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RESEARCH ARTICLE

Calculation of Presumed Bearing Capacity of Shallow Foundations According to the Principles of Eurocode 7

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Abstract

Various simplified design procedures can be found in the literature, intended for the design of shallow foundations of lower importance buildings and pre-dimensioning. In most cases, these design procedures are based on vaguely defined soil types and parameters, and are not compatible with Eurocode. The aim of this paper is to establish a „design procedure by prescriptive measures", according to the guidelines of Eurocode 7. Within this framework, previous design procedures are reviewed and a new procedure is developed for the simplified calculation of the bearing capacity of shallow foundations, conforming to the principles and rules of Eurocode 7.

Keywords

shallow foundations, bearing capacity, limiting bearing pressure, presumed bearing capacity

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1 Introduction

With Eurocode 7 (EC7) [17, 18] becoming effective, the former standards for geotechnical design [16] were withdrawn. According to the international practice, the Hungarian standard MSZ 15004-1989 [16] allowed for the use of a so-called „permissible bearing pressure" for preliminary dimensioning, and for foundation design in case of buildings with low importance. With this method, the central, vertical limit load of strip foundations and column footings with a given geometry could be obtained by applying the permissible bearing pressure along with shape and depth factors. In the light of the popularity of this method, and given that EC7 allows for employing design methods based on prescriptive measures, the authors have developed a new calculation method which conforms to the principles of EC7 and which is similar to the former method.

2 Design procedures according to ec7

The EC7 and the literature explaining it [2, 6, 23] clearly define and categorize the possible design procedures for shallow foundations. According to Section 6.4 of EC7, one of the following design methods may be applied.

2.1 Direct method

This is the most accurate, but also the most detailed calculation method, where different analytical models are applied to the different limit states: the ultimate limit state (ULS) needs to be evaluated by accurately modelling the supposed failure mechanism, while the serviceability limit state (SLS) has to be conducted through settlement analysis.

The analytical method for bearing resistance calculation in Annex D of EC7 [17] can be regarded as a direct method in the above sense. That calculation is based on the bearing capacity formula by Meyerhof [15], which is based on the Terzaghi's theory [25]. When a full load-settlement curve until bearing capacity failure is calculated using a numerical FEM-model and the ultimate and serviceability limit states are evaluated based on it, then, that constitutes another direct method.

Table 1 Presumed bearing values (BS 8004:1986)

Soil type	Bearing value (kPa)	Remarks
Dense gravel or dense sand and gravel	>600	Width of foundation (B) not less than 1m. Water table at least B below base of foundation. Susceptible to long-term consolidation settlement
Medium-dense gravel or medium-dense sand and gravel	200-600	
Loose gravel or loose sand and gravel	<200	
Dense sand	>300	
Medium-dense send	100-300	
Loose sand	<100	
Very stiff boulder clays and hard clays	300-600	
Stiff clays	150-300	
Firm clays	75-150	
Soft clays and silts	<75	
Very soft clays and silts	-	

2.2 Indirect method

Results from comparable experience and from field and laboratory measurements and observations are employed, and the design is carried out for SLS loads such that the requirements for all relevant limit states should be fulfilled.

Indirect methods comprise calculations which are based on extensive experience and field measurements, e.g. probing. One example for such an indirect method is the semi-empirical method presented in Annex E of EN 1997-1 [17] that is based on pressuremeter test results and is commonly used in France.

2.3 Prescriptive method

In this case, a so-called presumed bearing resistance is calculated.

EC7 categorizes the foundation design with the presumed bearing resistance as a prescriptive method based on conservative rules. As an example, Annex G of the standard contains a method to derive the presumed bearing capacity of spread foundations on rock [17]. There, an allowable bearing pressure can be calculated for different rock types, using the spacing of discontinuities and the uniaxial compressive strength of the rock.

Furthermore, a number of international guidelines [3, 11, 13, 20, 21, 23] belong to the prescriptive method, which include among others the method based on the „permissible bearing pressure", which was in use until the 1st of January 2011. in Hungary.

3 Presentation and evaluation of prescriptive methods

In engineering practice, the methods for pre-dimensioning play an important role, as they allow quick estimation of the buildings' main dimensions. During the design of larger buildings, the decision about employing shallow or deep foundations has to be made based on simple calculations and scarce geotechnical data. This has in turn an impact on the depth and number of soil investigations, the required quality of soil sampling and laboratory tests, which are used to set up the detailed geotechnical site investigation programme. Furthermore, the simplified methods may also be applied as the final stage in foundation design for buildings with low importance. Before the introduction of EC7 many European standards contained simplified design methods for shallow foundation. [3, 11, 20]

Among others, the British Standard [3] gives values for the presumed bearing resistance applicable for different soil types and conditions. An excerpt from BS 8004 is shown in Table 1. The BS states that these values may only be applied in pre-dimensioning, and furthermore it is required that the groundwater table should lie deeper than B (shorter dimension of the footing) beneath the foundation level – and thus have no effect on the bearing capacity – and the footing should be at least 1.0 m wide (B>1.0 m). As it can be seen, even for the same soil conditions, presumed bearing resistances may vary significantly.

