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On pile and piled raft footing settlement analysis

Sur pieu et pieu-plaque fondation tassement analyse

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ABSTRACT
The paper describes FEM simulation analysis to justify the concept of pile footing settlement as the sum of the "solid block" settlement and individual piles "punch-through" settlements

RÉSUMÉ
L'article decrit MEF simulation analyse pour supporte le princip de fondation de pieu tassement comme un somme de tassement de "bloc solide" et "pénétration" tassements de pieux individuelles

1 INTRODUCTION

The paper discusses the analysis of large spread footings, sitting on multiple (up to thousands) piles, pile footings (pile fields) with high and low raft. The latter one may be defined as combined pile-raft footing.

Such footings are used for heavy structures in order to reduce mean and differential settlements rather than to ensure high bearing capacity safety margin (the bearing capacity is high enough anyway thanks to large size and depth of the footing). On the other hand, due to pile interaction via soil, bearing capacity of individual piles in the pile group is much higher, therefore, the approach, proposed by M.Randolph (1994) and based on deformations (settlements) analysis instead of the conventional bearing capacity analysis, appears to be more appropriate.

For the analysis of plates on soil base Schwartz iterative algorithm is proposed, (see Fedorovsky & Bezvolev, 2000), which can be applied to any soil foundation analytical model and consists of the following sequential steps:

1. A certain initial Winkler ratio k distribution under the plate is assigned.
2. The plate analysis is carried out (with or without upper structure stiffness taken into account) for the specified load, and plate settlements w and soil reactions p are determined.
3. Determination of soil settlements w for the selected analytical model by soil reactions p, calculated in i.2. Recalculation of the Winkler ratio k distribution $k = p/w$.
4. Comparison of the new and the previous Winkler ratio. If the difference between the two is small enough then the analysis is finished. Otherwise, return to i.2.

This algorithm can be applied to piles or pile fields, especially for the case of closely spanned piles, when it is correct to average pile and soil reactions under the raft. Large-span piles must be analyzed individually (see Aleksandrovich et al, 2003). In any case pile footing settlements analysis is a must.

There are two approaches to such analysis. The first one is based on single piles settlement analysis and determination of pile group settlement with pile interactions via soil taken into account (Poulos & Davies, 1980; Levachev et al, 2002). Within the limits of this approach Barvashov & Fedorovsky (1980) developed 3-parametric contact model of pile field.

The second approach, widely applied in engineering practice (Tomlinson, 2001), is based on assumption that the actual pile footing is a certain conventional "solid block" footing on soil base. The first approach is more appropriate for small pile groups, but it is invalid for large pile groups (pile fields), where pile-soil interaction is substantially different from the case of single pile, e.g. distribution of soil resistances between the pile shaft and the pile tip, and also skin friction distribution along the pile shaft (see below). The second approach does not take into account pile distribution within the group, however, the latter can largely influence the settlements.

2 SETTLEMENT OF SOLID PILE-SOIL BLOCK

A more accurate approach combines the advantages of both above-mentioned ones i.e., pile footing settlement (mean and local) is defined as the sum of two components:

$$w = w' + w'', \tag{1}$$

where w' is the settlement of a solid block footing within the footprint of the actual pile footing; w'' is the additional settlement due to individual pile punching.

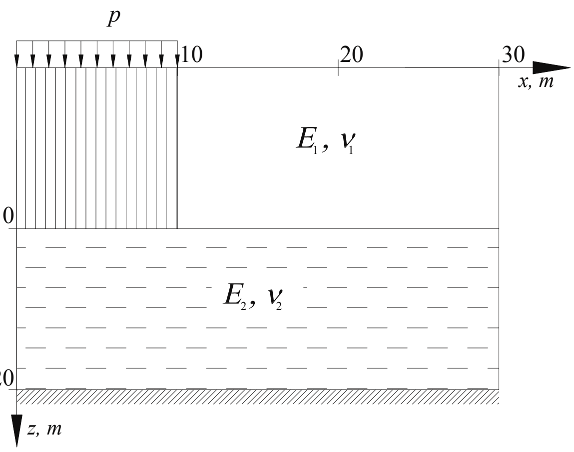

Fig. 1. An anisotropic solid block on the two-layer soil

Below are given methods for calculating the above settlement components. For the simplicity and due to the fact that footing behavior is far from failure, the calculations were conducted in terms of elasticity theory and for the case of massive plane footing in 2-D terms.

