Notes concerning analysis of drilled piers

Remarques sur le calcule des pieux de grand diamètre

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SYNOPSIS Part I, for which is responsible Z.Bažant, treats the evaluation of coefficient of structural strength of soil.Analysis of elastic settlement of drilled piers by linear elastic theory needs to introduce the appropriate value of the active depth in dependence of structural strength of soil. This strength can be found if coefficient of structural strength is known. Verification of this coefficient is possible by measurements of settlement of pier base. - Part II, for *which are responsible J.Šimek and O.Sedlecký, shows a method for evaluation of skin friction from* load test results and draws conclusions for analysis of bearing capacity of drilled piers.

PART I.COEFFICIENT OF STRUCTURAL STRENGTH

Structural strength is equal to the stress induced by applied load, which is not causing a measurable settlement. This is the zero settlement condition which arises when the soil grains compress only elastically, negligibly in compasiron to the settlement caused by the relative movement of grains. Measurement of structural strength in laboratory is difficult because the stress due to the own weight of apparatus may be greater than the structural strength. On the contrary the contact stress acting at drilled pier base may be equal or smaller than structural strength. This allows to determine the structural strength by drilled piers load tests.

When the elastic settlement analysis of drilled piers by linear theory of Poulos /Poulos 1980, Bažant 1979/ was introduced by the writter into the Czechoslovak Standard ČSN 731004 the question emerged of the appropriate value of Young´s modulus of soil to be used in the analysis. It was ascertained /Bažant 1984/ that this modulus is equal to triaxial modulus of deformation, if the assumption is made, that the soil on which the drilled pier is bearing undergoes compression in a thin layer below the pier base only. The thickness of this layer, the active depth, can be evaluated from the condition that the compression of soil arises when the increment of stress in subsoil is greater than the structural strength of soil. Structural strength is defined as the geostatic stress multiplied by coefficient of structural strength. Coefficient is given in literature /Bažant 1984/. However, for complicated geological conditions and great applied loads it is preferable to determine the coefficient of structural strength on site. His value can be obtained from the pier load tests delivering a zero settlement at pier base for small load, usually the first load increment. When the coefficient is known, the length of drilled pier producing a minimum settlement can be obtained.

EVALUATION OF COEFFICIENT OF STRUCTURAL STRENGTH

In the active depth concept the hypothesis is made that the structural strength equals the normal geostatic effective stress σ'_v multiplied by the coefficient of structural strength n. The compression of soil develops in the layer above the plane in which the vertical normal stress σ_z produced by pier head is greater than structural strength. The condition of zero compression is

$$\sigma_z \leq n\,\sigma'_v \qquad /1/$$

Solution of Eq.1 requires the knowledge of the coefficient n which can be found, if the stress is known at which the zero settlement arises.

In the literature /Bažant 1984/ the recommended values of the coefficient n vary between 0.1 and 0.4 according to the kind of soil. These values have been derived from the telltale measurements of displacements in the subsoil below loaded areas of shallow footings. The question may be raised, if these values apply to the subsoil below the drilled pier bases. The more precise values of n are also needed, if complicated geological conditions and/or great loads are involved.

Drilled piers load tests offer the method of finding the coefficient n,a method based on the fact, that the zero settlement of pier base occurs at the beginning of loading. If sufficiently small first load increment is realized, a zero settlement occurs as it was revealed by pier load tests /Reese and al. 1976, Wright and Reese 1979/ at which the instrumentation allowed to measure the settlement of pier base.

To find the coefficient n we insert into Eq. 1 the vertical normal stress

$$\sigma_z' = c_z\, Q_o \,/\, D^2 \qquad /2/$$

where D = length of pier, Q_o = head load at zero base settlement. Q_o should be the maximum load at which the base settlement is zero. If the load rises over Q_o /point b in Fig.3/, the structural failure occurs which is manifested by abrupt settlement, albeit a small one. Coefficient c_z tabulated by Sankaran /1981/ for Poisson´s ratio $\nu = 0.5$ depends on depth z, slenderness ratio D/d, where d = diameter of pier, and $K = E_p/E_s$, the stiffness ratio of Young´s moduli of concrete shaft E_p and of soil around pier E_s /Fig.1/.