Although, the BS doesn't provide direct values for safety against bearing failure, comparison with bearing resistance back-calculated from presumed shear strength parameters of the corresponding soil classes indicates that the factor of safety is at least three. The standard does neither include corrections that account for a deeper foundation level (depth factor), nor for the footing geometry (shape factor).

A number of cases can be found in the literature, where the permissable bearing capacity already contains provisions to fulfil the requirements in the SLS [11, 13]. In these cases, usually the allowable footing pressures that correspond to a settlement of 1 inch (25 mm) are given. In each case, the authors of these tables stress that the values should only be adopted in the pre-dimensioning phase.

Table 2 contains presumed bearing capacity values presented in the Handbook of Geotechnical Investigation and Design Tables [13]. Again, the author suggests using the values

Table 2 Preliminary estimate of bearing capacity [13]

Soil type	Description	Undrained shear strength (kPa)		Presumed bearing capacity value (kPa)
Clay	Very soft	0-12 kPa		<25
	Soft	12-25 kPa		25-50
	Firm	25-50 kPa		50-100
	Stiff	50-100 kPa		100-200
	Very stiff	100-200 kPa		200-400
	Hard	>200 kPa		>400
Sand	Very loose	I_D < 15%	φ'<30o	<50
	Loose	I_D = 15-35%	φ '=30-35o	50-100
	Medium dense	I_D = 35-65%	φ '=35-40o	100-300
	Dense	I_D = 65-85%	φ '=40-45o	300-500
	Very dense	I_D > 85%	φ '>45o	>500

for pre-dimensioning only. In case of sands and strip foundations, the values should be divided by 1.2, while the effect of the groundwater table in sand should be taken into account by multiplying the values by 0.5. Comparison with the values of BS 8004 shows that safety against failure (with presumed shear strength parameters) is even higher, but this method already takes into account the SLS limit state. If a settlement of 50 mm – i.e. the maximum allowable settlement in Hungary – is acceptable for sands, then the tabulated values found in the handbook may be doubled. In this situation, it is however more appropriate to speak of a presumed bearing capacity rather than about allowable bearing capacity calibrated for the SLS.

The Austrian standard [20] provides diagrams for bearing capacity calculations, while limiting the anticipated settlements to 2-3 cm. However, the standard states that they may only be used in simple situations. It provides bearing capacity for vertically loaded strip foundations without load eccentricity. Soils are divided into two groups, granular and cohesive soils. Using the diagrams in the standard, the calculation of the bearing capacity is quick and straightforward. Such a diagram is shown in Fig. 1, for the case of granular soils. The diagram may be used if there is no layer boundary beneath the foundation level, or the layers are homogenized based on local experience, and if the groundwater table has no effect on the bearing capacity, i.e. if its depth below the foundation level is larger than B.

The values of the diagram may be increased by 30 % if the soil is dense within 2B depth below the foundation level (at least 2 m). An additional 20 % increase may be applied in case of footings with square and circular layout. However, decrease of up to 40 % needs to be taken into account if the groundwater table reaches the foundation level.

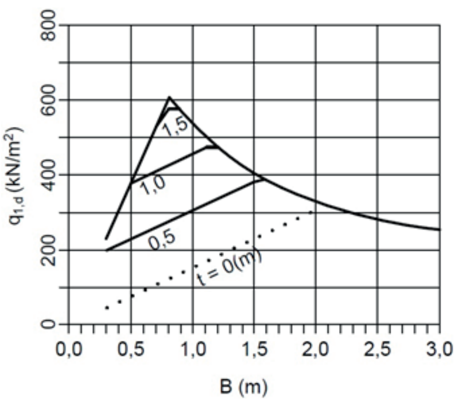

Fig. 1 Bearing capacity of granular soils [20]

This standard may be the only one in the literature which accounts for inclined loads. In this case, the calculated values must be decreased by the factor of $(1-Q_{h,d}/Q_{v,d})2.5$, where $Q_{h,d}$ is the horizontal design load, while $Q_{v,d}$ is the vertical design load.

While a single diagram accounts for granular soils, the standard contains 7 diagrams for cohesive soils, depending on the plasticity index and the consistency of the soil. The application of these is limited to cases where the soil has a stiff, very stiff or hard consistency, or dense/very dense state. The standard emphasised that it does not account for the increase of pore pressure due to rapid loading, and that it must not be used for collapsible soils. The increasing and decreasing factors are similar to those presented for granular soils. The values of the diagrams in ÖNORM B4435-1 [20] show good agreement with the tabulated values by Burt Look [13].

