations are calculated in these four alternatives, respectively, applying one of following equations:

$$F_{os} = F_s \quad = F_{01s} * \pi_s \qquad \text{for SFD} \tag{2}$$
$$F_{od} = \gamma_{R;h} \quad = F_{01d} * \pi_d \qquad \text{for ULSD} \tag{3}$$

Results of calculations are carried out in diagrams of slope declinations depending upon Janbu´s similarity coefficient λ [Koudelka/Procházka 2001] and different values of angle of shearing resistance ϕ. Diagrams in Fig. 2 and Fig. 3 compare all possible combinations of the values of unit weight, cohesion and slope height in their whole scales and the angle value of shearing resistance of 20° and 40° by means of the scale of Janbu´s similarity coefficient λ applying equations from the relations (2), (3) derived

$$F_{01s} = \ F_s \ / \pi_s \qquad \text{for SFD} \tag{4}$$
$$F_{01d} = \gamma_{R;h} / \pi_d \qquad \text{for ULSD} \tag{5}$$

The respective slope declinations were found in the minimization solution of a functional of the model by Fig. 1 [Koudelka-Procházka 2001] for the calculated numbers of minimal stability F_{01} and relevant values of Janbu´s similarity coefficient λ.

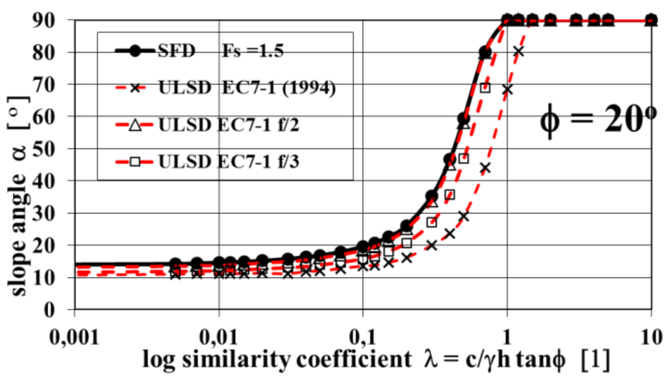

Fig. 2: Comparison of slope designs by different ULSD code alternatives to safety factor design of $F_s = 1.5$ Arbitrary combination of mean soil properties with angle of shearing resistance $\phi = 20°$ is included in Janbu´s similarity coefficient λ expressing their mean values

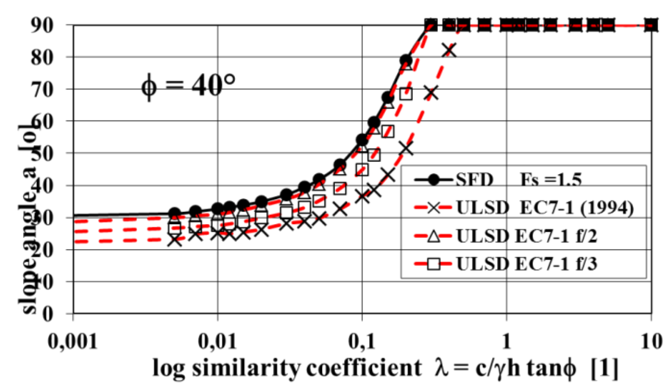

Fig.3: Comparison of slope designs by different ULSD code alternatives to safety factor design of $F_s = 1.5$ Arbitrary combination of mean soil properties with angle of shearing resistance $\phi = 40°$ is included in Janbu´s similarity coefficient λ expressing their mean values

The analysis contains a number of the diagrams, more than in Fig. 2 or Fig. 3 but histories of slope declinations are similar through the whole scale of angle of shearing resistance ϕ. The diagrams show the behavior of curves of design declinations α very clearly. It can be observed that the course of the SFD line along the whole interval λ <0.001; ∞> is markedly higher than courses of all ULSD EC 7-1 lines. This fact expresses the slope declinations regarding stability on the adequate critical slip surfaces designed to be more effective than the designs according to any ULS design and according to EC 7-1. All lines (designs) in all graphs end in a vertical slope for more or less cohesive soils, i.e., approximately in the interval λ <0.3; ∞>.