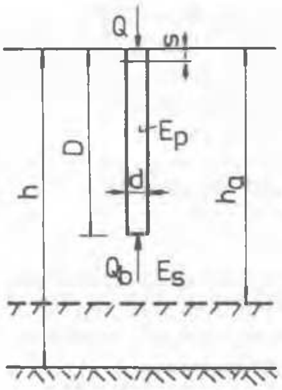

Fig.1. Parameters of drilled pier problem

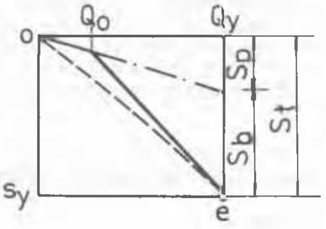

Fig.2. Elastic part of load-settlement curve

Coefficient of structural strength is according to Eq. /1/ and /2/ expressed by

$$n = c_z\, Q_o \,/ (\sigma_v'\, D^2) \qquad /3/$$

The vertical normal effective geostatic stress in a homogeneous soil

$$\sigma_v' = \gamma_d\, z - \gamma_w\, h_w \qquad /4/$$

in which γ_d = unit weight of dry soil, z = depth of the point for which the stress is calculated, γ_w = unit weight of water, h_w = depth of the point under the groundwater table. The measurements of the settlement of pier base s_b requires the instrumentation which is not applied at usual load tests, where only the head settlement s_t is measured. However, in this case it is also possible to find the load Q_o at which s_b is zero from the equation

$$s_b = s_t - s_p \qquad /5/$$

where s_p is the compression of concrete pier shaft /Fig.2/. If $s_t = s_p$, the settlement of base $s_b = 0$. In homogeneous soil it can be assumed that the load Q acting on the head of pier shaft decreases linearly from the head to the tip. When the base load $Q_b = 0$, the compression of concrete pier shaft is given by

$$s_p = 0.5\, Q\, D \,/ (A_p\, E_p) \qquad /6/$$

where A_p, E_p are the area, resp. Young´s modulus of concrete. However, the linear decrease of load is not always assured. The compression of the shaft is greater, when the load is transmitted to the surrounding soil by skin friction in more compressible layer at base. On the contrary, the compression of the shaft is smaler, when the load is transmitted to the soil in cavings situated near the pier head. Therefore in these circumstances one can not skip the measurement of the settlement of base.

If the coefficient n is known, it is possible for given load Q and D to find the relative active depth h_a/D from the graphs published by Bažant /1984/. Knowing h_a/D, one can find out the depth correction factor R_{ha}, and in turn, the Poulos settlement influence factor $I_{sa} = I_1\, R_k\, R_{ha}$. The graphs of factors I_1, R_k and $R_h = R_{ha}$ were published by Poulos to enable the practical estimate of settlement /Poulos 1980, Bažant 1979/. The elastic settlement is given by equation

$$s = I_{sa}\, Q \,/\, (E_o\, d) \qquad /7/$$

When the factor I_{sa} is introduced, the modulus of the layer under the pier base should be used in conjunction.

DRILLED PIERS WITH MINIMUM SETTLEMENT

If the coefficient of structural strength n is verified by the load test, the length of pier D_o can be computed assuring for given load Q the zero settlement at base. Expressing the geostatic stress by

$$n\, \sigma_v' = n\, \gamma_n\, D_o \qquad /8/$$

and inserting σ_z' from Eq. 2 the length of pier follows from Eq. 1

$$D_o = (c_z\, Q / n\, \gamma_n)^{1/3} \qquad /9/$$

Drilled piers having the length D_o produce the contact stress at pier base which is equal to structural strength. If this condition is satisfied, the soil below the base undergoes no compression, the drilled pier is behaving as a column bearing on rock and his settlement derives only from the compression of concrete. Drilled piers of the length D_o are useful on compressible soils of great depth in the case the rock is out of reach when foundations sensitive to settlement should be built.

RESULTS OF COMPUTATIONS

Sand

The drilled pier Q1 in Houston, Texas /Wright and Reese 1979/ is bearing on saturated dense silty sand to fine snad SP, exhibiting N = 32.

The length of pier D = 16.71 m, the diameter d = 0.914 m, D/d = 18 and groundwater level at 11.22 m above base. Coefficient n can be inferred from the head load at zero base settlement Q_o = 401 kN /Fig.3/. Vertical normal stress σ_z in the middle of pier base according to Eq.2 is $\sigma_z = 25 \times 401/16.71^2 = 36$ kPa which equals the structural strength. Coefficient c_z = 25 holding for D/d 18, K = 500 and ν = 0.5 was interpolated between the values tabulated for D/d = 10 and 25 /Sankaran 1981/. Vertical normal geostatic stress at base according to Eq.4 is $\sigma_v' = 20 \times 16.71 - 10 \times 11.22 = 222$ kPa. Introducing these values into Eq. 3 we get the coefficient of structural strength of saturated sand n = 36/222 = 0.16. The load Q_o = = 401 kN fulfills the condition that it is on the verge of equilibrium before the failure of structural strength starts /Fig.3, point b/.