The French national codes valid before Eurocode only recommended simplified shallow foundation design methods based on pressuremeter test, and the national annex of the currently valid EC7 doesn't contains any recommendation regarding this. The currently valid BS EN 1997-1:2004+A1:2013

Table 3 Bearing resistance design values ($\sigma_{R,d}$) of shallow foundations on granular soils with satisfactory safety if conditions set by A 6.3 table are met [10]

The smaller cover depth of the footing m	Design value of bearing resistance $\sigma_{R,d}$ (kPa) B					
	0.50 m	1.00 m	1.50 m	2.00 m	2.50 m	3.00 m
0.50	280	420	560	700	700	700
1.00	380	520	660	800	800	800
1.50	480	620	760	900	900	900
2.00	560	700	840	980	980	980
If the cover depth is 0.30 m ≤ d ≤ 0.50 m and the width of the footing B ≥ 0.30 m	210					

Table 4 Bearing resistance design values (σR,d) of shallow foundations in case of medium silt ($I_P \leq 4$ % and 35% < w_L < 50%), low plasticity clay ($I_P \geq 7$% and $w_L \leq 35$%) and medium plasticity clay ($I_P \geq 7$% and 35% ≤ $w_L \leq 50$%) [10]

The smaller cover depth of the footing m	**Design value of bearing resistance $\sigma_{R,d}$ (kPa) Average consistency**		
	firm	**stiff**	**very stiff**
0.50	170	240	390
1.00	200	290	450
1.50	220	350	500
2.00	250	390	560
Average uniaxial compressive strength $q_{u,k}$ (kN/m2)	120-300	300-700	> 700

doesn't comprise a simplified design method for shallow foundations unlike its predecessor, therefore unfortunately no benchmark is provided. Although, the national annex of the Austrian EC7 doesn't include a simplified method either, but based on our best knowledge its development is in progress, and a new, revised version will be published shortly.

The German DIN EN 1997-1 and DIN 1054 however contains a simplified design method for shallow foundations ("Vereinfachter nachweis in Regelfällen") harmonized with the EC. The tabulated values of bearing resistance design values ($\sigma_{R,d}$) were determined using $\gamma_R = 1.4$ partial factor and were derived based on the EC7 recommended failure theory. Soil properties assumed for the calculations (primarily the shear strength parameters) are unknown, thus they can only be estimated by back-calculating form the tabulated values.

The German code [10] provides the design values of bearing resistance ($\sigma_{R,d}$) in altogether 6 tables, from which the first two concern granular soils, while the rest covers cohesive soils. Within granular soils there is no differentiation between sand, gravelly sand and sandy gravel; in the first table resistance values correspond to 2 cm settlement, while the second table gives values for 1 cm settlement. In case of cohesive soils the maximum settlement is 2-4 cm if values of the tables are used. However it should be noted that practising German engineers propose the performance of a separate settlement calculation for wider footings (2-3 m). The German standard doesn't apply separate shape and depth factors, their effect is taken into account via the tabulated values. For granular soils the values are listed depending on the cover depth (0.5-2.0 m) and footing width (0.5-3.0 m); for cohesive soils (silt, clay with low and high plasticity) the values are given depending on the cover depth and consistency.

Three types of consistency are considered: stiff ($0.75 \leq I_c \leq 1.0$), very stiff ($1.0 \leq I_c \leq 1.3$) and hard ($1.3 \leq I_c$), for which uniaxial compressive strengths are also assigned that can be determined from undisturbed samples.

In the following two tables (Table 3 and 4) examples are shown for granular soils, and for medium silts, low and medium plasticity clays for cohesive soils.

Between the above presented resistance values linear interpolation may be employed. A separate table deals with the conditions upon which $\sigma_{R,d}$ can be increased, e.g. if the ratio of the sides of the footing is (L/B) is less than 2, then values given by Table 3 can be increased by 20 %.

In case of granular soils the depth of the groundwater is taken into account as follows:

1. if the groundwater level lies deeper than the width of the footing (d>B), then the tabulated values are valid;
2. if the groundwater level matches the foundation level, then the tabulated values should be reduced by 40 %;
3. if the groundwater level is between the above two conditions, linear interpolation should be applied;
4. if the groundwater level is situated above the foundation level, than the 40 % decrease of point 2 is only valid if the cover depth is larger than the footing width and at least 0.8 m.

A method similar to the above presented had been developed in 1964 in Hungary, which was later included in the standard for shallow foundations. In Section 2.4, the Hungarian standard [16] "Determination of bearing capacity and settlement of shallow foundations" allowed for the determination of the limiting footing pressure based on tabulated values.

These tables contained the so-called "permissible bearing pressure" for the main soil types (Table 5 and 6). The central, vertical limit load of a strip or spread footing could be determined by using the tabulated values and applying shape and depth factors, depending on the footing geometry and foundation depth.