Graphs in Figs. 2 and 3 give general views of designs according to EC 7-1 and the safety factor theory for the usually prescribed $F_s = 1.5$ throughout the whole practical range of soils. Using the theory of similarity and interpolation, it is possible to find a solution to an arbitrary example of any simple homogeneous slope. The solution is expressed in the form of slope angle α. The analysis has shown generally less effectiveness of the ULSD approaches compared to the proved safety factor design long practice.

The comparison could be even less unfavorable in practice if the variability of soil properties would be higher than the variability used in Lumb´s wide study [Lumb 1972]. Lumb´s results of an extensive inter-laboratory study are of shear strength of soils (Ottawa sand, residual soils) with the differentiation of the tests of effective shear strength (D) and total shear strength (UU). According to this study, the variability of materials can be characterized by the standard deviation, specified as $v_c = 0.15675$ and $v_{tan\Phi} = 0.192$ for the (D) tests, and as $v_c = 0.2127$ and $v_{tan\Phi} = 0.289$ for (UU) tests. These variability values are rather low and practical variability at sites would be probably higher especially of the cohesion one.

SHALOW FOUNDATIONS

The foundation design of EC 7-1 is based on the same concept and value definitions as are described above but other partial factors for soil properties are presented in Annex A, Chapter 3 and an informative numerical model and procedure is given in Annex D. The code presents no table of allowable or recommended stress values for subsoil under shallow foundations even characteristic or design ones.

The design procedures of shallow foundations, according to EC 7-1 and the Czech standard ČSN 73 1001 (1987 - hereinafter ČSN) are somewhat similar but not the same. The analysis compared both procedures and the detailed numerical models were presented earlier [Koudelka 2006, 2007]. Original symbols and subscripts are used for easier [or, clearer?] distinction. Geometrical relations are shown in Figure 4 ($\delta = 0°$).

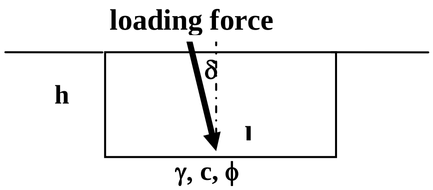

Fig. 4. Scheme of shallow foundation model

A previously presented similarity solution of the bearing capacity of shallow foundations [Koudelka 2006] is used according to EC 7-1. The similarity solution forms the basis of the comparative analysis base for simpler numerical analyzing. The range of the analysis is given by the ϕ *<5°; 45°>* and λ *<0; 5>* scales of the average values (results for 45° are not presented). The range should be sufficiently wide to involve all usual soils.

A correct comparative analysis needs the comparable values of the design stress of the bearing resistance R_d due to different values of the partial resistance factors $\gamma_{R:v}$ used for the design criterion. Hence, they are defined as the comparable design stress of the bearing resistance R_d' and the similarity functional P_d as follows:

$$R_d' = \gamma h\,[F_c + F_d + F_b\,] / \gamma_{R:v} \qquad (6)$$

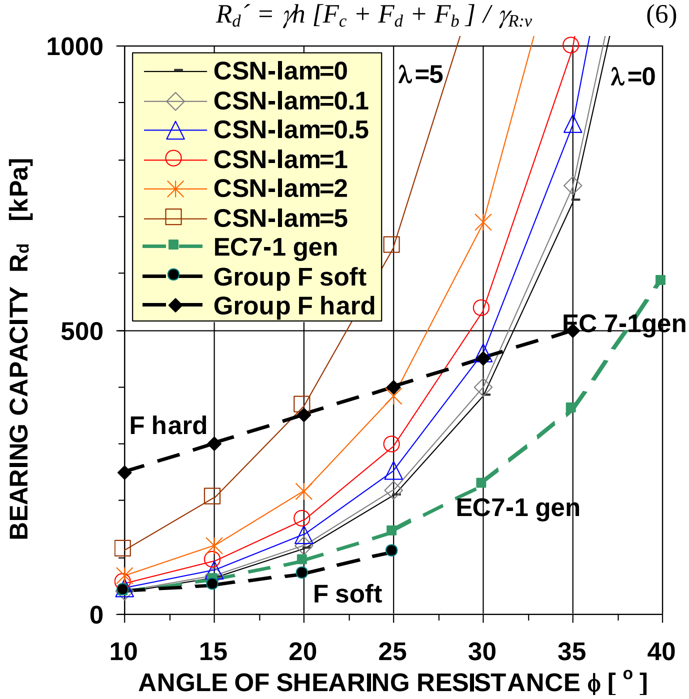

Figure 4a. Completed detail of the whole diagram comparing the results of the ČSN design procedure (full color lines) to the design bearing capacity after ČSN 73 1001, Table 15 for the soil group F (fine grained soils - black dashed lines - soft and hard). Green dashed line shows bearing capacity of a general soil with mean properties (database of the author Institute) using the EC 7-1 procedure.

$$P_d \;=\; [F_c + F_d + F_b\,] / \gamma_{R:v} \qquad (7)$$

where γ is soil unit weight, F_c , F_d , F_b - dimensionless functionals expressing influences of *cohesion c, foundation depth h* and *foundation wide b*, respectively. Even the code and standard constitutive equations and the functionals depend on values: $\beta = b/L$ and $\eta = h/b$ *and L* is foundation length. The analysis presents results for $\beta = \eta = 1$.

The analysis makes it possible to investigate not only γ, ϕ, c after their scales, but also after the scales of the geometrical parameters β, η. For the purposes of analysis presentation, the paper makes use of the cube foundation with an embedment depth of 1.0 m. The unit weight of soil masses usually does not vary too much and is considered constant at $\gamma = 20\ kNm^{-3}$.

The solid lines in the graphs mark the histories of bearing capacity for the constant values of Janbu´s similarity coefficient λ in dependence on the angle of shearing resistance ϕ. The value $\lambda=0$ is significant for non-cohesive soils, the value $\lambda=5$ is significant for cohesive soils.

The derivation of the geotechnical design parameters for the EC 7-1 analysis differs from that for ČSN (Koudelka 2007). The EC 7-1 statistical method requires statistical data of test sets. The analysis considered the data of two database sets. Firstly, for shear strength (ϕ, c), the statistical results of

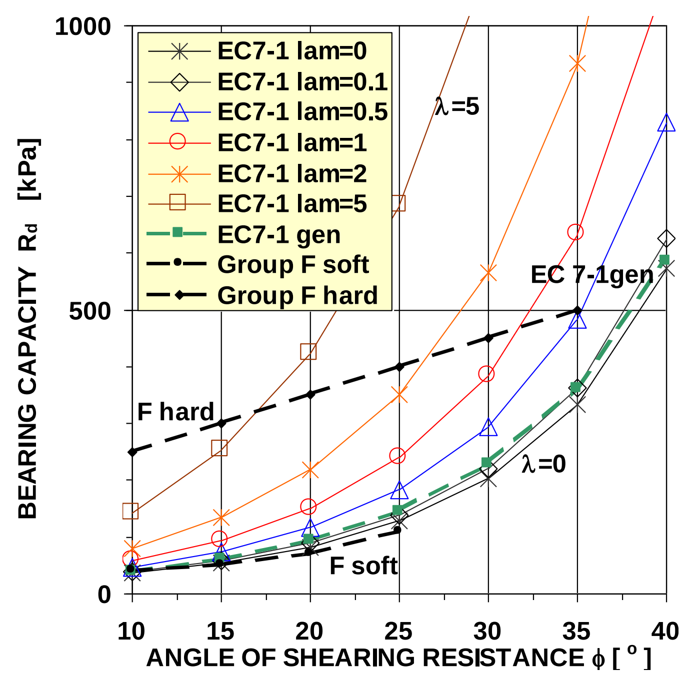

Figure 4b. Completed detail of the whole diagram comparing the results of the EC 7-1 design procedure (full color lines) to the design bearing capacity after ČSN 73 1001, Table 15 for the soil group F (fine grained soils - black dashed lines - soft and hard). Green dashed line shows bearing capacity of a general soil with mean properties (database of the author Institute) using the EC 7-1 procedure.

residual soils after Lumb´s wide inter-laboratory study [1972] were used. Secondly, the variability coefficients of *γ, ϕ, c* were calculated to obtain a general expression of soil variability of groups F (fine granular soils) and S (sandy soils) from a special database of 160 samples. The general variation coefficients were calculated statistically for each of 8, respectively, 5 group classes and the resulting values are the average of variation coefficients of the respective classes.