Consider a strip 20 m wide and 10 m deep pile field (i.e. a pile length l is equal to 10 m), located on two-layer elastic soil, rested on a stiff base (Fig. 1). The solid block footing i.e., pile field, is a transversally isotropic elastic body, whose vertical deformation modulus can be obtained by averaging pile and soil moduli

$$E_z = \frac{E_p F_p + E_1 (A - F_p)}{A}, \qquad (2)$$

where E_p is Young modulus of the pile material; F_p is a pile cross section area; E_1 is an upper soil layer deformation modulus; A is a soil base area per one pile. For the square pile grid $A = s^2$, where s is the pile span.

All other elastic parameters of the solid body are identical to those of the upper soil layer.

Three soil bases were analyzed: homogeneous ($E_1 = E_2 = 25$ MPa); slightly non-homogeneous (E_1 = 17.5 MPa, E_2 = 35 MPa) and strongly non-homogeneous (E_1 = 15 MПa, E_2 = 75 MПa). Poisson ratios for all cases $\nu_1 = \nu_2 = 0.25$. A thickness of the lower layer is taken as 10 m. Similar bases were investigated by Aleksandrovich et al (2003), and their common point is that for the infinite plate, loaded uniformly, the Winkler ratio is k_0 = 1500 kN/м^3.

Load the anisotropic massif with uniform surface load p = 0.5 MPa. Due to the symmetry it is sufficient to analyze just a half of the footing-soil system. Because the massif features high stiffness only along the vertical plane, and the shear in vertical plane depends on soil parameters, the settlement of the massif top is non-uniform (or rather parabolic): the settlement of the center w'_c is about 50% greater than that at the footing boundary w'_e (Table 1).

Table 1. Anisotropic massif footing settlement under uniform load 0.50 MPa

E_1, Mpa	E_2, MPa	Analytical settlements, mm		
		w'_c	w'_e	w'_m
25	25	143.9	96.1	128.0
17.5	35	112.0	70.9	98.3
15	75	56.5	34.6	49.2

This table also gives mean settlements of the top of the massif, calculated with the help of the following equation:

$$w'_m = \frac{2w'_c + w'_e}{3} \qquad (3)$$

Notably, the model in question possesses considerable distribution capacity. For homogeneous soil base Winkler ratio increases from 3475 kN/m^3 in the center to 5200 kN/m^3 at the edge, while for non-homogeneous soil base from 8850 kN/m^3 to 14450 kN/m^3.

3 INDIVIDUAL PILE PUNCHING

In order to calculate contributions of punching we shall use a model, proposed by Fedorovsky & Bezvolev (1994) for analyzing infinite pile field settlements. I.e. consider a cylindrical volume, having $R = \sqrt{A/\pi} = s/\sqrt{\pi}$ radius, symmetrical about the pile axis (see Fig. 2a). The volume includes the pile and the surrounding soil all the way down to the necessary soilbase depth. The load can be applied both to the pile (in the case of high raft) and to the raft top (in the case of low raft), whose part within the cylinder is also included in the analysis. This very case is analyzed below. Boundary conditions on the cylinder side surface are evident due to symmetry considerations: zero lateral (radial) displacements and shear stresses.

The solution of such a axisymmetrical problem yields the settlement of the infinite pile field. As to the finite pile field, it is better to evaluate the additional settlement due to punching by means of the following expression:

$$w'' = w''_1 - w''_2 \qquad (4)$$

Here w''_1 is footing top settlement in the initial analytical model, shown on Fig. 2a; w''_2 is idem if the pile and the soil within the pile length are replaced by a homogeneous massif with equivalent overall stiffness with modulus E_z (2) (Fig. 2b). As in the lower soil base portion (at depths, exceeding $2R$ from the plane at the level of pile tips) stresses and deformations are practically identical for both models, the lower soilbase layer can be limited by the above-mentioned thickness.

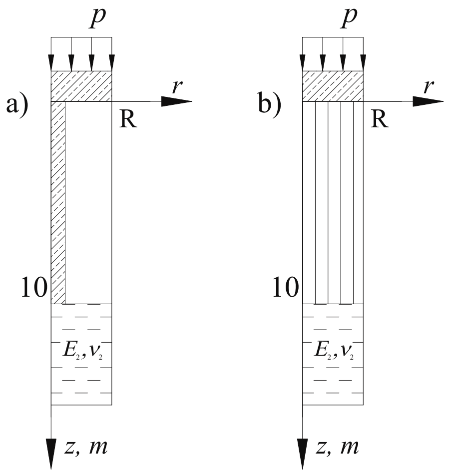

Fig. 2. Models for calculating pile "punch-through" settlement

Table 2 shows the results of such analysis. The soil base is the same as above. The pile is a 33.8 cm dia concrete pile (with circular cross section area equivalent to that of the standard square 30×30 cm pile). Its material modulus is E_p = 30 GPa.