Table 5 Permissible bearing pressure for granular soils [16]

Type of granular soil, medium dense	dry or damp	moist or wet	saturated, under GWL
	Permissible bearing pressure (σ_a) [kPa]		
coarse and fine gravel	650-780	650-780	520-650
sandy gravel and siltless gravel	580-780	580-780	450-600
medium-grained sand	480-650	480-650	300-400
fine-grained sand	300-400	300-400	200-250

Table 6 Permissible bearing pressure for cohesive soils [16]

Type of cohesive soil	Plasticity index	void ratio	σ_a [kPa]		
	I_p	e	$I_c \geq 1.2$	$I_c = 1.0$	$I_c = 0.5$
silty sand, sandy silt	1-10	0.5	400	350	300
		0.7	300	250	320
silty, sandy clay; low plasticity clay	10-20	0.4	500	400	320
		0.5	420	350	300
		0.7	350	300	200
		1.0	300	200	150
medium and high plasticity clay	>20	0.4	900	750	-
		0.6	720	600	-
		0.8	420	350	200
		1.0	350	250	150

Limiting pressure of granular soils according to the former Hungarian standard:

$$\sigma_{\mathrm{H}} = c_1 \cdot c_2 \cdot \sigma_a \leq 3 \cdot \sigma_a \tag{1}$$

where:

- σ_a permissible bearing pressure,
- c_1 depth factor as (t + B)/2,
- t smallest cover depth above foundation level,
- B width of foundation,
- L length of foundation,
- c_2 shape factor:

$c_2 = 1.0,\ if\ \frac{B}{L} \leq \frac{1}{5}$ in case of strip foundations and elongated, rectangular footings

$c_2 = 2.25,\ if\ \frac{B}{L} = 1$ if the footing is round or has a regular polygonal shape in plan;

$If\ \frac{1}{5} \leq \frac{B}{L} \leq 1$ then values of c_2 may be calculated by linear interpolation.

Limiting pressure of cohesive and macroporous soils according to the former standard:

$$\sigma_{\mathrm{H}} = c_3 \cdot c_a \leq 3 \cdot \sigma_a \tag{2}$$

where

$$c_3 = \frac{2 + t + B}{4}$$

The standard offered a truly quick and simple method for pre-dimensioning, but in some cases the exact values could have been determined only with difficulties. The tables in the standard allowed for a casual soil classification (according to the former – not EC7-compliant – soil classification system) and allocated vaguely defined values to each category, rendering the selection of the permissible bearing pressure an ambiguous task.

The method applies a shape factor to the limiting pressure, with values being in good accordance with other proposals found in the international literature [12, 13, 20]. However, in contrast to other proposals, the bearing capacity of the foundation can be increased considerably through the depth factor by selecting larger footing widths and foundation depths. When analysing the components of a bearing capacity formula, it can be seen that such an increase may indeed be achieved for granular soils. For cohesive soils however increasing the cover depth has practically no effect on the bearing capacity due to the low friction angle; it is predominantly influenced by the value of the cohesion. For cohesive soils according to EC7, when the bearing capacity is calculated for undrained conditions with the undrained shear strength, the effect of cover depth is insignificant.

4 Presentation of the proposed method

The goals set for the newly developed method were twofold. On one hand, the developed method should allow for a quick and simple estimation of the bearing capacity of the soil, according to international examples, the former Hungarian practice, and conforming to the rules of EC7. On the other hand, the flaws found in the former Hungarian method presented above should be corrected. It should aid engineers in the pre-dimensioning of foundations, and in case of favourable ground conditions it should be applicable for the design of smaller buildings or buildings with less loading than usual.

Table 7 Geotechnical input data for the presumed bearing capacity calculations

	Soil types	Condition	γ' [kN/m³]	γ'_{sat} [kN/m³]	φ' [°]	c' [kPa]
GRANULAR SOILS	sandy gravel (Gr>50%)	L	18	9	35	0
		MD	19	10	37	0
		D	20	11	38	0
	gravelly sand (Gr>20%, Si+Cl< 15 %)	L	18	9	32	0
		MD	19	10	34	0
		D	20	11	36	0
	sand (Gr<20% and Si+Cl<15%)	L	17	8	29	0
		MD	18	9	31	0
		D	19	10	33	0
TRANSIENT SOILS	silty sand (Gr<20%, Si+Cl< 40%, Sa>45%)	L	18	9	24	5
		MD	19	10	26	10
		D	20	11	28	15
	sandy silt (60%>Sa>20%, Cl<20%)	L	18	9	20	15
		MD	19	10	22	20
		D	20	11	24	25
	silt (I_p 10-15%)	P	18	9	16	20
		F	19	10	19	25
		S	20	11	22	30
COHESIVE SOILS	low plasticity clay (I_p 15-20%)	F	18	9	14	25
		S	19	10	17	35
		VS	20	11	19	45
	medium plasticity clay (I_p 20-30%)	F	18	9	11	30
		S	19	10	15	40
		VS	20	11	19	50
	high plasticity clay (I_p >30%)	F	18	9	7	30
		S	19	10	12	50
		VS	20	11	15	70

4.1 Geotechnical data

The geotechnical input data was assessed in light of the aims of the proposed new method. The ground investigation report for buildings of less importance is usually based on a smaller site investigation programme. It mainly contains results from small-diameter boreholes, and eventually dynamic probing.

