Reliability of ULSD Theory in Geotechnics

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ABSTRACT: The article discusses the theory of ultimate limit state design (ULSD) and its consequences. An influence of definitions both characteristic and design values of soil parameters of EN 1997-1 (Code) is analyzed. The article has two basic theoretical aims: a) To demonstrate the incorrectness of ULSD in *geotechnics* and due to it in-effectivity and risk. b) To present a concept of a more plausible and correct design theory which is simpler and in compliance with mathematical principles. A case of slope design is chosen from basic geotechnical problems because slope masses are most sensitive. Slope designs based on the design approaches of the Code are compared with a direct design value definition-based approach. Slope analysis exploits the results of a statistical analysis of an extensive database of soil material properties. The analysis is based on data sets of sandy and fine-grained soils and demonstrates the risk of homogeneity for ULS designs. Another simple geotechnical design concept suitable also for advanced numerical methods is suggested and a procedure example is presented.

KEYWORDS: Design theory reliability, Design value definition, Soil property database, European standard EN 1997-1, Slope design.

1. INTRODUCTION

The most complex and the most discussed problem of the present European Standard EN 1997-1 (Code) is the theory of ultimate limit state design (ULSD) of soil structures. European scientists and governments alike have been studying the limit state design since the establishment of Eurocode7 (EC7) 25 years ago. Several approaches to this problem have been discussed in the recent past; for instance, at the IS EUROCODE 7 - Towards Implementation, London (September-October 1996), XIVth IC SMGE in Hamburg (1998), XIIth EC SMGE Amsterdam (1999), GeoEng 2000 IS of ISSMGE/TC23 Limit State Design in Melbourne, IWS Kamakura (2002), XIIIth EC SMGE Praha 2003 ERTC10 IWS Evaluation of Eurocode7, and other conferences. Since the 1990's, Code drafts and the Code have followed almost improbable and unfavorable material design values that do not considering safety factors (B. Hansen 1953). This theory was also accepted in Eastern Europe under the Soviet Union's constraints in the 1960s (1st ČSN standard on LSD in 1966).

Development of the Code and discussions continued, and the first Code was edited in 2004 (see EN 1997-1) implementing the afore mentioned idea of *almost improbable*, unfavorable material design values for ULSD. Then, the development of the Code continued (ISSMGE Dublin 2005, XVIthIC Amsterdam 2005, XIIIDEC Lublin 2006) and a new Code version design (prEN EN 1997-1:2018) was edited in 2020 without a change of the discussed design value theoretical base of ULSD (*material* design value definition). The very wide discussion not too satisfying practical experiences with ULSD in geotechnics has continued and still continues. The crux of the issue is the lack of a proper definition for the input *material design* values. Some European countries have improved the ultimate limit state of structures in their National Annexes Documents or and other national standards, while other countries have simply accepted the ULSD given in the Code. However, no general agreement on the issue among European countries (e.g. at XVIIIth IC Paris 2013) has been achieved. However, CEN in 2014 initiated the process of revising the Code. The discussion continued on the levels of both the technical committees of CEN/TC250 (TC250) and ISSMGE (TC205, TC304), and the reference (2018). The process of the Code revision and an edition of a new version of EN 1997-1 is not closed till this time (Jan. 2020).

Since then, new definitions have been discussed and analyzed in recent studies. For instance, Schneider (2011) defined the characteristic value as the mean value reduced by a standard error fractile of 5%, while Bolton (2018) defined the design value as the 'worst credible strength', both of which were suggested in the last TC205 conference. These definitions may yield more probable material design values, but not the most probable design values, according to Koudelka's (TC205-2016, TC250-2018, TC250-2019). The reason behind this theoretical speculation surrounding the reliability of soil parameter design values according to the Ultimate Limit State is explained later on in this Article.

The development of the Code has been only partially successful and rather complicated. However, the basic problem of ULSD (advanced numerical methods included) has remained unsolved because a reliable, unified geotechnical design could not be arrived at even after 30 years. The history and development of the Code certainly pique the interest of scientists, and their analysis would be quite useful to engineers. However, such an analysis is not an objective of the Article.

2. DESIGN IN GEOTECHICS

At present, the Code's theory for Ultimate Limit State (ULS) in geotechnics is based on a serious theoretical error. Soils are natural materials and are entirely different from steel or concrete. They have only one relevant and deciding engineering property – *shear strength*. The general ULSD concept is valid for solid structures designed in the *elastic state* of stresses (Figure 1 (a)). Bearing parts in stress histories (pressure, tension) of materials used in engineering structures in the elastic state are *linear*. Hence, in this case, the principle of superposition is valid and different (partial) coefficients can be applied in the design. The ULS design theory in European Standards is correct and valid for artificial materials only (EN 1992 – concrete, EN 1993 – steel, EN 1994 – steel & concrete, EN 1995 – timber, EN 1996 – masonry).

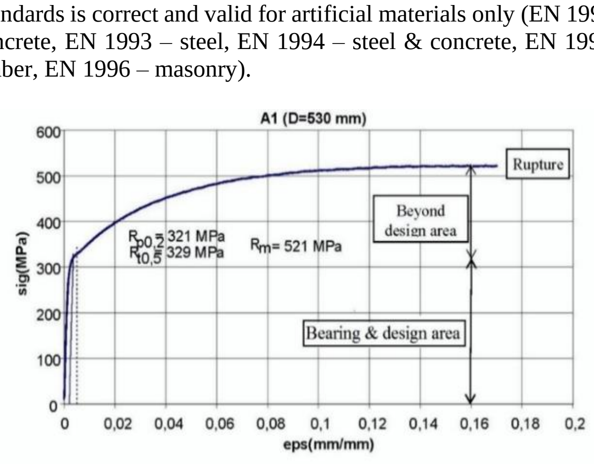

Figure 1 (a) Diagram of traction test of steel with *elastic* design area and *plastic* area beyond design area. The rupture area is not noted.

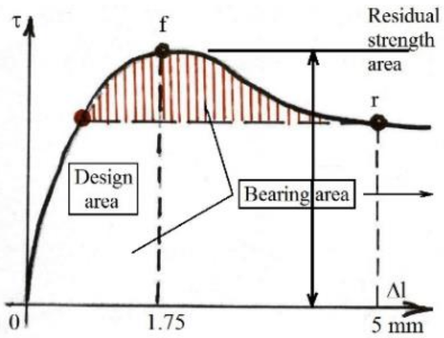

Figure 1 (b) Diagram of shear strength test of soil *without elastic* design area and with *complex plastic* area. Rupture area does not exist and *residual strength area* is practically constant.

On the contrary, soils, i.e. their shear strength, show complex *plastic* behavior. Moreover, this behavior often is affected by various natural influences. Stress histories of soil shear strength are complexly *non-linear*. The histories contain usually more phases: non-linear, softening and residual (Figure 1 (b)). The principle of superposition *(the definition of improbable material characteristic values and material partial factors)* for geotechnical design is, by principles of mathematics and physics, incorrect, and its application gives inexact or erroneous results not only for ULSD but also for *advanced numerical stress/strain models*.

This article has two basic theoretical aims which could be benefit greatly the geotechnical practice:

a) To demonstrate the incorrectness of ULSD in *geotechnics* and due to it in-effectivity and risk.
b) To present a concept of a more plausible and correct design theory which is simpler and in compliance with mathematical and physical principles.

The substitution of the material design value definition as a highly improbable input value and acceptance of the definition as *the most probable cautious input value would create a unified theoretical base for geotechnical designs, both of which are based on simple numerical models of limit equilibrium and advanced stress/strain numerical models*.

3. CONCEPT

The article compares design results of all relevant design approaches according to the Code (AP 1/1, AP 1/2 and AP 2 ≡ AP3) and a procedure noted as "Factor for Earth Resistance Design" (FERD) using real sets of material properties of unique soil groups according to a reliable soil database (DATABASE ITAM 2013). Slope stability was selected for the presented numerical analysis demonstrating theoretical problems concerning geotechnical design. A reason of the election is high sensitivity of soil slope masses and of course, their designs also. The analysis is carried out from three point of view:

A - Design of slope inclination angle
B - Influence of soil homogeneity variability on design of inclination angle
C - Influence of soil homogeneity variability for slope safety/risk

An example of the slope design procedure FERD is based on inputs of the soil group F5 and shows an applicability the procedure. Also, conclusions contain possible adequate adjustments of the Code.

4. METHODS

Below presented methods were applied by the author. The Apriori Integration Method and the calculation program file MINISLOPE v. 1.5 can be substituted by any other relevant method and programme and of course, the data set too.

4.1 Apriori Integration Method

The Apriori Integration Method (AIM) was developed in the late 1970s and 1980s (Koudelka–Procházka, 2001) when the problem of slope stability functional minimization was being researched. After deriving adequate analytical formulation of the functional it came to light that the results of the minimization in an elliptical integral could not be solved explicitly using the mathematical methods of the time. The problem could be solved only by numerical minimization using the method of slices for the integration of forces, which was being used by other authors as well (Janbu 1954, Myslivec 1954, Bishop 1955, Bishop–Morgenstern 1960, Cousins 1978 and others). Some other methods solve the same limit equilibrium using the method of slices along with the internal forces exerted between vertical slices of the mass above the slip surface; thus, the solution in this case is implicit and not smooth. Moreover, differences between the explicit and implicit solutions are negligible. A mathematically smooth functional is one that can be considered most suitable for application in mathematical methods. Smooth slope stability functional had been defined analytically by a method called the Apriori Integration Method.

