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A design method for the 'reticulated pile structure' for the stabilization of slopes and excavations

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ABSTRACT: Sliding slope stabilization can be carried out adopting the so-called "reticulated-pile structure". This design solution is also adopted to support high excavations, when no other "traditional" solution is applicable (e.g. anchored diaphragm walls). The soil is reinforced by micro-piles both owing to the formation of a continuous "nucleus" of improved characteristics and by the presence of the piled structure which reacts to the applied loads. The accurate design of the reticulated-pile structure is a very difficult task, owing to its statically indetermination and to the difficulty in the soil-piles interaction analysis. The paper reports a rational approach to design such a structure, already used in many real applications. The method allows to evaluate the stress-strain states of the whole structure and of the single elements which compose it. Analyses have been performed with the aim of pointing out the effects of design factors on the general behaviour of this structure.

1. INTRODUCTION

The stability of a sliding slope can be achieved by installing piles which penetrate the firm sub-soil underlying the sliding mass. Several investigators have faced this kind of procedure (e.g. Esu & D'Elia 1974, Ito & Matsui 1977, Fukuoka 1977, Viggiani 1981, Carrubba et al 1989, Poulos 1995); nevertheless an established and rigorous design approach is still to be developed.

According to Viggiani (1981) the stabilization of a slope via the installation of piles involves three main steps:

1. evaluating the total shear force needed to increase the safety factor to a desired value. This step can be approached by means of classical stability analyses;
2. evaluating the maximum shear force that each pile can support to resist sliding of the unstable mass. In this case the response of piles to the laterally moving soil involves the evaluation of the mechanism of failure under different conditions;
3. selecting the type and the number of piles in the most suitable arrangement within the slope. Engineering judgement is often adopted, after having considered the outcomes of steps 1. and 2.

Almost all the approaches already quoted refer to large diameter vertical bored piles, arranged in one row.

A different approach of facing the problem of sliding slope stabilization is to consider the so-called "reticulated pile structure" (Lizzi 1978); in this case micro-piles are generally used; they are executed with different rakes and are connected by a rigid cap. The construction gives rise to the formation, inside the soil mass, of a continuous "nucleus" of improved characteristics, causing small disturbance to the potentially unstable blanket.

The use of the reticulated pile structure (RPS) for the stabilization of slopes or to reinforce soil masses where large excavations are planned, dates back over twenty years ago (Lizzi 1970, Berardi 1974, Cantoni et al 1989). Nevertheless theoretical or analytical studies are very scarce as well as the experimental verifications. The accurate design of the RPS is infact a very difficult task, owing to its statically indetermination and to the difficulty in the soil-piles interaction analysis. A rational approach, already established and used in many real applications (Berardi & La Magna 1984), is reported in the paper, updated and improved, especially in the simulation of the interaction between piles and reinforced soil. Analyses have been performed with the aim of pointing out the effects of design factors such as the number of piles, their arrangement and their rake. In this context, and referring to the three design steps, this paper is mainly focused on point 3., even if the method allows to evaluate the complete stress state in the piles that form the RPS.

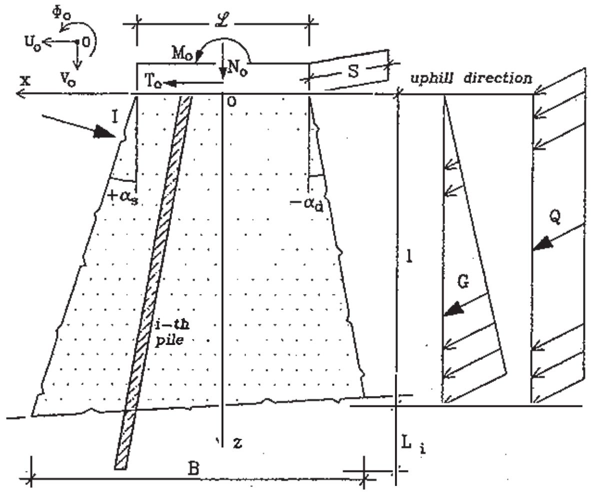

Fig. 1 General reference scheme

2. DESIGN PROCEDURE

The RPS is basically constituted by a nucleus of soil, reinforced by the piles, with improved mechanical characteristics in comparison with the original ones, connected to the underlying stable subsoil.
The general hypotheses that form the basis of the proposed procedure are the following (Fig. 1).