The soil samples obtained from small-diameter borings are usually disturbed samples, the possibility for the retrieval of undisturbed samples is limited. These disturbed samples are of quality classes 3-4 of category B [19] may only be used for soil classification tests (grain size distribution, plasticity index, organic content). Rarely direct shear tests are also carried out, but shear strength parameters are mainly determined from experience or taken from tables based on soil classification.

Through the results of dynamic probing, the density of granular soils may be classified as loose, medium dense or dense. The person conducting the drill also classifies the condition of the soil layers with these categories according to the drilling resistance of the layers. The evaluation of the density of cohesive soils according to dynamic probing results is a more complex task, in such a case it is advised to rely mainly on laboratory tests and draw upon the consistency index.

In ground investigation reports of simpler tasks, the designers will – at least in the Hungarian practice – find the results of soil classification with the description of the layers, their density and consistency, and mainly "tabulated" data on shear strength and oedometric modulus. In light of the above, the authors have assembled Table 7 with soil types and the related characteristic values of geotechnical input parameters for the calculation of presumed bearing capacities.

Soils were classified according to geotechnical practice into three main groups: granular, transient and cohesive soils. The main groups contain three soil types each. For clays, in accordance with the Hungarian geomorphological conditions, physical properties associated with normally consolidated soils were taken into account. The soil types were defined according to the soil classification system of EC7; this allows engineers to match the soils in a ground investigation report to those in Table 7. The abbreviations for the condition are loose (L - I_D=15-35%), medium dense (MD - I_D=35-65%), dense (D - I_D=65-85%), as well as firm (F - I_c =0.51-0.75) stiff (S - I_c =0.76-1.00) and very stiff (VS - I_c =1.01-1.50).

To each of the 27 varieties of soil type and condition in Table 7, the authors have assigned values for bulk density and shear strength, based on literature data and professional experience. For the sake of comprehensiveness and ease of use, but also keeping in mind the approximate nature of the proposed method, simplifications were introduced when assigning the values in the table (especially the unit weights). These may be regarded as characteristical values belonging to the defined soil categories.

4.2 Failure mechanism

To allow for clarity and comprehensibility in each aspect of the proposed method, in its background, application and limits of applicability, a short overview shall be given on the applied mechanism of bearing failure and the analytical calculation of the limit load.

The load bearing capacity of the soil was termed "bearing pressure at failure" in the former MSZ 15004 [16], and the limiting bearing pressure was calculated by dividing it with safety factors, while the EC7 [17] uses the terms "characteristic and design value of bearing resistance". In accordance with the design procedures, there are in principle four ways to establish the limit load of a foundation:

1. By applying experience gained from nearby buildings;
2. Experimentally by load testing;
3. Through analytical methods developed for limit states;
4. Through indirect methods, e.g. using probing results.

From the above approaches, Nr. 3 will be important further on, since the presumed bearing capacities can be determined by applying limit state calculations to derive the bearing capacities.

A number of theories exist, but the limit state analysis based on failure surface is the most widespread [1, 5, 8, 9, 22]. The proposal found in Annex D of EN 1997-1 [17] also belongs to this group of theories, which is based on curved slip surface theory of Terzaghi [24]. These in turn are based on plane strain conditions and uniform pressure distribution at the foundation base. The limit load can be determined depending on the alignment and shape of the slip lines, and the distribution of shear stresses along them.

The main features of Annex D [17] will be outlined shortly because the bearing capacity calculation is influenced by the extension of failure surface primarily in terms of groundwater.

Since the base of the foundation is rough, the friction prevents the soil mass in region I. to slide laterally, and it acts as if attached to the foundation. Point D therefore moves vertically downward, and the slip surface emanates from this point with a vertical tangent. Due to the friction along the line BD, the section of the slip surface between DE is curved, and becomes straight beyond point E (section ED), with an inclination of 45°-φ'/2, with a corresponding passive Rankine state in region III.

It follows Terzaghis theory that the shape of the failure surface depend on the internal friction angle. As the internal friction angle increases, so does the distance at which the slip surface reaches the gound level and the depth below foundation level which it reaches.

When applying this theory, the largest depth reached by the slip surface lies at approximately 0.75·B for φ'=15°, while it increases to cca. 1.2·B for φ'=30°, therefore a uniform depth of B was taken into account in later calculations.