The mentioned reference (Koudelka–Procházka, 2001) presents solutions of all numerical stability models used in common practice. An example of the numerical AIM model of an applied basic slope is briefly explained below (a complete model development and an example see Koudelka–Procházka, 2001, Sections 2.2 and 3.1):

Let $y = t(x)$ be the boundary of the slope surface (terrain) and $y = f(x)$ describe the shear surface, the admissible form of which is an arc of the circle. Let us assume that f is a function, i.e. there is only one value of y for every x within the admissible interval. In accordance with the principal idea of the static model (the equilibrium on the fixed shear surface together with the respective denominations is shown in Figure 2), it is possible to define the safety factor F on the shear surface as follows:

$$F = (N^{*}\tan\phi + C)/T \qquad (1)$$

where N and T are the normal and tangential components of weight of the soil above the shear surface, respectively, with respect to the shear surface, ϕ is the angle of soil shearing resistance and C is the cohesion component. Further, we define the following expressions and simplify them:

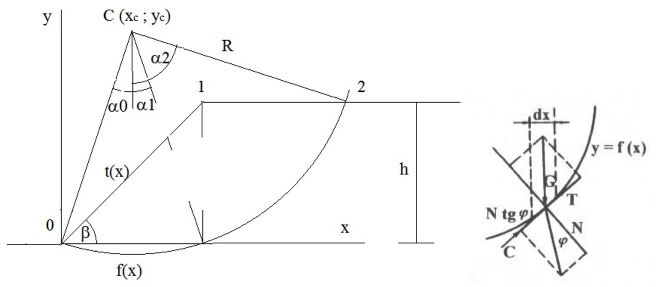

Figure 2 (a) Scheme for integration of forces on the slip surface. The integration is performed from the point *0* across *1* and *2* back to the initial point *0*, respectively, (b) Scheme of the forces acting on the slip surface.

$$p(x) = (x - x_C)/R = \sin\alpha, \qquad q(x) = \sqrt{(1 - p^2(x))} = \cos\alpha \quad (2)$$

where x_C is the coordinate of surface circle centre, R is the radius of the circle and α is the angle between the tangent to the slip surface at the given point and the x axis (see Figure 2 (a)).

Then, the general integrals of particular influences can be derived as follows:

$$F_1(p) = (\sqrt{(1 - p^2)})^3/3, \qquad F_2(p) = p^2/2,$$
$$F_3(p) = (\arcsin p + p\cdot\sqrt{(1 - p^2)})/2, \qquad F_4(p) = p^3/3,$$
$$F_5(p) = p - (p^3/3), \qquad F_6 p) = \arcsin p \qquad (3)$$

The contribution of the part of the circle between the boundary circle points i and j (which we will place within brackets like the influence on abscissas) will be:

$$[T] = [A \cdot F_2(p) + B \cdot F_1(p)]_i^j \ ,$$
$$[N \cdot tan\ \phi] = [A \cdot F_3(p) - B \cdot F_5(p)]_i^j \cdot tan\phi \ ,$$
$$[C] = c \cdot R \cdot [F_6(p)]_i^j \qquad (4)$$

where the constants of the line-segment integrals in this model can be expressed as:

$A^{01} = \gamma \cdot R^2 \cdot k_0$, $B^{01} = \gamma \cdot R \cdot k_0 \cdot x_c$, $B^{12} = \gamma \cdot R \cdot h$

and the constants of the integrals of shear surface arches can be expressed as:

$A = \gamma \cdot R \cdot y_C$, $B = \gamma \cdot R^2$

with the given values of γ, R, h, $k_0^{01} = (y_1 - y_0)/(x_1 - x_0)$ and the relations,

$t^{01}(x) = y_0 + k_0^{01} * x$, $t^{12}(x) = h$,

$f^{20}(x) = y_c - \sqrt{(R^2 - (x - x_c)^2)}$

The consequent equation for the safety factor F according to relation *(1)* is as follows:

$$F = (\textstyle\sum_i^n [N] \cdot tan\phi + \sum_i^n [C]) / \sum_i^n [T] \qquad (5)$$

The equations in (4) show that the integration is transformed into polar coordinates with parameters α and R. Material parameters of a homogeneous mass are constants. The integrals according to the relations in (3) are valid in general.

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4.2 Database ITAM 2013

This analysis uses a statistically-analyzed database. A database of identification and shear strength data with credible laboratory tests was developed at the Institute of Theoretical and Applied Mechanics of the Czech Academy of Sciences (ITAM) (Koudelka 2011). The last database version (DATABASE ITAM 2013 – see the reference) has data on 294 samples of Czech and abroad soils. The database was compiled in accordance with the standard soil classes—gravel, sandy and fine-grained—and it uses international symbols of soil classes and groups by Casagrande (see Table 1).

Table1 Distinction of soils in Database by Casagrande (USCS, 1952) and Czech Standard ČSN 73 1001 (1987)

Class	Group	Symbol	Class	Group	Symbol
Sandy	S2	SP	Fine-	F3	MS
Soils	S3	S-F	grained	F4	CS
	S4	SM	soils	F5	ML, MI
	S5	SC		F6	Cl, CI
				F7	MH, MV, ME
				F8	CH, CV, CE

The gravel and sandy soils S1-SW and fine-grained soils F1-MG and F2-CG are not considered owing to a low number of samples and hence not shown in Table 1. The comparative analysis is based on sandy (S) and fine-grained materials (F). It applies data of unit weight and effective shear strength and derived soil constructive characteristics of k_π and k_λ (see free DATABASE). Residual shear strength data were not applied.

4.3 Constructive similarity of soils

Analysing the similarity between the numerical models of limit equilibrium in general is evidently necessary to exist a geometrical similarity between the models, but it is still insufficient. Similarity solutions of three basic geotechnical tasks (shallow and pile foundations and slope stability) showed that the similarity of the analysed tasks depends on one of two similarity coefficients or on both them. The similarity coefficients are denoted by their authors as they follow:

$$\pi = \frac{c}{\gamma h} \quad (6), \qquad \lambda = \frac{c}{\gamma h\ tan\varphi} \quad (7)$$

where π – Hamilton`s similarity coefficient; λ – Janbu's similarity coefficient (Janbu 1954), and the parameters are: c – cohesion; φ – internal resistance angle; γ – unit weight; and h – parameter of geometrical similarity (e.g. slope height, foundation width, pile depth or other). Generally, both the similarity coefficients are dimensionless and valid for geotechnical limit balance tasks.

The idea of constructive soil similarity (Koudelka P. 2019) is derived from the similarity theory of geotechnical numerical models to build a criterion for engineering soil comparison. Transformation of Eqs. 6 and 7 for soil masses is applied only to edit out the parameters of geometrical similarity. Following this transformation, the following expressions for similarity between materials of the analysed masses are derived:

$$k_\pi = \frac{c}{\gamma} \quad (8), \qquad k_\lambda = \frac{c}{\gamma\ tan\varphi} \quad (9)$$

where constructive characteristics k_π and k_λ have a dimension in $[m]$.

The constructive characteristics make it possible to evaluate and compare soil structural resistances and their relative differences without exact knowledge of structure or geometry. Moreover, exploiting constructive characteristics does not require any new or special test. This new approach towards rating technical properties of soils gives true constructive characteristics of real soils.

4.4 Method of calculation

The stability functional of a simple homogeneous slope, according to equation (10), was numerically minimized in the early 1980s as follows:

$$F(f) = \frac{R_{e:}(f(x_c, y_c, R_c), t(x_c, y_c,), \gamma, \varphi, c)}{T(f(x_c, y_c, R_c), t(x_c, y_c,), \gamma,)} \qquad (10)$$

where:

R_c - radius of slip surface with the center at point C (x_c;y_c)
$R_{e:}$ - passive forces on the shear surface
T - active forces along the shear surface
F - function describing the form of permissible (circular) shear surface with the centre *(x_c,y_c)* and radius *R*
t - function describing the form of slope surface, ground urface
γ - unit weight of soil
φ - internal resistance angle
c - cohesion

The minimization was performed considering practically complete scales of the material, and geometry input parameters whose material inputs were expressed in the scale of *λ<0.005;∞>* and geometry inputs in the slope angle scale of *cotanβ<0.1;10>*.

5. NUMERICAL MODEL

The numerical analysis model exploits results of the minimization and thus, follows the model described in a previous section. The model is described as follows: a simple slope both with a horizontal platform and an upper surface in a homogeneous soil mass. Both the toe and deep surfaces are cylindrical (Figure 3).

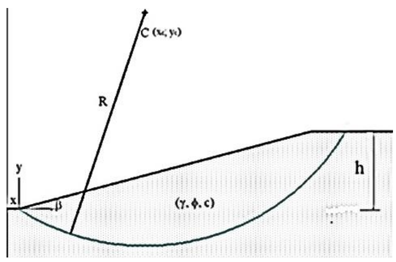


Figure 3 General scheme of numerical model of homogeneous mass with circular slip surfaces.

5.1 Input parameters

The analysis applies results of a statistical evaluation data processed by DATABASE ITAM 2013, especially the mean values of unique soil class groups as they follow in Table 2 and Figure 4. The maxima and minima of constructive characteristics k_λ of the unique soil classes are exploited as probable real limits.

Table 2 Inputs of soil properties according to the DATABASE ITAM 2013

STATISTICAL QUANTITIES OF DATABASE							
Mark	Sam.	Mean values			Variability coeff.		
	n	γ_m	ϕ_m	c_m	v_γ	v_ϕ	v_c
-	1	kNm^3	°	kPa	1	1	1
SANDY SOILS							
S1							
S2	17	1810	39,1	7,3	0,062	0,128	0,627
S3	12	1901	36,7	13,8	0,085	0,161	0,794
S4	29	1950	33,8	19,9	0,063	0,195	0,860
S5	12	2070	27,6	23,1	0,042	0,057	0,331
Ø$^{2-5}$	70	1927	34,7	16,2	0,064	0,161	0,775
FINE GRAINED SOILS							
F1							
F2	1	2197	29,9	27,0	0	0	0
F3	35	1961	27,4	34,5	0,050	0,142	0,556
F4	40	2008	23,8	44,2	0,065	0,154	0,613
F5	23	1989	22,7	37,4	0,048	0,137	0,601
F6	67	2016	22,2	44,8	0,046	0,151	0,474
F7	11	1988	19,3	74,0	0,066	0,152	0,695
F8	20	1955	17,3	45,4	0,069	0,303	0,338
Ø$^{3-8}$	196	1995	22,7	43,7	0,055	0,162	0,564

a)

b)

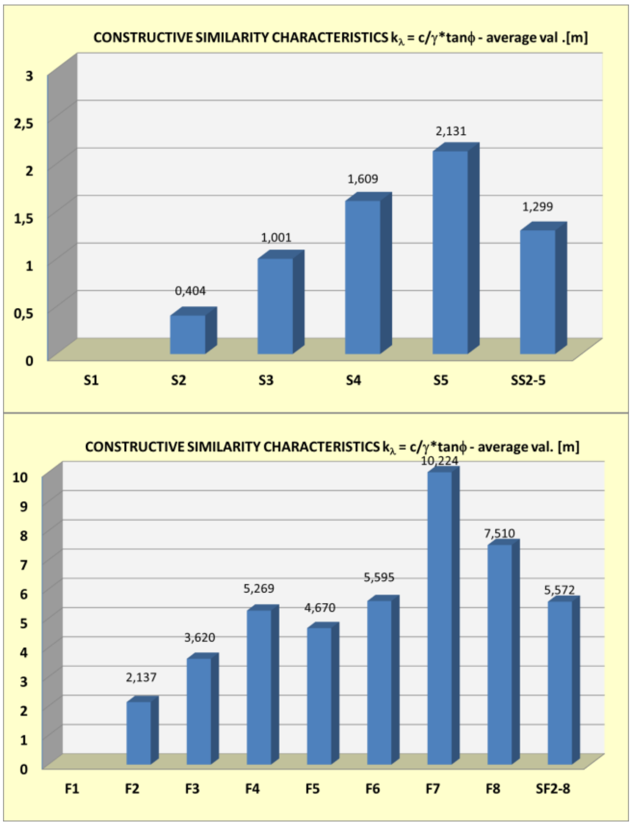


Figure 4 Mean values of constructive characteristics of sandy (a – above) and fine-grained (b – below) soils.