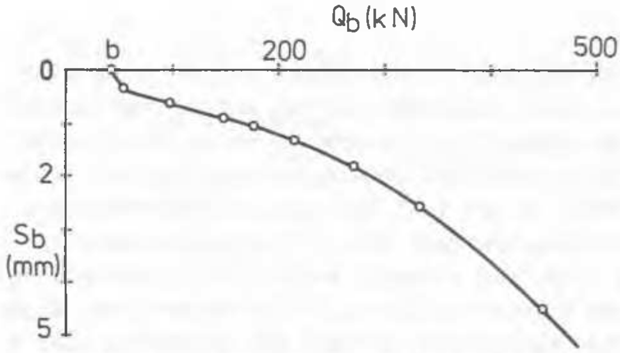

Fig.3. Settlement at base of pier Q1

Gravelly sand

The drilled pier T1 of Honshu-Shikoku Bridge, Japan /Takahashi 1981/ is bearing on saturated very dense gravelly sand having N = 40 to 50. The pier has the extreme length of D = 70 m and diameter d = 3 m, D/d 23. Groundwater level is 63.5 m above base. The greatest load attained at the test is 40 000 kN. Even if the pier is not bearing on the rock, his settlement is minimum as if it was so. From the graph of permanent settlements the guess can be made that the zero base settlement is achieved for head load Q_o = 30 000 kN. For interpolated c_z = 33 the vertical normal stress is σ_z = $= 33 \times 30\,000/70^2 = 202$ kPa. The ensuing load Q_b = 1 436 kN is apparently below the sensitivity of measurement. Vertical effective geostatic stress $\sigma_v' = 21\text{x}70 - 10\text{x}63.5 = 835$ kPa Coefficient n = 2o2/835 = 0.24. The length which assures zero settlement at base follows from Eq. 9 as $D_o = (33\text{x}30\,000/0.24\text{x}21)^{1/3}$ = = 58.13 m. The actual length of 70 m is greater than needed.

Clay

The drilled pier S1 in Houston, Texas /Wright and Reese 1979/ is bearing on fissured clay CH. The length D = 7.05 m, d = 0.76 m, D/d = 9 and groundwater level 2.47 m above base. Head load at zero base settlement Q_o = 107 kN. Vertical normal stress for c_z = 10 is σ_z = $= 10\text{x}107/7.05^2 = 22$ kPa. Vertical normal effective geostatic stress $\sigma_v' = 20\text{x}7.05 - 10\text{x}2.47$ = = 116 kPa. Coefficient n = 22/116 = 0.19.

CONCLUSIONS

Under the assumption that the limit Q_o was securely ascertained computations of the coefficient of structural strength have given the following results:

/a/ saturated dense silty sand n = 0.16
/b/ saturated very dense gravelly sand n = 0.24
/c/ fissured clay n = 0.19

For these kinds of soils the value advocated in literature /Bažant 1984/ is n = 0.2. The agreement is satisfactory and it holds for drilled piers of lengths between 7.05 m and 70 m and of diameters between 0.76 m and 3 m.

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PART II. BEARING CAPACITY

The influence of shaft resistance on the bearing capacity has been known since the beginning of pile foundation.Its significance has considerably increased owing to the introduction of technologies of cast-in-place piles. According to the results of in situ loading tests the shaft (skin) resistance equals 60-80% of the overall bearing capacity. The exception is represented by the point-bearing piles supported at the bottom by rocks. The results of more then 300field loading tests consider: the activation of shaft resistance, the coefficient of earth pressure on the pile, the active zone around the pile, the distribution of the vertical and horizontal stresses, the values of the limit stage coefficients etc. The activation of shaft resistance occurs even under a minor settlement of the pile,even if this one is less than 1mm. Such a small displacement of pile is possible even under small loading of the pile top or even in the case of slightly compressible subsoil under the bottom. According to our expiriences may be considered rock, weak rock with the modulus of deformation more then 500 MPa. A further phenomenon bas been observed in the case of piles concreted directly in the boring - the shear area is formed arround the pile shaft. The soil in the immediate surroundings of pile surface hardens due to the concreting and the shear area is at a distance of 20-50 mm from the actual pile surface (Fig.4).During the application of pressure grouting, during separate concreting (prepac system) the shear area is situated essentially at a greater distance. In the case of permeable soils and weak rocks their strengthening is achieved by the use of cement grouting or by grouting suspension. In the case of less permeable soils or impermeable soil the strengthening during concreting and grouting is due to the hydratation of the concrete or of the cement suspension. The other case is the protection of pile shaft by PE or PVC foils. Essentially it may be said that the technology has a considerable influence on the bearing capacity of boring piles. During the observation of individual soil strata, deformations increase in the direction toward the pile surface and become apparent.