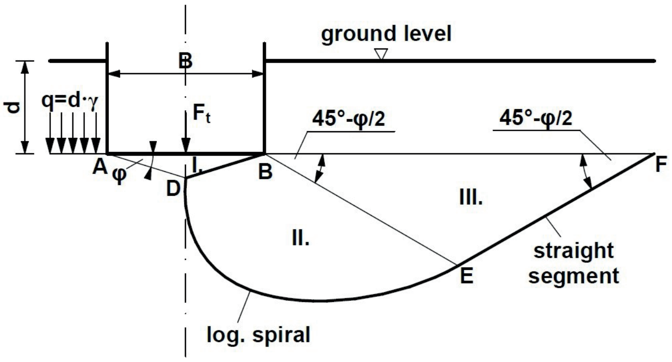

Fig. 2 Determination of the bearing capacity of a strip foundation after Terzaghi's theory

4.3 Calculation of bearing resistances

Keeping an eye on the purpose of the suggested method, i.e. it is intended to be applied in pre-dimensioning and for the design of low-importance buildings, the foundation dimensions and the loading types were selected to match these goals. In the authors' opinion, the foundation width is not expected to exceed B=2.0m for smaller buildings and favourable ground conditions, and the smallest cover depth around the footing is not likely to be larger than d=2.0m. Furthermore, the foundation loads are expected to be central and vertical. If the load is inclined or eccentric, then the slip lines will be distorted which reduces the bearing capacity. This is included in the bearing capacity formula through inclination factors. Inclined loads cannot be handled in a simplified calculation therefore the method presented here cannot be applied in those cases.

The bearing resistance calculations were carried out with each set of the geotechnical parameters given for the 27 soil types in Table 7 (assuming that the soil beneath the foundation level is at least 1.5 times thicker, than the footing width), both for strip foundations and column footings with a square base. First, the characteristic value of the bearing resistance (R_k) was calculated according to Annex D of MSZ EN 1997-1:2006 under drained conditions, then dividing this value by the footing area, the formerly used bearing pressure at failure (σ_t) – a term not used in EC7 – was calculated.

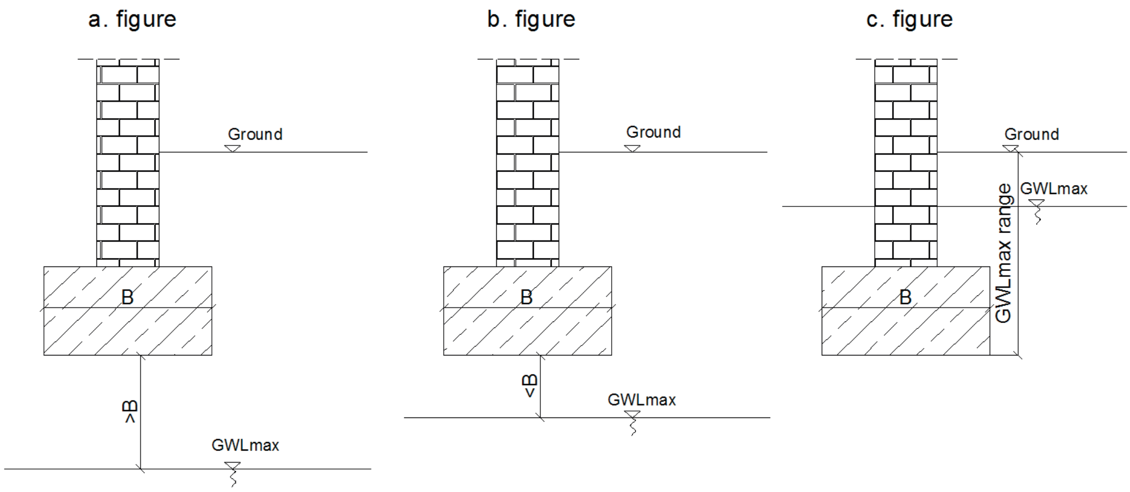

Fig. 3 Assumed positions of the groundwater table in the calculations

The influence of the groundwater level was examined in 3 situations for each soil type (Fig. 3). In the first situation (Fig. 3a) the groundwater has no effect on the bearing resistance since it lies deeper than the footing width beneath the foundation level. In the second situation (Fig. 3b) the area enclosed by the logarithmic spiral slip line is mostly submerged under water. In the third situation the foundation lies completely under the water table, and the weight of the cover is reduced by the buoyancy force. Of course, if the groundwater table lies at the border of the investigated situations presented in Fig. 3, interpolation between the presumed bearing capacity values is allowed.

The initial values of strip foundation width and cover depth were B=1.0m and d=1.0m for the calculation of the characteristic value of the bearing resistance, as in the case of the former "permissible bearing pressure" method.

Taking into account the slight increase in safety on the action side introduced in Eurocode, literature data, and previous Hungarian experience, the safety against failure was set to n=2.25. This can also be interpreted as a model factor of 1.6 besides the partial factor against bearing capacity failure (γ_R=1.4, 1.4·1.6≈2.25). If a partial factor of 1.4 is taken into account as the weighted average of permanent γ_G=1.35 and variable action γ_Q=1.5 on the action side, then global safety becomes approx. 2.25·1.4=3.15, which harmonizes with the international practice and domestic experience.