5.2 Outputs

Outputs of the programme MINISLOPE v.1.5 are both textual and graphical. The outputs can be different according to the given task. Tasks for the design of slope inclination and slope safety were applied. The text output presents the given inputs, the designed slope inclination or slope safety coefficient, and complete data critical slip surface with '*minimum minimorum*' earth resistance. The graphical output depicts the critical surface position and parameters. The analysis exploits data on slope inclinations or safety but does not present other parameters separately because, the author does not deem these parameters significant to the study.

6. DESIGN PROCEDURES

The ULSD in the Code, both in theory and practice, are based on a general definition of characteristic values which guarantee safety of designs of all structures composed from different materials and wide system of partial factors for a derivation of design values. This design concept solves the uncertainty of design inputs that are more or less random. These factors are specified for load, geometry, resistance and material. Characteristic values of load, geometry and resistance and the relevant partial factors are acceptable for structures in geotechnical design. On the other hand, the partial *material* factors and statistically *uncertain* definition of *material* characteristic and design values (performed in the Code by *three different* stipulation methods and *four* approaches) are not acceptable for soils, which introduces difficulties in the geotechnical design (see section 2 and Figures 1 (a), (b)). This design situation leads the presented analysis to compare the almost improbable models according to the Code procedures (see 5.1) with the most probable models of which procedure respects principles of mathematics (see 5.2).

The inputs (mean values) presented above in section 4.1 (Table 2 and Figures 4 (a), (b)) were further transformed into design values according to the relevant approaches of the design theories. The analysis compares two theories, Ultimate Limit State theory in the Code, which applies four approach procedures (AP), and the theory of the *most probable* earth resistance. The latter theory ensures design safety by the partial factor for earth resistance (FER), $\gamma_{R;e}$.

6.1 EN 1997-1 - ULSD

The Code defines basic characteristic values of a material according to their general definition (Par. 2.4.5.2(2) cautious estimation of the value affecting the occurrence of the limit state) and considers three stipulation methods for the characteristic values (Pars. 2.4.5.2(7) (a cautious estimate of the mean value of values of a large surface or volume of the ground), (8) (cautious estimate of the lowest or highest value) and (11) which is the most precise (probability of the worse value is less than of 5%).

The deviation of a characteristic value then depends on the variability coefficient of the property/parameter set by the equation:

$$X_k = X_m \, (1 - u_{0.05} \cdot v) \qquad (11)$$

where are

X_k characteristic value of the property

X_m mean value of the property

$u_{0.05}$ normalized parameter of the normalized distribution function for lower value probability of *5%*; Pearson III function considered for the value of $u_{0.05} = 1.65$,

v variability coefficient

The variability coefficient v can also be considered a criterion of homogeneity.

With partial material factors, the Code involves altogether four design approaches (A1/1, A1/2, A2, A3) for the derivation of the design values. Contemporary (2019) international discussions on selection of a reliable approach to design value assessment for geotechnical tasks are inconclusive. To ensure that the conclusions are valid both general and specific cases, it is necessary to apply the statistical method (2.4.5.2(11)) for the assignment of characteristic values, which must in turn have (statistically-) sufficiently wide sets of property data on relevant soils. This fact and the facts mentioned above form the crux of the main ULSD difficulties.

The analysis takes into account the approach procedures (AP) of design values summarized in Table 3. The approaches AP1/2 and AP3 apply the same material partial factors and factors of earth resistance for the given task of slope stability, and hence their respective design value results are the same. It should be noted that the analysis specifies AP1/2 only.

Table 3 Design approaches and partial factors, where *u* is considered value of statistically normalized parameter of the normalized Pearson distribution of type III.

Input		Mean values			Characteristic values u = 1.65				Design values			Analysis
Probability/factor		50%			Variability coefficient				Material partial factor			Earth resistance
Coeff./Factor		-	-	-	v_γ	$v_{\varphi'}$	$v_{c'}$	p(%)	$\gamma_{m\gamma}$	$\gamma_{m\phi'}$	$\gamma_{c'}$	$\gamma_{R,e}$
AP FERD[1)]		According to			-	-	-	50	1.0	1.0	1.0	1.5
EC7-1	AP1/1[3]]	Unique			According to			< 5	M1 = 1.0			R1 = 1.0
	AP1/2[4]]	soil groups of			unique soil groups of				M2 = 1.0	M2 = 1.25		
	AP 2	DATABASE			DATABASE				M1=1.0			R2 = 1.1
	AP3[4]]	ITAM 2013[2)]			ITAM 2013[2)]				M2 = 1.0	M2 = 1.25		R3 = 1.0
[1)] Approach according to the factor for earth resistance						[3)] Not considered						
[2)] www.itam.cas/Software/Koudelka DB/						[4]] The same design values of soil properties						

6.2 Factor (for) Earth Resistance Design

The design approach based on the factor for earth resistance (FERD) is simple. It uses *cautious mean material values* of soil properties as the design values, *no partial material factors* ($\gamma_m = 1.0$) *but* other *special factors for earth resistance*. The analysis considers the unique soil resistance factor, of $\gamma_{R;e} = 1.5$.

7. RESULTS

A comprehensive analysis was conducted on all exploitable soil groups from DATABASE (see Table 1), i.e. four sandy groups S2–S5 and six fine-grained groups F3–F8. The desired slope designs and estimations were presented in diagrams. Three types of graphs are presented for particular soil groups in a parametric extension of the similarity coefficient, λ, of the given group:

A - Design of slope inclination angles according to the ULSD (approaches 1/1, 1/2 and 2) and FERD

B - Influence of soil homogeneity variability on design of inclination angles according to the ULSD (approaches 1/1, 1/2 and 2) and FERD

C - Influence of soil homogeneity variability for slope safety/risk of the ULSD (approaches 1/1, 1/2 and 2), and the FERD in terms of safety factor F_s (see Eq. (5))

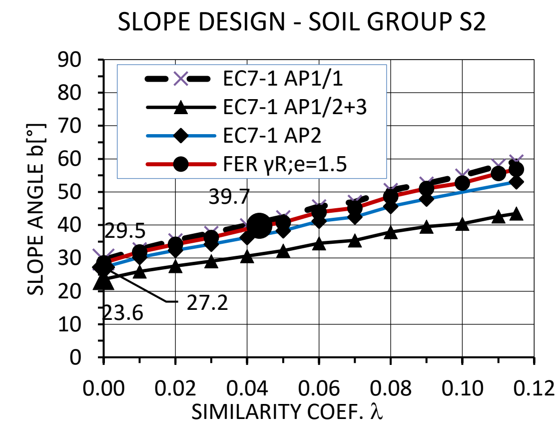
where the big point marks show comparable results of the mean material design values for slopes of the height of $h = 10$ *m*.

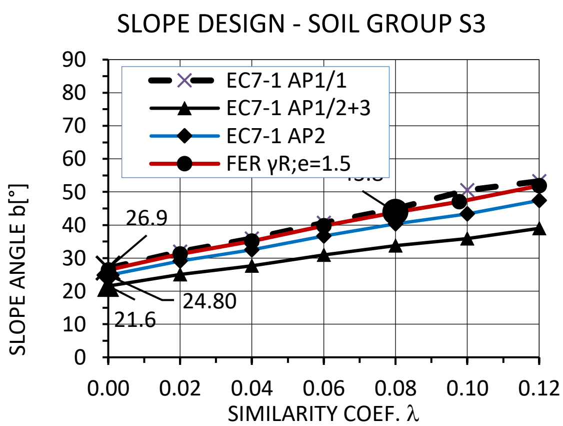
The horizontal-axis scales depict parameter λ, which contains all the needed calculation inputs (both material and geometric). The calculations are carried out in ranges of relevant and real scales of λ in the DATABASE groups, i.e. between the minimal and maximal group values. In accordance with the similarity theory, each value of λ expresses *any relevant combination* of the parameters. We should take into account that it would be incorrect to compare approach results for the *same value* λ because the Code procedures change material properties differently and therefore the resulting values of λ are different. For example, considering group mean material properties and the slope height of *10 m*, the approach values of λ in the result graphs are different (see the big marks).

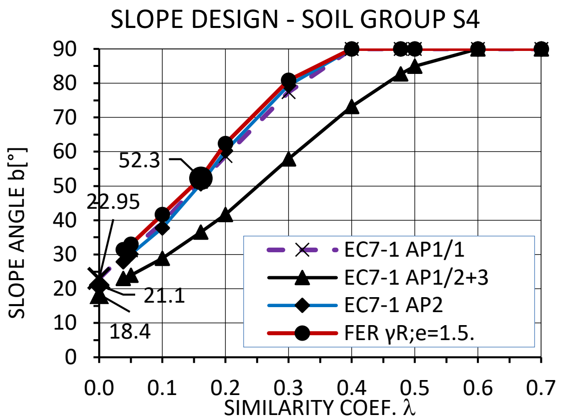
For clarity, the graphs are presented in three parts. This segmentation would make possible a better overall comparison of the unique points of view of the analysis.