Table 2. Dependence of additional pile settlements due to punching versus pile span, caused by 0.5 MPa pressure on low pile raft

E_1, MPa	E_2, MPa	w'' settlements (mm) versus pile span s, m					
		1.0	1.2	1.5	2.0	3.0	6.0
25	25	5.4	9.4	14.4	22.9	38.8	60.1
17.5	35	9.9	15.2	23.0	35.7	60.2	98.3
15	75	9.7	14.7	21.7	35.4	62.1	116.3

Table 3. Total mean settlements of pile raft footing

E_1, MPa	E_2, MPa	$w_m = w'_m + w''$ (mm) depending on pile span (m)						w'_{cf}, mm
		1.0	1.2	1.5	2.0	3.0	6.0	
25	25	133.4	137.4	142.4	150.9	166.8	188.1	140.2
17.5	35	108.2	113.5	121.3	134.0	158.5	196.6	133.7
15	75	58.9	63.9	70.9	84.6	111.3	165.5	107.0

Now it is possible to calculate the total, local and mean pile raft footing settlements. As the bending stiffness of the raft was not considered in the analysis, the results in Table 3 relate to the mean settlement for comparing with the results of conventional "solid block" pile footing analysis. w'_{cf} settlement analysis for conventional footing was also conducted by means of FEM.

The "solid block", as per recommendations in Tomlinson (2001), was assumed to be $(2/3)l$ = 6.67 m deep and 23.33 m wide. The "solid block" dimensions here did not include pile penetration in the "bearing layer", because there is no clear definition what it is.

An evident assumption is confirmed that the conventional "solid block" footing method essentially underestimates settlement variation with respect to pile span even without plastic component of settlement taken into account.

4 ELASTOPLASTIC BEHAVIOUR

The above analytical models are also valid for elastoplastic soil base. We omit multiple relevant questions and just give some analytical results for "solid block" clay soil base with the following parameters: deformation modulus E = 25 MPa; specific gravity γ= 20 kN/m^3; angle of internal friction φ = 15°; cohesion c = 50 kPa. The pile is the same as in the previous example. Friction parameters on soil-pile interface were assumed the same as for soil, i.e. the pile is perfectly rough.

Fig. 3 shows skin friction profiles (soil resistances) on pile side surface for three different types of loading. Their common feature is that in all cases pile settlement is equal to 5 cm i.e., it is great enough for total friction mobilization on a single pile. This case is illustrated by curve 1 i.e., the analysis was carried out for the above-described "cylinder" model with large pile span (s = 6 m). In this case it was purely pile footing i.e., high pile raft same as for curve 2 case, which corresponds to small pile span (s = 1 m). It is evident that soil response largely concentrates near the pile point and there is no skin friction for the most part of the shaft, where soil and pile settlements are equal. There are just small shear stresses close to the soil surface. Notably, soil compression in the lower pile portion, where τ is not 0, forms up above-mentioned additional pile punching settlement.

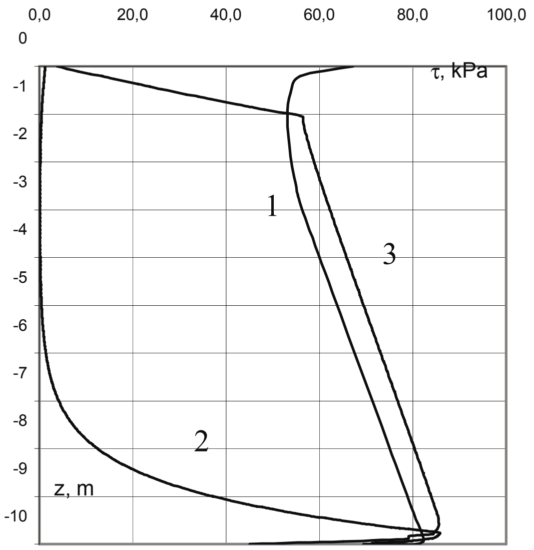

Fig. 3. Curves of skin friction along the pile shaft for the single pile (1), the pile from a field with a span s = 1 m (2) and for the single pile from pile-raft footing (3)

Curve 3 corresponds to raft-on-piles footing with s = 6 m. Under the raft, where settlements are close to pile settlements, shear stresses are small and drop down to zero at z = 0. Below they reach the limit, and, since the pressure under the raft increases vertical and horizontal stresses in soil, the limit shear stresses, linearly depending on radial ones with $\tan\varphi$ non-zero, also increase, as compared with the single pile case (curve 1). It is known that the above-described increase of the side friction as well as the tip resistance due to the pressure under the raft leads to practically unlimited growth of the pile bearing capacity in combined pile-raft footings (Hanisch et al, 2002). Aleksandrovich et al (2003) proposed to calculate the side friction for piles included to such footing by the method of τ-w curves with the relative pile-soil displacement as w.