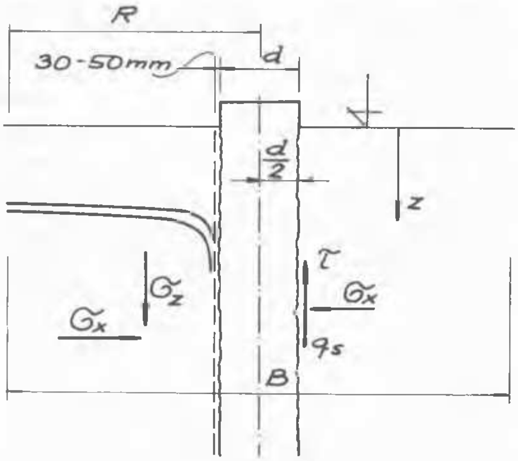


Figure 4. The active zone around the pile

The distance from the pile centre is denoted as R and designated as the active zone around the pile.It is the area to which the effects of the vertical loading of the pile are transfered in the soil and also the deformations. For non-cohesive soils it is approximatelly R = 4d, for cohesive soils R = 3d, where d is the diameter of pile. It is evident that a greater deformation of soil and therefore also vertical stress was measured in the vicinity of the surface.According to the measurement the deformation and the vertical sterssσ_z were zero at the distance delimited by the boundary of active zone. The other measurement was for the estimation of the horizontal stress σ_x to receive the value K, the coefficient of earth pressure on the pile (Fig.5a,5b). The value is generaly used as a constant even if actually is not. The pile surface is rather uneven,this value can reach the coefficient of passive earth pressure. Some authors consider coefficient at rest K = 1 - sinφ. From the tests carried out in the laboratory and in the field different values of K were obtaind, but never less than K = 1 and never K = 1 - sinφ. These results, testing of vertical and horizontal contact stresses on the shaft and at the bottomof piles were the bases for the equations for determination of the bearing capacity of large diameter boring piles. These types of piles are the main technology of deep foundation in Czecholovakia, where about 200 000 meters of piles are realised every year.