7.1 Design inclination angles

For evaluating the graphs in Figures 5 and 6, it is necessary to consider that the coefficient λ expresses not only material soil properties but also changeable slope height. Cases from different approaches with the same coefficient λ value cannot have the same parameters and hence are incomparable. Of course, the histories give the valid slope design according to particular approach procedures. The comparison of slope designs for a height of *10 m* is represented both by big mark positions and inclination values. These solutions are adequate to the relevant design values derived after Table 2.







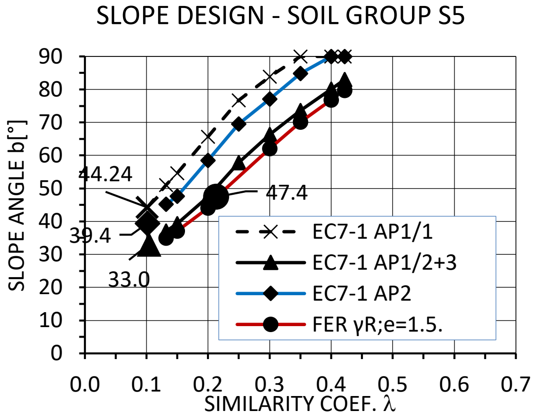
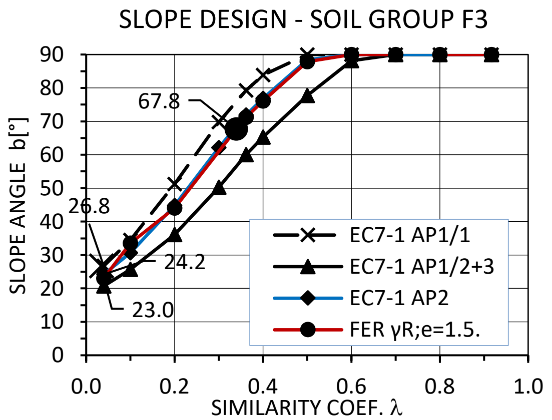
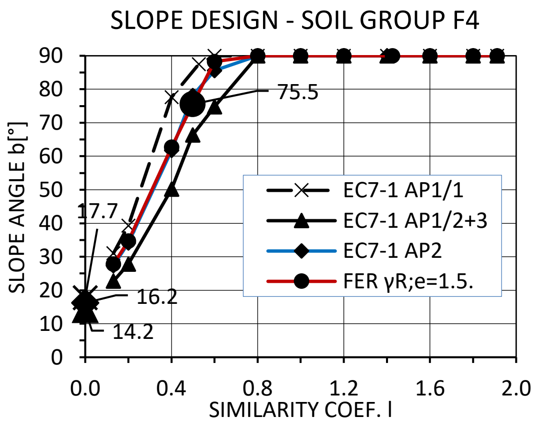
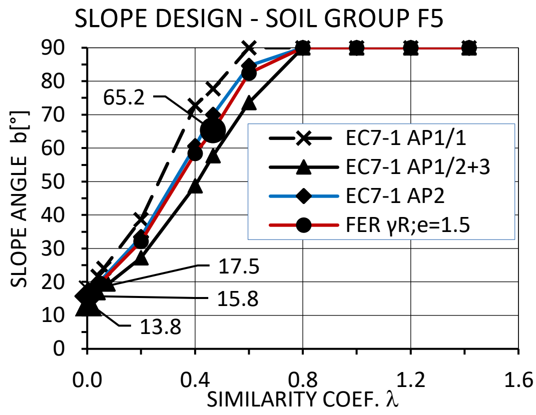
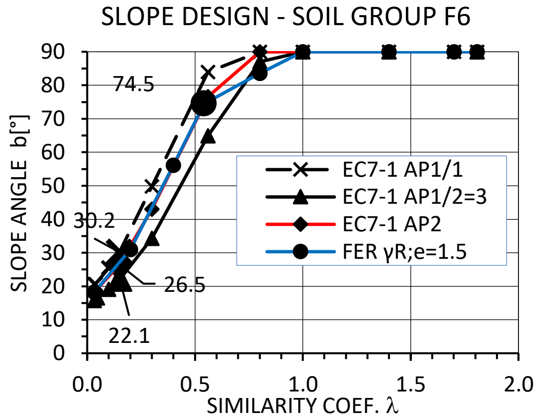


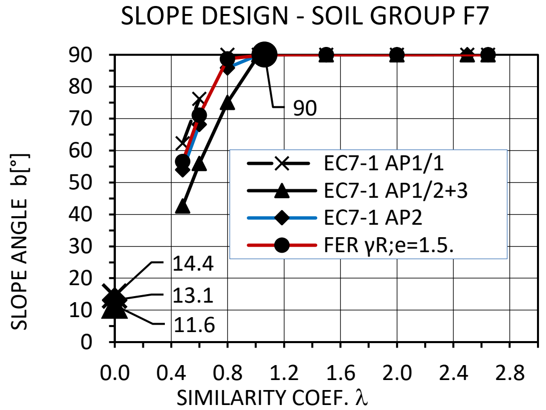
Figure 5 A - Design slope inclination angles for arbitrary parameter compositions and the adequate variability according to Table 2 – sandy groups.











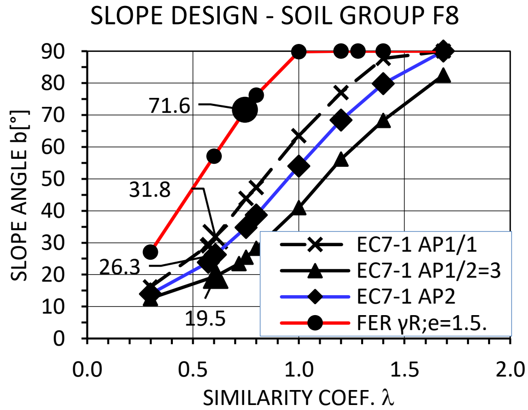


Figure 6 A - Design slope inclination angles for arbitrary parameter compositions and the adequate variability according to Table 2 – fine-grained groups (end).

7.2 Influence of soil homogeneity

The same approaches as the Code were used based on the assumption that a wide random variability of soil properties was possible, and an exigency to ensure geotechnical structural integrity in the event that the site soil mass would be different from the ones found through previous geological investigation. All means for attainment of structural safety were aimed at design material properties of the soil mass, i.e. definition of characteristic values, statistic stipulation method (2.4.5.2(11), and material partial factors (partial load factors are a safety reserve for structure service). Other points of view and the possibility of changing influences on the soil material (water, saturation, pressures, compaction etc.), are not taken into account in practical scenarios because the partial factors for soil resistance $\gamma_{R:e}$ are considered to be fixed at $\gamma_{R:e} = 1.0$ (AP1/1, 1/2, 3) or $\gamma_{R:e} = 1.1$ (AP2).

On the other hand, opposite possibilities of less variability are not considered. We will now take another consideration: DATABASE contains data from different sites. Its mean of material property values and variability coefficients are values of the *group* and not values of the *site*. The mean group values, and especially variability coefficients, are probably not on-site data, and site variability coefficients, probably less. If the variability coefficients are low, i.e. the soil is more homogeneous, the design parameters will be closer to the mean material values, the design results will come close and even behind, the FER design results. The designs of more or less homogeneous masses may be *without satisfactory structure safety* (see section 6.3). This section summarizes the results of analysis of this problem, i.e. design slope inclination angles of masses with declining homogeneity using input data of particular soil groups.

An investigation both of homogeneity influence group designs (graph type B – slope inclination angle) and design risk (graph type C – slope safety), which contain three steps distinguishing themselves by values of variability coefficient *v* noted as follow:

v_m - medium soil property values, height *10 m*, medium variability coefficients,

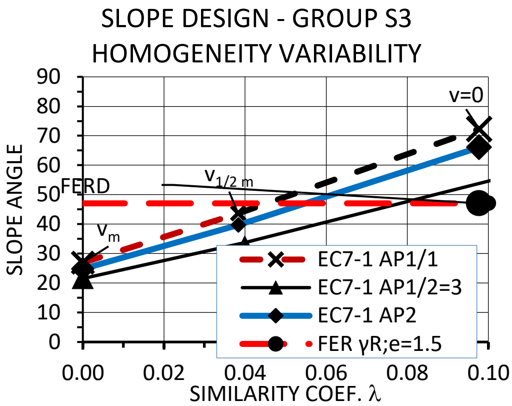
$v_{1/2m}$ - medium soil property values, height *10 m*, variability coefficients $v_{1/2m} = v_m/2$,

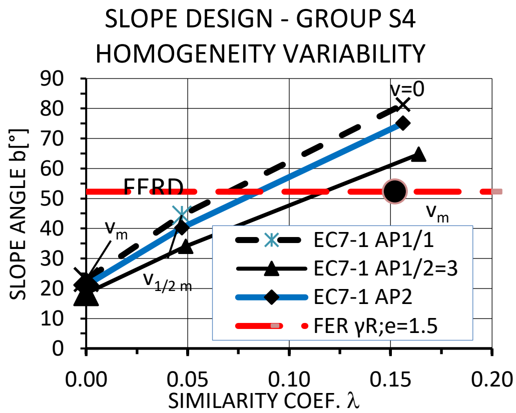
$v = 0$ - homogeneous mass, medium soil property values, height *10 m*, variability coefficients of $v = 0$.

In other words, the analyses of types B and C are based on the same inputs but they solve different tasks, i.e. slope design and assessment.