With the procedure presented above, the design value of the presumed bearing capacity (pressure) was calculated from R_k/2.25, and the resulting pressure values are shown in Table 8, rounded to 25 for the sake of simplicity. The symbol selected for the design value of the presumed bearing capacity (pressure) is $\sigma_{pb;d}$.

It should be underlined that the position of the groundwater table has practically no effect on the bearing resistance for cohesive soils (as can be seen from the values in Table 8), but plays an ever increasing role as the soil grains get coarser. Furthermore, in the formula for bearing resistance consisting of 3 parts, only the bearing capacity factor for cohesion has a pronounced effect on the results in case of cohesive soils, whereas changes in the foundation width or cover depth have almost no effect on the resulting bearing pressures at failure (the characteristic value of bearing resistance).

4.4 Deriving the shape and depth factors

The bearing capacity of a foundation is influenced by its width, the cover depth, and the type of foundation (strip foundation or column footing) [1, 4, 12, 21]. For this end, shape and depth factors have been determined that account for these circumstances. However, opposed to the previous Hungarian method, and conforming better to the structure of the bearing resistance formula, these are different for each of the three soil groups. To determine these factors, the bearing resistance calculations were extended to combinations of foundation width of B=0.5-2.0 m and cover depths ranging between d=0.5-2.0m, both for strip foundations and column footings with a square base.

The main goal for the determination of the shape and depth factors was to find correlations which allow for a maximum deviation of ±10% from the exact values obtained from the bearing capacity formula for the same width and depth. The derived shape and depth factors are shown in Table 9, as a function of foundation width and cover depth, for the three different soil groups.

As it can be seen from the table, compared with the former Hungarian method, the shape factors for column footings have changed a little for transient and cohesive soils (1.3 instead of 1.25), but for granular soils the values may be significantly different depending on width and depth. The depth factor corresponds to the one found in the former method for granular soils (which yields the best fit to the values obtained from the bearing capacity formula). However, the effect of foundation width is smaller in the newly introduced soil group, for transient soils. It is negligible for cohesive soils; hence it does not appear in the depth factor for this soil group. Thus the depth factors resemble well the tendency that as the soil gets more

Table 8 Proposed values for the presumed bearing capacity of strip foundation of B=1.0m, d=1.0m

	Soil types	Condition	$\sigma_{pb;d}$ (kPa) distance between FL and GWL >B	$\sigma_{pb;d}$ (kPa) distance between FL and GWL <B	$\sigma_{pb;d}$ (kPa) GWL above FL
GRANULAR SOILS	sandy gravel (Gr>50%)	L	450	350	250
		MD	575	450	300
		D	725	575	400
	gravelly sand (Gr>20%,Si+Cl< 15 %)	L	300	250	150
		MD	400	325	225
		D	550	425	300
	sand (Gr<20% and Si+Cl<15%)	L	200	175	100
		MD	250	200	150
		D	350	275	200
TRANSIENT SOILS	silty sand (Gr<20%, Si+Cl< 40%, Sa>45%)	L	150	125	100
		MD	250	225	175
		D	350	325	275
	sandy silt (60%>Sa>20%, Cl<20%)	L	175	150	125
		MD	250	225	200
		D	325	300	275
	silt (I_p 10-15%)	F	150	150	125
		S	225	200	175
		VS	325	300	275
COHESIVE SOILS	low plasticity clay (I_p 15-20%)	F	150	150	125
		S	250	225	200
		VS	350	325	300
	medium plasticity clay (I_p 20-30%)	F	150	150	125
		S	250	225	200
		VS	375	375	350
	high plasticity clay (I_p >30%)	F	125	100	100
		S	250	225	225
		VS	375	375	350

Table 9 Shape and depth factors for strip foundations and column footings with square base

Soil group	shape factors (s_{pb})		depth factors (d_{pb})
	strip found.	column foot with square base	
granular	1	$1.3\text{-}0.2 \cdot B+0.1 \cdot d$	$(B+d)/2$
transient	1	1.3	$(B/2+t+2.5)/4$
cohesive	1	1.3	$(d+4)/5$

If $\frac{1}{5} \leq \frac{B}{L} \leq 1$ than values of (s_{pb}) may be calculated by linear interpolation.

and more cohesive, the effect of foundation width and depth on the bearing capacity reduces. When inserting B=1 m and d=1 m into the formula of the depth factor, we get d_{pb}=1.0 for each soil group, i.e. we arrive at the base value.