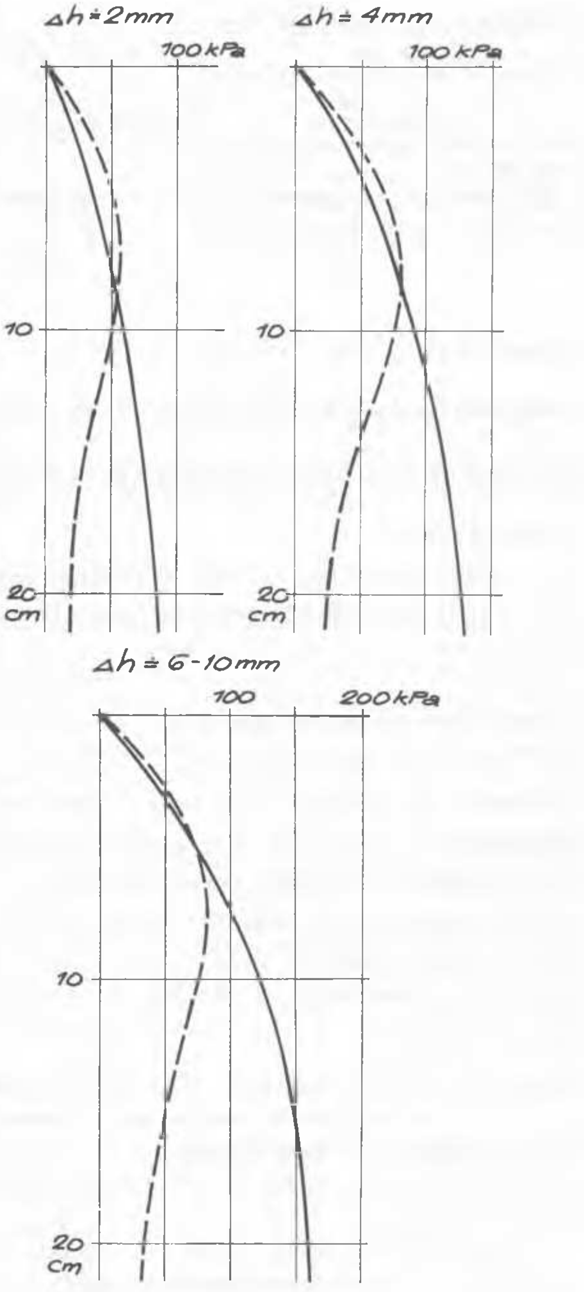


Fig.5a The influence of vertical displecement Δh for vertical and horizontal stresses (in laboratory).

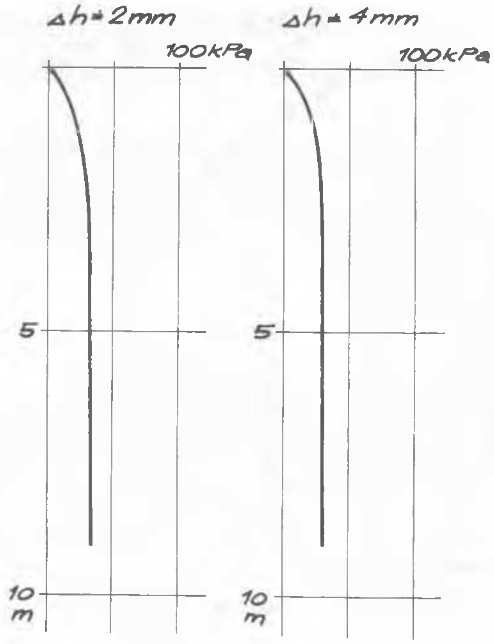


Fig.5b The influence of vertical displacement Δh for horizontal stresses on pile.

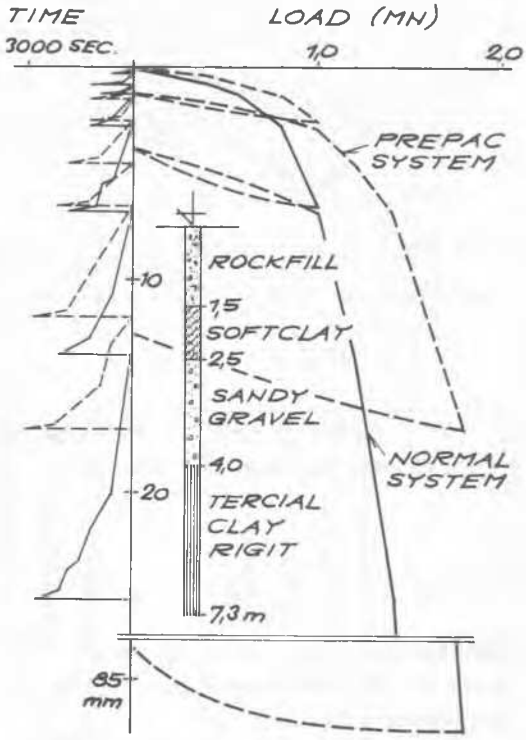


Fig.6 Loading diagram

The equation (10) takes into account many main influences, technology, protection by PE or PVC foils,theory of limit stages, activation of shaft resistance, system of concreting etc. Fig.6,7 shown the loading diagram for normal system of conreting and for prepac system and loading diagram for single pile and pile group.

PERMISSIBLE BEARING CAPACITY

Permissible bearing capacity according to the Regulation for foundations of multi-storied slab concrete structure (Šimek 1975) is