7.3 Risk of soil homogeneity

The big marks at lines in the Figures 5 and 6 (type A) represent slope designs of heights *10 m* each, the corresponding mean constructive characteristics k_λ, and the simultaneously designs of all corresponding input combinations of the given group values of λ_m. These designs of slope inclinations by the Code approaches are based on the mean v_{mc}, variability coefficients of mean soil properties v_m, i.e. $v_{m\gamma}$, $v_{m\phi}$, and which demonstrate the homogeneity of the given soil group. *It is probable that sometimes soil group homogeneity is less than soil site homogeneity*; thus, it is necessary to analyze a possible risk of





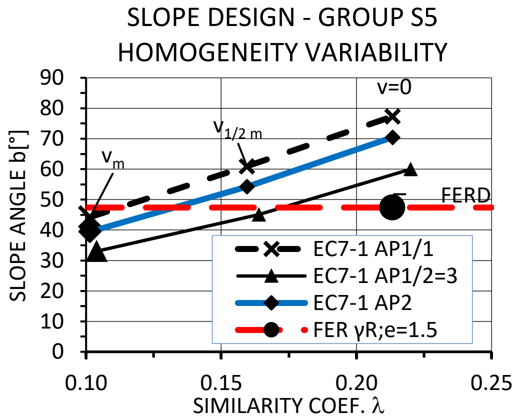
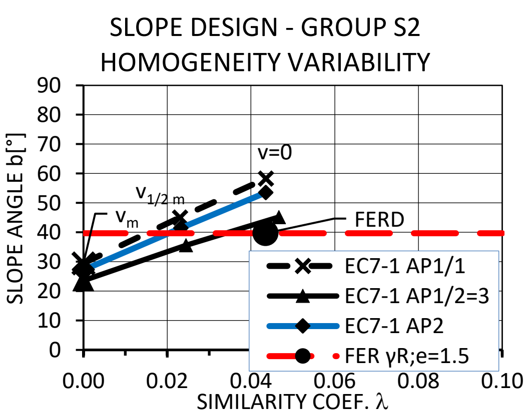
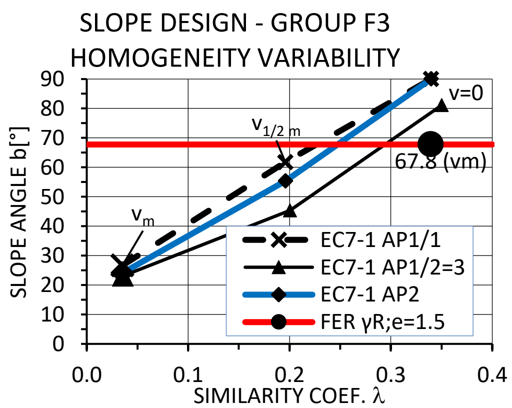
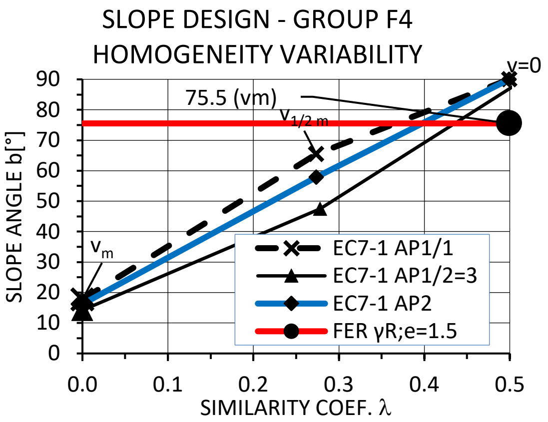


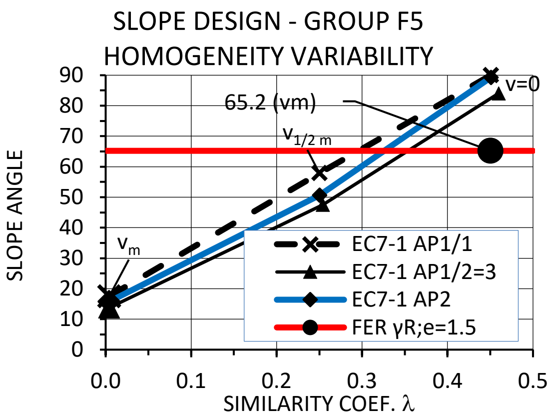
Figure 7 B – Homogeneity influence – Design slope inclination angles for mean variability v_m according to Table 2 – (see big marks) and histories of decreasing variability *v* – sandy soils.

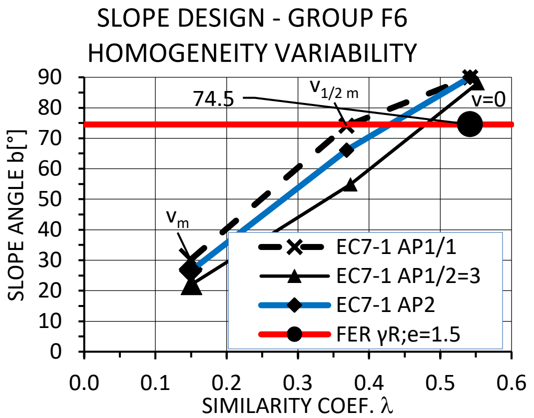
homogeneity at ULSD (no influence on FERD). The risk is evaluated according to values of safety factor F_s.

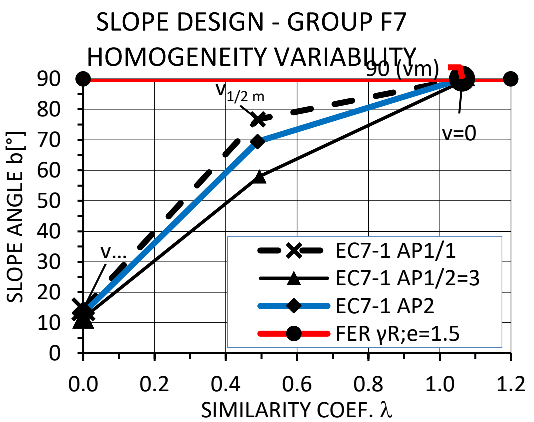












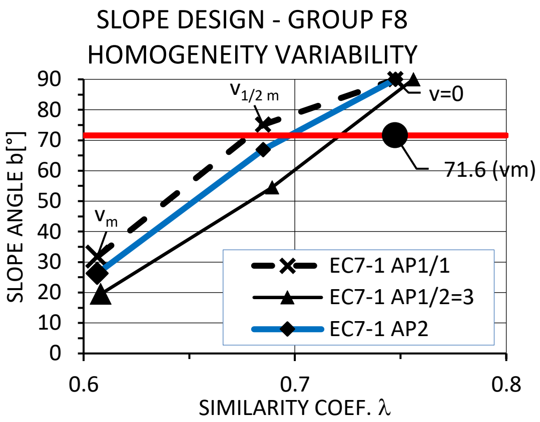
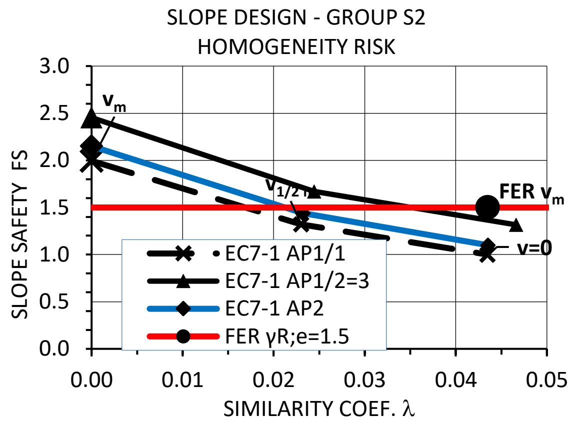
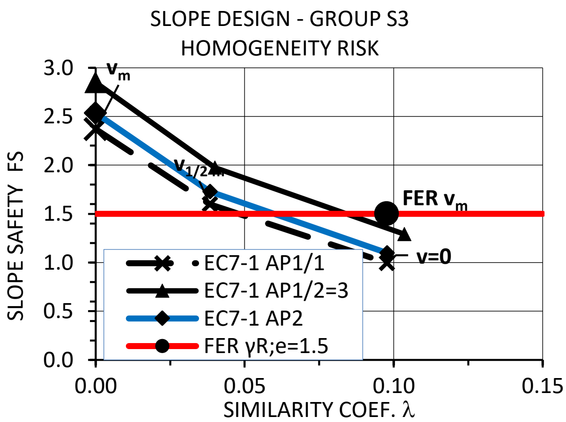
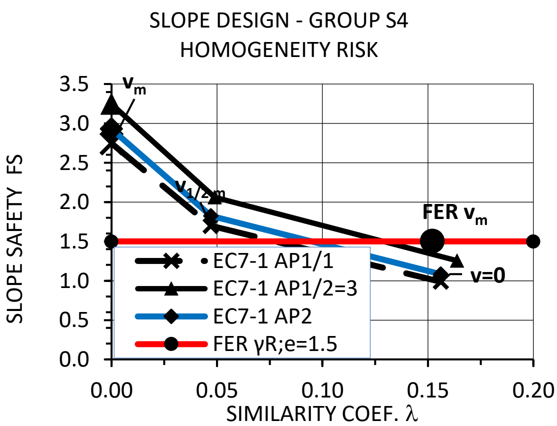


Figure 8 B – Homogeneity influence – Design slope inclination angles for mean variability v_m according to Table 2 – (see big marks) and histories of decreasing variability v – fine-grained soils.