The thick dashed green lines marked "EC 7-1gen" in all chapter figures show the bearing capacity of the foundation on

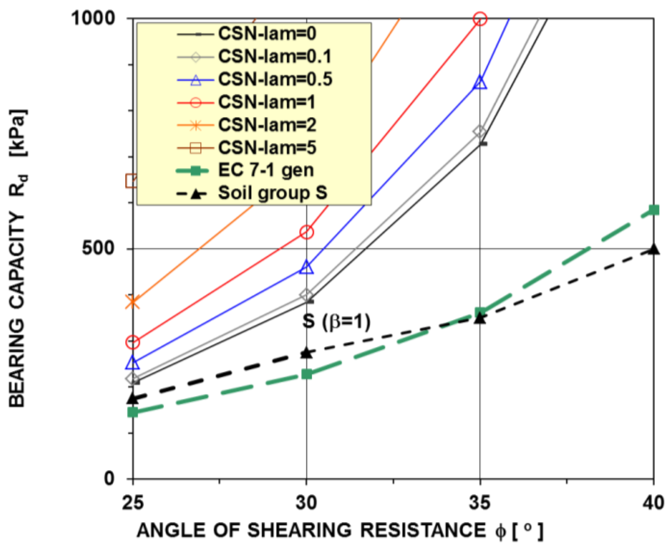

Fig. 5a. Completed detail of whole diagram comparing the results of the ČSN design procedure with the bearing capacity design after ČSN, Tab.15 for soil group S (sandy soils – lower full black line). Green dashed line shows bearing capacity of a general soil with mean properties (database of the author Institute) using the EC 7-1 procedure.

subsoil with the properties after the aforementioned physical property database of Czech soils. After derivation according to the EC 7-1 procedure, the sample variation of these soils has led to only the zero design value of cohesion.

The other shorter dashed black lines marked "F soft/hard" or "S (β=1)" in Figs. 4a and 4b, or Figs. 5a and 5b show in detail the bearing capacity designed according to the tables in ČSN 73 1001 as the second possible way of bearing capacity determination in the Czech geotechnical practice. The lines marked "F soft" define the bearing capacity of fine granular soils of group classes F1-F8 with *soft consistence*; the lines marked "F hard" define the bearing capacity of fine granular soils of group classes F1-F8 with *hard consistence*. The area between both dashed lines characterizes the range of soil consistence influence on the bearing capacity of the shallow foundation. The lines marked "S (β=1)" in Figs. 5a and 5b characterize the bearing capacity of sandy soils of group classes S1-S5.

The long-term experience in the Czech Republic with the ČSN design procedure, it has been recognized that the design procedure after the standard par.86-89 gives values too high for higher shear strength values. In view of this, the practice has adopted the general use of the stress values of the bearing capacity design after Tables 15 and 16 of ČSN 73 1001 and the design procedure has been used rather exceptionally. It has been generally recommended to use the procedure with great caution. On the contrary, the tabled design stress values have been used successfully for a long time and appear be reliable.