4.5 Calculation procedure according to the proposed method

With the help of the values of presumed bearing capacities given in Table 8 and the shape and depth factors from Table 9, the bearing resistance of foundations may be estimated. The assumed ground investigation methods, the limited extent of laboratory tests, the experience-based values for shear strength and the simplifications made when deriving the presumed bearing capacities all bear uncertainties, thus the method proposed by the authors bears larger uncertainty than that connected with the characteristic value of the bearing resistance calculated according to EC7 [17]. To account for these, instead of the safety factor for bearing resistance of γ_R=1.4 after EC7 [17], a larger value of 2.25 was adopted – as explained before – for the calculation of the design value of the presumed bearing capacities.

With the help of the values and factors defined above, the bearing resistance of a foundation against central, vertical loads may be calculated and checked against the design value of the vertical, central foundation loads as follows:

$$V_d \le R_{pb;d} \tag{3}$$

$$R_{pb;d} = \sigma_{pb;d} \cdot s_{pb} \cdot d_{pb} \cdot A \tag{4}$$

where:

V_d (kN)	design value of the vertical, central foundation load,
$R_{pb;d}$ (kN)	design value of the presumed bearing resistance
$\sigma_{pb;d}$ (kPa)	design value of the presumed bearing capacity (pressure),
s_{pb}	shape factor,
d_{pb}	depth factor,
A (m^2)	base area of foundation (for strip foundations: B; for column footings with a square base: B^2).

The above discussed method gives similar allowable pressure values on the foundation level as the previously presented correlations of Austrian and currently valid German codes. These pressures are nearly twice as much as the ones found in the Anglo-Saxon literature [3, 13]. But comparison of safety levels between the calculation method proposed by the authors and the ones found in the literature cannot be made unambiguously. In the method of the authors the soil categories, descripted with clearly defined physical soil properties (grain size distribution, plasticity index, consistency index, and relative compaction), can be classified into narrow ranges, and shear strength parameters are also assigned to them. Contrary to this, the other, discussed methods give broader soil classes and physical properties, moreover shear strength parameters cannot be deduced and the global safety against failure is not defined. Analysing the calculation methods' pressure values and their correspondence, safety levels of the different methods can be similar, but use of soil categories characterized by broad limits and vaguely defined physical properties may lead to significant over-dimensioning, or in a worst-case scenario, to insufficient safety.

5 Conclusions

The paper presents a simplified calculation method to estimate the bearing resistance of shallow foundations, according to the principles and rules of EC7 [17].

Before developing the new method, a research was carried out into the background of the previous, similar, Hungarian method and international counterparts. Geotechnical parameters were assigned based on experience to the most commonly found Hungarian soil types, which were later used as input for the analytical bearing capacity calculations. The calculations were extended to different foundation geometries, and possible positions of the groundwater table. It was found that both the tabulated values of the presumed bearing pressures and the modifying factors needed to be adjusted. Furthermore, emphasis has been put on the aspect that the details of the proposed method (soil groups and classification, etc.) should comply with the currently effective standard.

For the calculations, nine soil types were defined and categorized into 3 groups, based on their grain size distribution and plasticity. For each soil type, 3 sub-types were set up, based on compactness and consistency. Geotechnical parameters that are required for bearing capacity calculations were assigned to each soil type and sub-type, based on experience. The effect of the groundwater level was analysed more thoroughly than in earlier methods, by considering three possible situations: deep-lying water table with no effect; water table closely below the foundation level; and a completely submerged foundation. Having carried out the bearing resistance calculations for strip foundations and column footings of different widths and cover depths according to the method in Annex D, section D.4 of EC7 [17], so-called presumed bearing capacities were assigned to each soil type and condition, conforming to the principles of EC7. To assess the effect of geometrical features (foundation width and depth), shape and depth factors have been assigned to each soil group.

The newly proposed method covers a wider range of applicability and provides a more precise estimate of bearing resistance than the previous Hungarian and international methods. Nevertheless, it is still intended to be used only for pre-dimensioning in geotechnical category 2, but in geotechnical category 1 it may also be applied for final design.

It must be stressed that the method is not a substitute for the analysis of serviceability limit states, i.e. settlement calculations. According to our experience, the majority of building damage can be traced back to settlement problems, especially for motions arising from differential settlement. Despite of the fact that using the soil categories given, cover depth, footing width intervals and allowable bearing pressure values, the allowable 4-5 cm settlement of separate footings (according to EC7 annex H [17]) is not expected to be exceeded, the calculation of settlements and differential settlements still needs to be carried out, along with their evaluation as to whether or not they are compatible with the structure.

Besides validating the basis for the application of the proposed design method, the field of application, along with its limits also needs to be clarified. The geotechnical categories introduced in EC7 provide an excellent basis for this, but it is well-known that determining the geotechnical category is in many cases an ambiguous task that requires the close cooperation of the geotechnical and structural engineer.

Finally it is emphasised that the presumed bearing capacity ($\sigma_{pb;d}$) is not a soil mechanical parameter, thus it does not need to be included in a ground investigation report. It has to be determined in the course of the design process, in conjunction with the structural system of the building, the building loads, the sensitivity against settlement, etc.

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