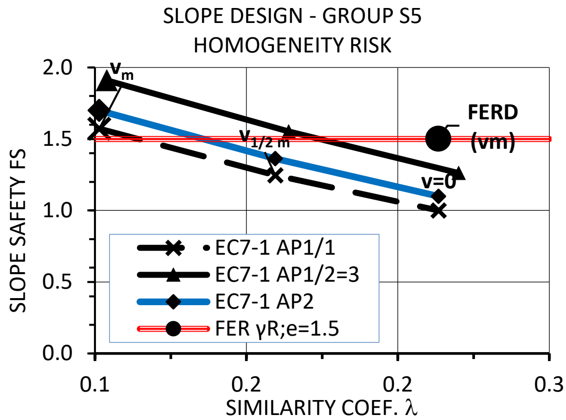
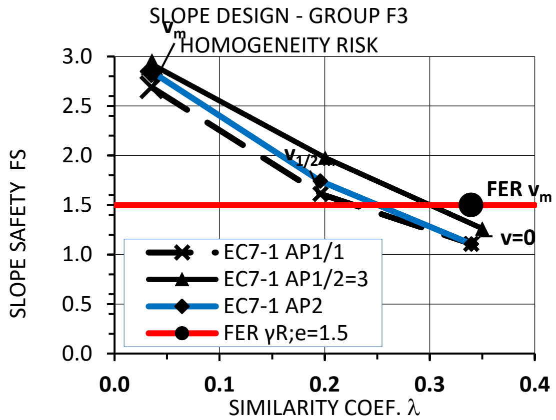
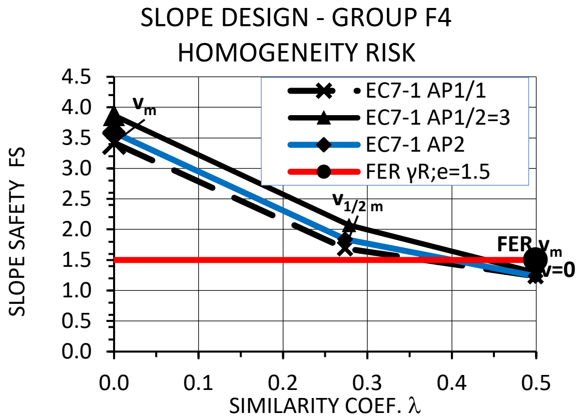
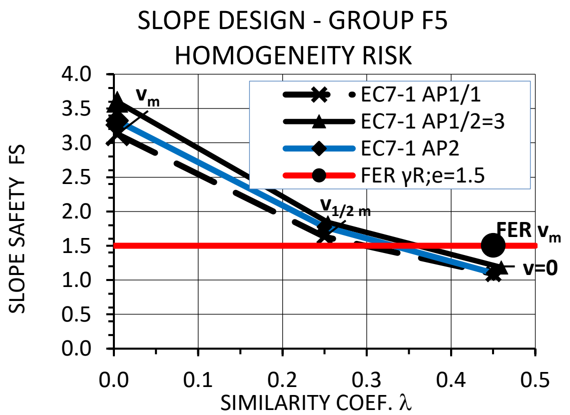
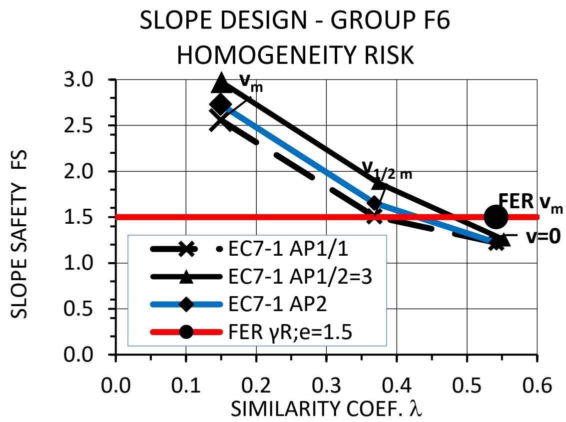


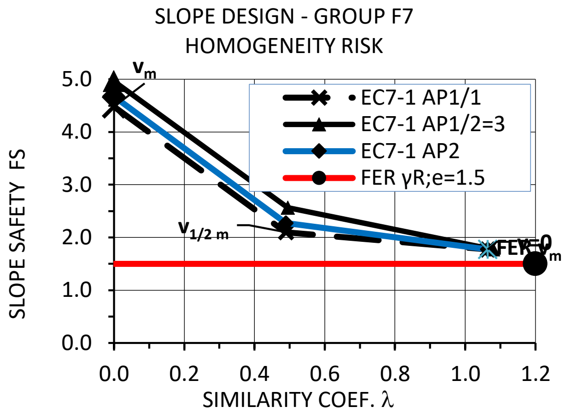
Figure 9 C – Homogeneity risk – Safety factors for mean variability v_m according to Table 2 – (see big marks) and factor histories during decreasing variability v – sandy soils.











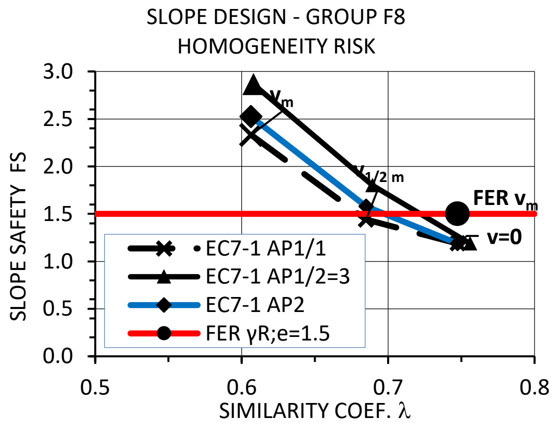


Figure 10 C – Homogeneity risk – Safety factors for mean variability v_m according to Table 2 – (see big marks) and factor histories of during decreasing variability v – fine-grained soils.

8. DISCUSSION

The examples of slopes of height *10 m* each and the adequate designs considering the mean group properties in Figures 5–10 are represented by the big marks. The examples can make a good comparison of results of the different Code approaches among themselves and the designs according to the factor for earth resistance $\gamma_{R;e}$ (FERD). The line histories show the influence of parameter changes (by Janbu's similarity coefficient λ) on the slope inclination designs (Figures 5–6), and the influence of soil variability/homogeneity on slope inclination (Figures 7–8) and safety/risk (Figures 9–10).

At first, view in all result figures in section 6 shows that mutual relations between the Code approaches are the same; the least conservative are the designs according to the approach AP 1/1, the less conservative is the approach AP2, and the most conservative are the approaches AP1/2 = AP3. The ULSD results of the same values of λ cannot be compared with those of the FERD results owing to very different design parameters. This comparison is supported by slopes of *10 m* represented by the big point marks.

The analysis affords both the minimum and maximum values of soil constructive characteristics k_λ of each soil group after the division of the slope height into *10 m*, which defines the real limits of the relevant group scale of λ. This definition makes it possible to derive the limit values of the constructive characteristics k_λ of each soil group through a simple back calculation.

It is logical and useful to discuss and evaluate the influences and their effects separately in accordance with section 6 above. Janbu's similarity coefficient λ is a basic parameter of the all result diagrams. It is important to note that the scales of the parameter λ of partial groups are different from one another and from the quantity line intervals' as well.

8.1 Inclination angles of design

Figures 5 and 6 show dependence of the slope inclination angle β on Janbu´s similarity coefficient λ, for sandy soils and fine-grained soils, respectively. The group dependences differ significantly and are influenced by group cohesion values (lower/higher cohesion leads to lower/higher value of λ). This is observable in Figures 5a and 5b (S2, S3) for sandy soils where the quantity line scales are in the intervals of $0 \le \lambda \le 0.115$ or $0 \le \lambda \le 0.274$, but in Figures 5c and 5d (S4, S5) they are in the intervals of $0.038 \le \lambda \le 0.700$ or and $0.101 \le \lambda \le 0.422$, respectively. The group inclination lines thus differ; the short lines of groups S2 and S3 finish in a slope inclination interval $40° < \beta < 60°$, whereas the long lines of groups S4 and S5 reach a vertical inclination of β=90° because higher amounts of fine-grained particles (SM, SC) increases soil cohesion.

Figure 6 gives adequate knowledge on fine-grained soils; the group lines are longer than for sandy soils, e.g. the group similarity is less. The line extents of the groups F3, F4, F5, F6, F7 and F8 are in the intervals of $0.035 \le \lambda \le 0.917$, $0 \le \lambda \le 1.912$, $0.004 \le \lambda \le 1.417$, $0.032 \le \lambda \le 1.809$, $0 \le \lambda \le 2.646$ and $0.299 \le \lambda \le 2.619$, respectively. All intervals are wide and all, except for group F8's history (Figure 6f), begin at approximately $\lambda = 0$. Group F8's history begins at the value $\lambda = 0.299$. All Code line histories reach for a vertical inclination of $\beta = 90°$ after the parameter value reaches $\lambda \ge 0.25$. These results are logical owing to cohesive soil masses of types MS, CS, ML, MI, CL, CI, MH, MV, ME, CH, CV and CE.

Relations between the ULS (Code) designs and FER designs are highly significant. They depend on combinations of the variability coefficients, v_γ , v_ϕ and v_c of the partial groups in Table 2. For the most part, in both sandy soils (S2, S3, S4) and fine-grained soils (F3–F7), the FERD line histories lie in the vicinity of the ULSD line histories AP1/1 or AP2. Variability of these soils is surprisingly similar:

- Means of variability coefficients of sandy groups S2-S4 are $v_\phi = 0.161$ and $v_c = 0.760$. It means a usual and not a high value of variability of shearing resistance angle ϕ with a low dispersal. On the other hand, a high value of cohesion variability v_c with higher dispersal.
- Means of variability coefficients of fine-grained groups F3-F7 are $v_c = 0.588$ and $v_\phi = 0.147$, which appears a usual and not high value of variability of shearing resistance angle ϕ with a very low dispersal. On the other hand, a relatively high value of cohesion variability v_c means lower dispersal.
- Very low variability of unit weight v_γ in an interval of ≤ 0.05; $0.09 \ge$ need not be considered as its influence is insignificant.

The inclination line relations of the other group, S5, are significantly different. The line histories in Figure 5d (group S5) are opposite to those in Figures 5a, b, c (groups S2, S3, and S4):

- The inclination line of FER designs is under the lines of the ULS designs. The intervals λ are relatively to the fine-grained groups shorter ($0.101 \le \lambda \le 0.422$). The example results (big marks) of the Code approaches are nearer to the FER design of $\beta = 47.4°$, and the slope inclination of AP 1/1 is of $\beta = 44.2°$. These unusual results follow from an unusually low variability of shear strength: $v_\phi = 0.057$ and $v_c = 0.331$. It means *a lower variability of shear strength makes the results more optimistic*. This is important, and it raises the questions – *how low the shear strength variability of a real partial site could go and what influence does the lower variability have on design*. These questions are answered in sections 7.2 and 7.3.

The inclination line relations of group F8 also differ:

- The inclination lines of the Code approaches in Figure 6f are in their usual relations with those in Figures 6 a, b, c, d, e (groups F3–F7); however, the line history of the FER designs is significantly higher. The lines are in the interval $0.299 \le \lambda \le 2.619$ and relatively longer than the other groups. The example results (big marks) of the Code approaches are farther from the FER design of $\beta = 71.6°$ and the design slope inclinations of the Code approaches are in the interval $19.5° \le \beta \le 31.8°$. These results follow from an unusually high variability of shear resistance angle, $v_\phi = 0.303$ and an unusually low variability of cohesion, $v_c = 0.338$. It means that the high shearing resistance variability has a strong influence on the Code approach design but not on the FER design. *To obviate possible dangerous designs of steep slopes, the FER design procedure used in this analysis should be adopted.*

The real group intervals λ described above, according to DATABASE ITAM 2013, give the probable scales of group material properties. It should be noted that the comparable example results (big marks) of the Code approaches are placed on point positions of $\lambda = 0$ or on positions of $\lambda = \lambda_{min}$, and also from real intervals before λ_{min} (S4, S5, F4, F7). The comparable example results according to FERD (big *round* marks) are placed approximately on the mid-interval positions. It can be noted from the big mark positions that the adequate slope inclinations according to the Code are milder than the adequate slope inclinations according to the FERD. Table 4 presents another point of view on effectivity/conservatism of the slope designs, and compares the example designs of possible slope heights and inclinations according to different design procedures based on the group mean value parameters.