5 PRACTICAL EXAMPLE AND CONCLUSIONS

As an example the proposed approach was applied to the settlement prediction for a residence house, that has been built in Moscow in 2003. The house includes two separate many-storeyed buildings, of 34 and 23 storeys. Both buildings have pile field footings from piles of length 21 m and 18 m, respectively, with the pile span s = 1.4 m. These piles are bored DPT-piles. DPT (Discharge Pulse Technology) consists in a compaction of a liquid concrete and the surrounding soil by means of microexplosions induced by electric discharge pulses.

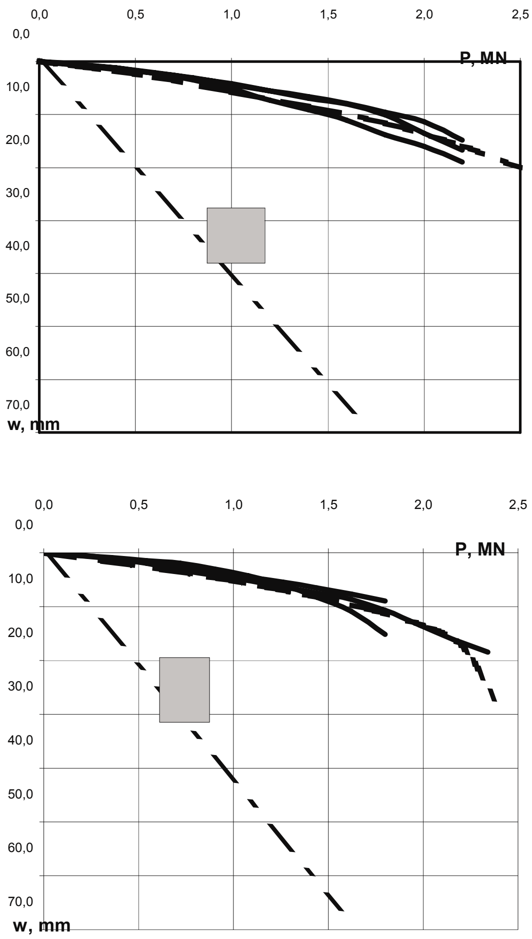

Fig. 4. Curves load per pile – settlement for single piles (experimental ——– and analytical – – –) and pile field (– - ·– - –). Gray rectangle shows a range of loads and settlements for 34-storeyed (a) and 23-storeyed (b) buildings

The pile diameter d = 338 mm was adopted for the analysis. The soil base was assumed two-layered for simplicity. A dense lean clay with design parameters γ' = 21.9 kN/m^3; E = 32 MPa; φ = 22º; c = 33 kPa presents an upper layer 17 m in thickness. A dense water-saturated fine sand with design parameters γ' = 11 kN/m^3; E = 48 MPa; φ = 45º; c = 3 kPa is located below. The calculations were performed by means of FEM-code PLAXIS using an elasto-plastic soil model with isotropic hardening (see Brinkgreve & Vermeer, 1998).

Notice that considered piles have the very high bearing capacity, especially the tip resistance. So a working load per pile in footing is limited by the pile material strength. Fig. 4a displays experimental curves load-settlement for single pile, obtained in the static load pile tests at the greater building site, and the respective analytical curve. There is also shown the analytical curve mean load per pile – mean footing settlement for the pile field. A gray rectangle shows an appraisable range of loads (0.9…1.1 MN per pile) and a measured range of settlements (28…38 mm). Analogous curves are shown in Fig. 4b for the smaller building. Here appraisable pile loads falls within the range between 0.6 and 0.8 MN and measured settlements – between 18 and 31 mm.

As evident from Fig. 4 qualitative results of the analysis agree satisfactorily with the experiment. As regards quantitative results, they essentially depend on the adopted soil model and, especially, its parameters.

We would like to conclude by stating that the proposed method can be applied along with Schwartz algorithm for the case of variable pile span and non-uniformly loaded footing.

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