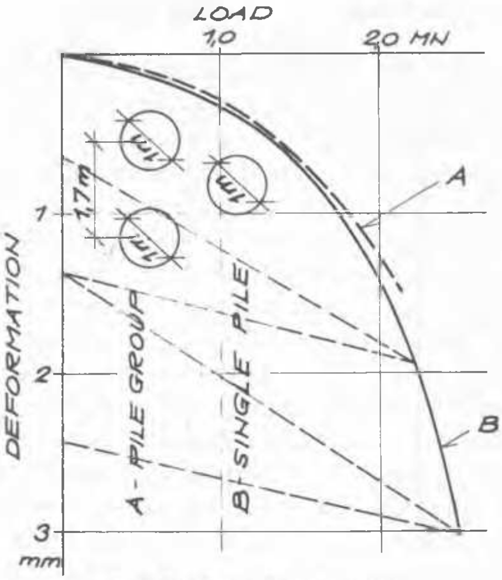


Fig.7 Loading diagram

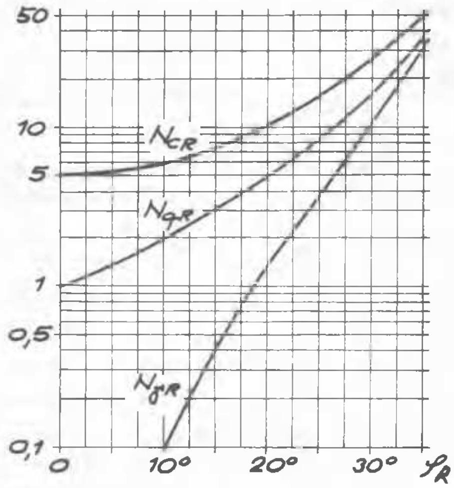


Fig.8 Bearing capacity factors

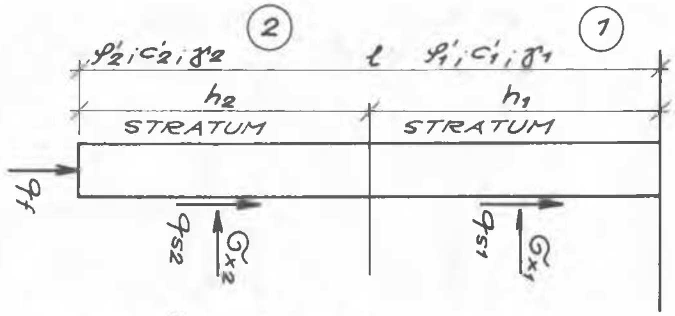
$$Q_p = 1{,}3\ A\ q_f + u \sum_{i}^{n} h_i\ q_{si} \qquad (10)$$

q_f...resistanceat the bottom of pile
A ... area of the cross-section of pile
u ... perimeter of pile
h_i... thickness of the i-stratum of subsoil
q_{si}.. unit shaft resistance of the i-stratum of subsoil

$$q_f = 0{,}5\gamma_{1R}\ N_{\gamma R} + \gamma_{2R}\ l\ N_{qR} + c'_R\ N_{cR} \qquad (11)$$

or

$$q_f = 0{,}1\ q_d \qquad (12)$$



$$q_{s1} = m_3 \gamma_1 \frac{h_1}{2}\ tg\,(\alpha_1 m_1 \varphi_1') + m_2 c_1'$$

$$q_{s2} = m_3 \left(\gamma_1 h_1 + \gamma_2 \frac{h_2}{2}\right)\ tg\,(\alpha_2 m_1 \varphi_1') + m_2 c_2'$$

$$q_f = \frac{1}{2} m_3 \gamma_2 \alpha N_{\gamma R} + m_3 (\gamma_1 h_1 + \gamma_2 h_2) N_{qR} + m_2 c_2' N_{cR}$$

Fig.9 Equation (10) for determination permissible bearing capacity

Coefficient of limit stages are :
$m_1 = m_3 = 0{,}9$ and $m_2 = 0{,}5$
It means $\varphi_R = m_1 \varphi$, $c'_R = m_2\ c'$, $\gamma_R = m_3 \gamma$
where φ', c', γ are the soil properties from the engineering geology investigation.

In the equation (10) and (11) are
l ... total length of pile
$q_{si} = \sigma_{xi}\ \tan\ (\alpha \varphi_{iR}) + c'_{iR}$
$\sigma_{xi} = \sigma_{zR}\ K$
σ_{zR} ... reduced geostatic effective stress in the centre of i-stratum of subsoil
α... technology coefficient

- $\alpha = 0{,}8$ for using the bentonite suspension,
- $\alpha = 0{,}8$ for using PE or PVC foils (thickness max 0,25 mm),
- $\alpha = 0{,}9$ for using steel tubes,
- $\alpha = 0{,}9 - 1$ for separate concreting (grouting, prepac system),
- $\alpha = 0{,}6$ for using PE or PVC foils (thickness over 0,25 mm),

In the equation (10) the coefficient of earth pressure on pile is supposed to be K = 1. For the settlement over 25 mm it may be K = 1,2.