Table 4: Comparison of design heights *h* of slopes according to the FERD procedure with the designs according to Code procedures.

Approach	Inputs	S2		S3		S4		S5	
		β	h	β	H	β	h	β	h
-	-	°	m	°	M	°	m	°	m
AP1/1		29.5	unlimited	27.0	unlimited	23.0	unlimited		8.6
AP1/2	Table 2	23.6		21.6		18.4		47.4	5.2
AP2		27.2		24.8		21.1			7.1
FER		39.7	10.0	47.1	10.0	52.3	10.0		10.0

Approach	Inputs	F3		F4		F5		F6		F7		F8	
		β	h	β	H	β	h	β	h	β	h	β	h
-	-	°	m	°	M	°	m	°	m	°	m	°	M
AP1/1			1.1	17.7	unlimited		0.11		3.1	14.4	unlimited		5.5
AP1/2	Table 2	71	0.8	14.2		65	0.08	74	2.3	11.6		72	4.2
AP2			1.0	16.2			0.10		2.7	13.1			4.9
FER			10	75.5	10		10		10	90	10		10

Note: ULSD examples considering no *design* cohesion can have *theoretically* unlimited heights – see and compare the slope inclination, β.

The example results in Table 4 confirm the results in Figures 5 and 6, which show that under the consideration that FERD results verified over a long practice period of time are substantially more effective/non-conservative/optimistic than the Code designs are considering the variability coefficients v_m of the particular groups. The possibility of lower values of $v < v_m$ due to a higher mass homogeneity is evaluated in the following sections 7.2 and 7.3.

8.2 Influence of soil homogeneity

This part of the analysis is addressed to influence of the lower both soil and group variability of $v \leq v_m$ of the above presented slope examples applying the group mean parameters (see in Figures 5 and 6 – big marks). This is valid for the ULS designs. The FER designs are independent of soil variability and their inclination histories in Figures 7 and 8 are constant. The variability values v_m represent the material property sets of the groups, i.e. the sets of all similarly identified soils (from the database) whose samples were collected from different sites. Owing to it, the values v_m may be more or less considered as the upper limits of variability. Figures 7 and 8 give a good picture of the influence of decreasing variability. In these figures, the examples indicated by $v_{m/2}$ are calculated applying half the group values of property variability $v_{m/2} = v_m/2$. The example results, as indicated by v_0, are designed for fully homogeneous masses.

Figures 7 and 8 show the dependency of slope angle on similarity coefficient λ, and simultaneously on variability coefficient v. Mutual relations of the ULS example designs remain intact during the decrease in variability, akin to the values of v_m (in Figures 5 and 6): the designs according to AP1/1 are the least conservative ones; the designs according to AP1/2 (and also AP3) are the most conservative ones; and the designs according to AP2 lie in between. Slope angles β of variable property masses (v_m) of all groups correspond to the values of the examples (big marks) in Figures 5 and 6. A decrease in the soil property variability leads to an increase in the design slope angles above the adequate values according to the FER designs, however, overlapping magnitudes are not the same.

As mentioned in Chap. 7.1, the ULS design inclinations for the sandy groups S2, S3 and S4 (Figures 5 a, b, c) and their mean variability are very mild (around 20°-30°). FER designs are steeper in the interval, ≤ 40°; 52 °≥. If the variability decreases by half ($v_{m/2}$), then the ULS design inclinations increase significantly in $35° \leq \beta \leq 45°$, i.e. under or slightly above the FER designs. Inclination values for the group S5 (Figure 5d) somewhat differ. The example results considering a variability of v_m (β = 33°, β = 44°) are closer to the FER design inclination of β=47° and resulting inclinations for the decreased variability of $v_{m/2}$ are either closer to FERD or distinctively above.

All slope inclinations of fully homogeneous sandy soils (S2–S5) according to the ULS approaches, which are designed considering no variability of v_0 ($45° \leq \beta \leq 75°$), are steeper than the FER design inclinations of $40° \leq \beta \leq 52°$. It means that large parts of more homogeneous sandy soils between the property variability values of $v_{m/2}$ and v_0 *lead to less standard designs with safety issues.*

The designs in fine-grained soils, considering mean variability v_m similarly to sandy soils, produce ULS design inclinations (see Figure 6) very mild, milder than for sandy soils: F3, F6 and F8 in an interval of $20° \leq \beta \leq 26°$, F4, F5 and F7 in an interval $12° \leq \beta \leq 16°$. The FER designs for the groups F3-F6 and F8 are much steeper in the interval $65° \leq \beta \leq 76°$. The design slope of the group F7 is vertical. If the variability decreases by half ($v_{m/2}$), then the ULS design inclinations decrease very substantially at an interval of $45° \leq \beta \leq 77°$ but under FER designs. An inclination value for the group F8 AP1/1 of *$\beta = 75°$* (Figure 6f) reaches slightly over the FERD inclination value of *$\beta = 72°$. The ULS design inclinations, considering the variability of $v_{m/2}$ and comparing them to the FER designs, appear more conservative for steep slopes.*

All slope inclinations of fully homogeneous, fine-grained soils (F3–F6 and F8), according to the ULS approaches designed, and considering no variability of v_0 ($81° \leq \beta \leq 90°$), are steeper than the FER design inclinations $65° \leq \beta \leq 76°$. The group F7 is an exception because of the FER design inclination of *90°*. It means, except for F8, the larger part of a more homogeneous sandy soil between the property variability values of $v_{m/2}$ and v_0 *leads to a less standard/safe ULS design.* However, the *vertical slope of the soil group F7 and other very steep slopes, according to FERD (using $\gamma_{R;e} = 1.5$), do not appear safe either.*

8.3 Risk of soil homogeneity

Uncertainty around the slope design of highly homogeneous soil masses at unique sites should be analyzed. The results of this analysis are presented in Figures 9 and 10. Risk/effectivity of all the design examples on soil groups, including non-standard and steep slope designs, is analyzed applying assessments of the slope designs from the previous, Chap. 7.2, and the slope inclinations from Figures 7 and 8 (big marks). They are fed the same input sets as the Code approaches, and the assessment gives the resulting safety factor/partial factor for earth resistance. All group designs according to FER are calculated for the partial factor for earth resistance of *$\gamma_{R;e} = F_s = 1.5$*. Their constant lines and value in all graphs serves as a criterion.

All relations between the results of the Code approaches are visually opposite, but in fact, are similar; AP1/1 is the most conservative (lowest histories); AP1/2 ≡ AP3 are the most optimistic (highest histories); and AP2 is between them (mid histories). Features of histories are very similar and almost parallel; consequently, we can discuss sandy soils and fine-grained soils in the same breath.

Safety factors of the designed slopes of soil homogeneity variability v_m of sandy groups S2-S4 and fine-grained groups F3-F8 in the intervals of $1.94 \leq F_s \leq 3.24$ and $2.52 \leq F_s \leq 4.96$, respectively, *appear distinctly uneconomical*. Safety factors of designed slopes of the group S5 in an interval $1.55 \leq F_s \leq 1.70$ are approximately in accordance to the FER designs.

Safety factors of the examples considering the decreased homogeneity variability $v_{m/2}$ for the approach AP1/1 lie in a sandy soil interval $1.25 \leq F_s \leq 1.70$ and fine-grained soil interval $1.44 \leq F_s \leq 2.10$. Similarly, safety factors for the approach AP1/2 are in the intervals $1.55 \leq F_s \leq 2.06$ and $1.80 \leq F_s \leq 2.57$, respectively, and the factors for the approach AP2 are in the intervals $1.36 \leq F_s \leq 1.82$ and $1.58 \leq F_s \leq 2.28$, respectively. The interval values support the factors of almost all Code designs fluctuate in the interval $1.5 \leq F_s \leq 2.5$, i.e. above the FER design factor value *$F_s = 1.5$*, except the approaches AP1/1 and AP2 for groups S2 of F_s (1.32;1.44, respectively) and S5 of F_s (1.25;1.36, respectively), and AP1/1 for the group F8 of F_s (1.44). The factors for sandy groups S2–S4 appear acceptable because the slopes are not steep. The safety factors for group S5 appear risky for the slope inclinations of *β (61°, 54°, respectively)*. The safety factors of the fine-grained soils (F3-F8) are high but slope inclinations are steep *$45° \leq \beta \leq 77°$*; thus, the safety factor values may not be entirely uneconomical. *A problem ´slope steepness against slope reliability´ should be solved.*

The extreme situation of safety of the fully homogeneous slope considering no soil property variability v_0 is summed in Table 5.

Table 5 Slope inclination designs β in homogeneous masses and their safety factors F_s.

Approach	Inputs	S2		S3		S4		S5	
		β	F_s	β	F_s	β	F_s	β	F_s
-	-	°	1	°	1	°	1	°	1
AP11		58	**1**	72	**1**	81	**0.99**	77	**1**
AP12	v_0	45	1.3	56	1.3	56	**1.25**	60	**1.25**
AP2		53	**1.1**	66	**1.1**	66	**1.08**	71	**1.1**
FER	v_m	40	1.5	47	1.5	52	1.5	47	1.5

Approach	Input	F3		F4		F5		F6		F7		F8	
		β	F_s	β	F_s	β	F_s	β	F_s	β	F_s	β	F_s
-	-	°	1	°	1	°	1	°	1	°	1	°	1
AP11		90	**1.11**	90	**1.23**	89	**1.1**	90	**1.22**	90	1.77	90	**1.19**
AP12	v_0	81	**1.26**	89	**1.26**	84	**1.18**	88	**1.26**	90	1.77	90	**1.19**
AP2		90	**1.11**	90	**1.23**	89	**1.1**	90	**1.22**	90	1.77	90	**1.19**
FER	v_m	88	1.5	76	1.5	65	1.5	75	1.5	90	**1.5**	72	**1.5**

It is obvious that the safety factors F_s of the homogeneous slope, according to the Code approaches in Table 5, get near to the values of the partial factors for shear strength *$\gamma_\phi = \gamma_c = 1.0$* or *1.25*, or the partial factors for earth resistance $\gamma_{R;e}$*=1.0* or *1.1* (see Table 3). *The resulting Code designs of the very steep slopes for homogeneous masses are risky and unacceptable in practice. Simultaneously, the partial factor for earth resistance* $\gamma_{R;e}$*=1.5 does not appear to be sufficient for steep and vertical slopes. The Code statistics stipulation method for the characteristic values (Par.2.4.5.2(11) should be eliminated and substituted by a direct definition of design values.*

9. SUGGESTED CONCEPT OF DESIGNS IN GEOTECHNICS

An example of the slope design analysis according to different Code approaches and factor for earth resistance shows the result of using the incorrect definition of *very improbable* design soil properties. A suggested design concept is to substitute the Code definitions of the characteristic values (2.4.5.2) and design values of geotechnical parameters (2.4.6.2), respectively, by a direct definition of *most probable* design values of soil properties and a *new system* of partial factors for earth resistance. An instance of the new definition is as follows:

Design value *of geotechnical parameter shall be selected as a cautious estimate of* ***the most probable*** *value.*

Also, this concept is fully in compliance to the principles of stress/strain models of advanced numerical methods.