The Czech experience with the EC7-1 design procedure is not

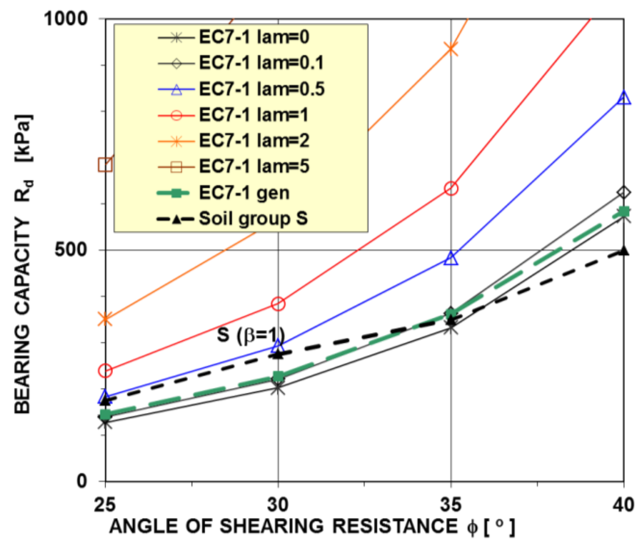

Fig. 5b. Completed detail of whole diagram comparing the results of the EC 7-1 design procedure with the bearing capacity design after ČSN,Tab.15 for soil group S (sandy soils – lower full black line). Green dashed line shows bearing capacity of a general soil with mean properties (database of the author Institute) using the EC 7-1 procedure.

extensive. Some such analyses are known [IWS Dublin 2005: e,g, Bergdahl, Orr, Simpson], but these analyses usually concern some factual case(s) and are not numerous. Their evaluation is important and interesting even though the designs are less optimistic, but this does not support the behavior of the foundation bearing capacity model. Consequently, a cautious access to the EC7-1 design procedure has also been adequate.

The presented analysis proves that the cautious use of the standard/code design procedure has been relevant. The resulting bearing capacities of both procedures exceed the value of 500 kPa from the angle of shearing resistance about ϕ=20° and for higher shear strength values, the excess is many times as high. It can be seen that the ČSN design procedure for higher shear strength gives higher values than the design procedure according to EC7-1. The results of both procedures for the angle of shearing resistance ϕ under 20° are tolerably similar.

If we compare the results of both design procedures with the values of the table design stress in Figs.1,b,c and Figs.2b,c it is possible to find two areas of their correspondence. One area for fine granular soils (group F) is shown by the dashed lines for soils of hard and soft consistencies respectively and the thick dashed line marked "EC7-1gen" for soils in general. The second area for sandy soils (group S) extends from about ϕ=25° up around the thick dashed line marked "EC7-1gen" for soils generally.

Both reliable areas proven by long-term experience appear suitable for exploitation in standardization. The proof of the excessively optimistic part of the designs according to code/standard procedures makes the procedures dubious. From this fact it follows that it is not necessary and effective to calculate the bearing capacity value with the risk of optimistic results. However, the most important fact is that *both analyzed numerical models are based on geometrically dimensionless solutions and that an absolute size of the foundation is not taken into account.*

It is well-known that foundations of larger sizes can be loaded relatively less than smaller foundations. Thus, the design stresses on subsoil under geometrically similar foundations should not be the same. This problem could be solved by the elimination of both design procedures from the code/standard and for usual cases by the use the tabular design values which distinguish the absolute foundation size. Of course, complex and very important cases should be solved by advanced methods and procedures.

PILE FOUNDATIONS

Another analysis related to pile foundations was carried out earlier and presented at a previous Conference (Koudelka 2008). So this paper summarizes basic information and results of the analysis.

The pile design according to EC7-1 does not contain any numerical model and determines that design shall be based on one of the following approaches:

a) Results of static load tests, which have been demonstrated, by means of calculations or otherwise, to be consistent with other relevant experience;
b) Empirical or analytical calculation methods whose validity has been demonstrated by static load tests in comparable situations;
c) Results of dynamic load tests whose validity has been demonstrated by static load tests in comparable situations;
d) Observed performance of a comparable pile foundation, provided that this approach is supported by the results of site investigation and ground testing.

Design values for parameters used in the calculations should in general be accordance with the EC7-1 requirements for geotechnical data, but the results of load tests may also be taken into account in selecting parameter values. The code recommends no tabular values.

The latest Czech standard containing a numerical pile model was ČSN 73 1004. This standard has been superseded by the latest Czech pile standard ČSN 73 1002, "Pile foundations" of April 1, 1989. The standard contains only the tabular "*design*" bearing capacities of the driven and bored piles with regard to their profile and the density I_D or the consistency index I_C and, of course, regarding pile length in the bearing layer(s). No calculation procedure is presented.

The analysis examined the numerical model of bearing capacity according to ČSN 73 0004 for a similar wide range of parameters such as the above mentioned analyses so its results can be compared to the table bearing capacities of ČSN 73 0002.