The new systems for the factors for earth resistance can be applied for each geotechnical problem separately, according to national experiences. Their possible concept is presented below.

9.1 Slope stability

The design value definition may be considered in two ways: a) the most probable values the soil set properties ($\gamma_d = \gamma_m$; $\phi_d = \phi_m$; $c_d = c_m$) or b) soil properties of the sample *a* with the most probable value of similarity characteristics k_λ by Eq. (9) ($k_d \approx k_a \rightarrow \gamma_d = \gamma_a$; $\phi_d = \phi_a$; $c_d = c_a$).

The system of factors for earth resistance $\gamma_{R;e}$ exploiting the analysis results is based on the principle of a variable factor dependent on inclination magnitude, as shown in Table 6.

Table 6 Dependence of partial factor for soil resistance $\gamma_{R;e}$ on slope inclination angle β

Quantity	Unit	Variable partial factor of soil resistance $\gamma_{R;\varepsilon}$ - slope angle β														
β	°	0	5	10	15	20	25	30	35	40	45	50	55	60	70	80
$\gamma_{R;e}$		1	1.05	1.13	1.18	1.24	1.29	1.35	1.41	1.46	1.5	1.5	1.5	1.55	1.69	1.96	2.23

Note: Values of partial factor for earth resistance $\gamma_{R;e}$ are valid for situations of usual risk. In situations of increased risk or importance, the values of the factor should be increased, for instance:

a) Risk of large economic damages ………. *5 %*

b) Risk of health hazard or loss of life ………. *8 %*

The maximal factor value is of $\gamma_{R;e}$*=2.5*.

9.2 Example

For illustration it is selected the simple slope (see Figure 3) with input parameters of the F5 soil property set (ML, MI) and eliminated one sample with extremely high cohesion. Then input parameters are:

$$h = 10\ m\ ,\ \gamma_d = 1989.35\ kg/m^3 = 1989.35*9.81/1000 = 19.51\ kN/m^3\ ;$$
$$\phi_d = 22.7°; \qquad c_d = 37.4\ kPa.$$

Following slope design calculations may be carried out by any method. Here is applied the minimization results of the functional (10) which were summarized in a programme called MINISLOPE v. 1.5 (attached in Koudelka–Procházka, 2001). The results are relations between the minimal stability number F_{01} and Janbu´s similarity coefficient λ for different slope inclinations in the ratio of *1:n* expressed where *n* is *cotan* β.

Step 0:

Task: To design slope inclination with the factor for soil resistance of $\gamma_{R;e} = 1.5$.

Calculation:

$\lambda = 37.4 / (19.51 * 10 * \tan 22.7°) = 0.4594 \quad \rightarrow \quad F_{01} = \gamma_{R;e} / (\gamma * h) = 7.9059 \rightarrow$

$n = cotan\beta = 0.43908 \quad \rightarrow \quad \beta = 66.29°.$

Evaluation:

The factor $\gamma_{R;e}$ value should be into an interval <1.69; 1.96> (see Table 6). Interpolation more correct factor $\gamma_{R;e}$ in Table 6 gives

$\gamma_{R;ecor0} = 1.69 + (1.96 - 1.69) * (66.29° - 60°)/(70° - 60°) = 1.86$

Conclusion:

The factor for soil resistance is low, the slope inclination shall be milder.

Step 1:

Task:

Approximation the considered value of factor for soil resistance to a more correct value $\gamma_{R;ecor}$ according to Table 6 for $\beta_1 = 60°$ (an adequate factor is of $\gamma_{R;ecor1} = 1.69$). To assess the slope of inclination of β_1.

Calculation:

$n = cotan\ 60° = 0.57735 \rightarrow \quad \gamma_{R;e1} = 1.61 < \gamma_{R;ecor1} = 1.69$

Conclusion:

The factor for soil resistance is low, the slope inclination shall be milder.

Step 2

Task:

Approximation the considered value of factor for soil resistance to a more correct value $\gamma_{R;ecor}$ according to Table 6 for $\beta_2 = 58.2°$. To assess the slope of inclination of β_2.

Calculation:

$\gamma_{R;ecor2} = 1.55 + (1.69 - 1.55) * (58.2° - 55°)/(60° - 55°) = 1.640$

$n = cotan\ 58.2° = 0.62003 \quad \rightarrow \quad \gamma_{R;e2} = 1.645 \approx \gamma_{R;ecor2} = 1.640$

Conclusion:

The factor for soil resistance $\gamma_{R;e2}$ is adequate to the values in Tab. 6.

Step 3 - Control task:

To check the slope design according to the Step 2 in a case of the most unfavorable soil of the set.

Procedure:

The similarity characteristics k_λ of soils of the set are compared and found the minimal one.

Calculation:

$k_{\lambda min} = 2.602$. Its property values are of $\gamma_{dmin} = 19.67\ kN/m^3$, $\phi_{dmin} = 25.6°$; $c_{dmin} = 25.0\ kPa$.

$n = cotan\ 58.2° = 0.62003 \quad \rightarrow \quad \gamma_{R;e3} = 1.348 > \gamma_{R;emin} = 1.1 \sim 1.2$

Conclusion:

The slope of the inclination of $\beta_2 = 58.2°$ is safety.

9.3 Other geotechnical structures

Design concept of spread and pile foundations, retaining structures, and others should also be based on the direct definition of geotechnical parameter design value mentioned above. It should be noted that each type of structure needs its own system of factors for earth or other adequate resistance. The systems should distinguish between different cases and conditions, and especially the period of structure service on basis of adequate analyses and experience.

10. CONCLUSIONS

Geotechnical design concerns soils which are unlike specific construction materials. Geotechnical structures are designed in highly complex plastic non-linear strain areas (see section 2). The analysis presented in this article and also the practice show that the geotechnical design only needs simple but important adjustments as opposed to other Codes.

The wide general analysis of the ULS-based slope designs and designs based on FERD makes it possible to draw conclusions on the type of application ideal for producing the design in a simpler, theoretically correct manner; a brief concept of adjustments of the

Code is specified in brackets:

a) The direct definition of design values of soil parameters such as '*Design value of geotechnical parameter shall be selected as a cautious estimate of the most probable value'. (to eliminate: par.2.4.5.2 "Characteristic values of geotechnical parameters", par.2.4.6.2 "Design values of geotechnical parameters", pars.2.4.7.3.4.1 up to 4 "Design Approaches" except of DA1 combination 1; to define design values anew – see above)*
b) Adequate factors for soil resistance can ensure safety and reliability of soil structures and reduction in costs (to eliminate the partial factors for earth resistance in the Annex A and to specify new others.
c) *The new system of the partial factors for soil resistance would afford an opportunity to subdivide values of the factors more adequate to importance, conditions, service time and riskiness of the soil structure.*
d) The adjustments mentioned above lead to defining one design approach accurately (instead of actual of 12 possibilities).
e) The adjustments are in accordance to all major geotechnical problems.
f) The suggested concept of designs in geotechnics based on FERD is applicable, stress/strain numerical models of advanced methods including.

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12. DATA AVAILABILITY

a) Data available in repository online:
   DATABASE ITAM 2013; Koudelka P.- Hudek J: Soil properties. Czech Academy of Sciences-Institute of Theoretical and Applied Mechanics, Praha, Czech Republic; free at www.itam.cas.cz/Software/Koudelka DB/.
b) Data and models available from corresponding author by request:
   The reference: "Koudelka P., Procházka P. (2001): *A priori Integration Method – Analysis, Similarity and Optimization of Slopes*. 2nd ed., ČSAV Academia, 168 ps. Prague, Czech Republic" containing also simple program file MINISLOPE v. 1.5 (in DOS) is sell off. An author copy is available.

13. NOTATION LIST

AIM - Apriori Integration Method (see Koudelka-Procházka 2001, further K+P 2001)
CEN - European Committee for Standardization, TC250
EN 1997-1 - European Standard Eurocode 7: Geotechnical design – Part 1: General rules (the Code)
FERD - Factor for Earth Resistance Design
IS SMGE - International Society for Soil Mechanics and Geotechnical Engineering
- TC205 Safety and Serviceability in Geotechnical design
- TC304 Engineering Practice of Risk Assessment and Management

ULSD - Ultimate Limit State Design (EUROCODE EN 1997-1: Geotechnical design - Part 1: General rules)
k_λ - constructive characteristics of soil [*m*] according to eq. (9)
k_π - constructive characteristics of soil [*m*] according to eq. (8)
v_m - coefficient of homogeneity/variability of group *mean* property values (γ_d - unit weight, ϕ_d - angle of shearing resistance, c_d – cohesion) in Janbu´s similarity coefficient λ
$v_{m/2}$ - coefficient of homogeneity variability of group property values ($\gamma_{dv/2}$ - unit weight, $\phi_{dv/2}$ - angle of shearing resistance, $c_{dv/2}$ – cohesion) considering half values of the variability coefficients of each soil property in Janbu´s similarity coefficient λ
α - angle between the circulant to the given slip surface point and axis *y* (see Figure 1)
β - angle of slope inclination
$\gamma_{R;e}$ - partial factor for earth resistance (EN 1997-1)
λ - Janbu´s similarity coefficient (Eq. (7))
λ_m - medium value of Janbu´s similarity coefficient of a soil group

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