A general homogeneous mass and vertical axial loading force according to the scheme in Fig. 6 was assumed. The two components of pile bearing capacity (that of toe and pile face shear strength) were calculated separately and the ideal pile bearing capacity was summed like the upper limit of the whole pile bearing capacity. A general analysis of the distribution of both bearing capacity components did not seem adequate and useful.

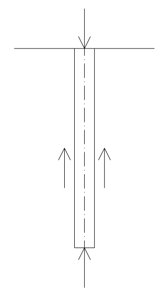

Fig. 6. Scheme of axially loaded pile and bearing capacity components.

Following the analysis, the ultimate pile bearing capacity depends on load distribution between the toe and the shaft face of the pile due to the deformation of the soil mass both under the toe and around the pile and also slightly less to the deformation of the pile itself. An analysis of load distribution required a number of other parameters which led to an extraordinarily large number of possibilities and combinations. From the point of view of the analysis, a simpler definition of the complete pile bearing capacity appeared to be useful and was applied.

The analysis took the position that the EC 7-1 design concept on pile load tests based was better than an analytical calculation in ČSN 73 0004 [Koudelka P. 2008].

CONCLUSIONS FOR ULSD IMPROVEMENT

All three analyses described above show a substantial and deciding influence of the statistical definition of characteristic values and partial property factors at the designs according to

Ultimate Limit States of EC 7-1.

A comparison of Figs. 2 and 3 shows that slopes (angles) designed by the system EC 7-1 are obviously milder (lower) throughout the whole range λ and ϕ, i.e., the whole practical range of soil properties, than designs respecting $F_s = 1.5$ (safety factor theory). The only exception is the vertical slope *(α=90 ^{o}).* It can be observed that SF designs provide vertical slopes for lower values λ than EC 7-1 ULS designs. It signifies that the SF designs approach for vertical slopes are of less cohesive materials than ULS designs according to EC 7-1 which need more cohesive materials for vertical slopes.

The second analysis of designs of shallow foundations calculated according to models of EC 7-1 and ČSN 73 1001 was presented resulting in the conclusion that both calculation procedures were not sufficiently adequate to real subgrades.

The third analysis of designs of pile foundations showed that the former simpler analytical numerical model is not adequate yet and the EC 7-1 design concept aiming on pile tests is more suitable due to present technology.

Summarizing, a recommendation for ULSD improvement of the EC 7-1 is as follows:

1 Substitution of the statistical definition of material characteristic values and partial factors of soil property with a new definition of design values of soil properties such as this: *Design value of a soil property is the cautious mean (most probable) value.*

2 Simultaneous modification of the relevant partial factors for resistance, e.g., the factor for slope resistance can be defined as changeable after a slope inclination (see Fig. 7) and other circumstances.

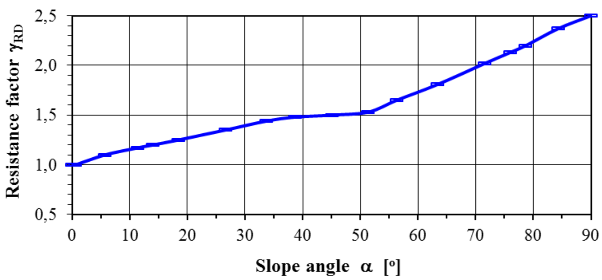

Fig. 7. Dependence of resistance factor for slopes γ_{RD} on inclination

3 Elimination of the numerical model for shallow foundations and the code completion with the design tabular values of stresses on subsoil under the foundation.

4 The pile foundation design concept of EC 7-1 appears suitable and it is possible to recommend it.

EC 7-1 takes in no mention on ULSD using advanced numerical models (FEM, BEM). It appears in the second decade of 21st century this problem should be dealt in.

However, there is also a fourth four major problem of geotechnics, i.e., earth pressure. An informative procedure on the calculation of earth pressure is found in Annex C of EC 7-1 and partial factors for soil parameters in Annex A. The code ULSD problem of earth pressure loading has not been analyzed due to the obsolescence of the theory applied. An independent research of the earth (lateral) pressure theory has been in progress since 1998 and a section on passive pressure during rotation about the top presented at this Conference is found in paper No. 3.15b.

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