- the RPS is a massive structure, along its base a natural stabilizing reaction R_ϕ ,due to friction , is mobilized. This reaction can be increased by proper ground treatments.
- A stabilizing reaction R_c , due to cohesion, can be mobilized, according to the natural and/or improved soil characteristics.
- The micro-pile construction techniques (e.g. grouting applying pressure) gives rise to a rather continuous nucleus.
- The RPS is connected to the stable mass by N piles (i =1 to N), which are the reinforcement of the nucleus and that resist to the shear forces along the base.
- Considering the deformability of the pile section (in the part of length l) as a structural element reinforcing the nucleus, the stabilizing forces, transmitted by the soil, are taken into account.
- The pile section, having length L_i , embedded into the stable mass, is assumed as a structural element subjected to the reactions arisen along and normally to the pile axis and due to the displacement in the same direction.

The structural analysis is performed with the "displacement method"; the N piles are fixed in the rigid pile cap. The degree of indetermination is equal to 3(N-1). By the general equilibrium system (see App. I) the horizontal and vertical displacements (Uo and Vo) and the rotation (Φo) of the center of the pile cap are computed. The system is written by means of the 3N compatibility equations, expressed using the stiffness coefficients (three conditions for each pile support).

The external forces and moment I, N_o, T_o and M_o, G and Q, are indicated in Figure 1.

The pile having diameter d_i is the structural element for which (Figs. 2-3):

- the reactions in the node B_i are $\mathcal{U}_i$, $\mathcal{V}_i$ and $\mathcal{M}_i$;
- along l_i the earth pressure resultants Q and G are shared out in the forces P_{ji}, while the external forces I in the I_{ji} . Along l_i the resistant forces $R_{\phi ji}$ and R_{cji} also act; all the above mentioned forces depend on the position of the i-th pile and of the j-th force within the depth l_i;
- the i-th pile is supported, at the lower end, by the entire part L_i : the pile-soil interaction is taken into account by means of a Winkler-type model, (Matlock & Reese 1960), considering the coefficient of horizontal subgrade reaction K_h (spring C_R in Fig. 2; see App. II);
- the displacement η_{Ai} of the node A_i , along the pile axis, depends on the stiffness C_p of a spring at the pile tip and of springs C_c simulating the deformability due to the forces along the shaft. This leads to

$$\eta_{Ai} = \beta V/C_p + (1\text{-}\beta)V/C_c \qquad (1)$$

being V the axial force in A_i and β an empirical coefficient, depending on the hypothesis assumed for the distribution of the load along the pile. Both the parameters C_p and C_c can be evaluated as functions of K_h.

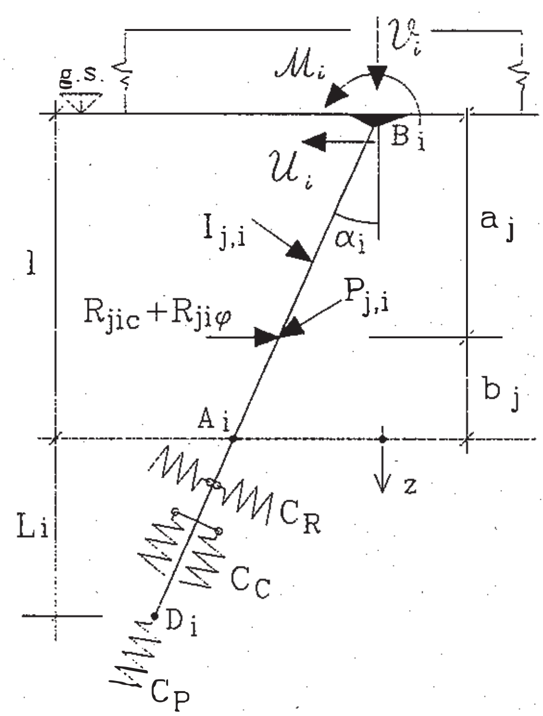

Fig. 2 Reference scheme for the i-th pile

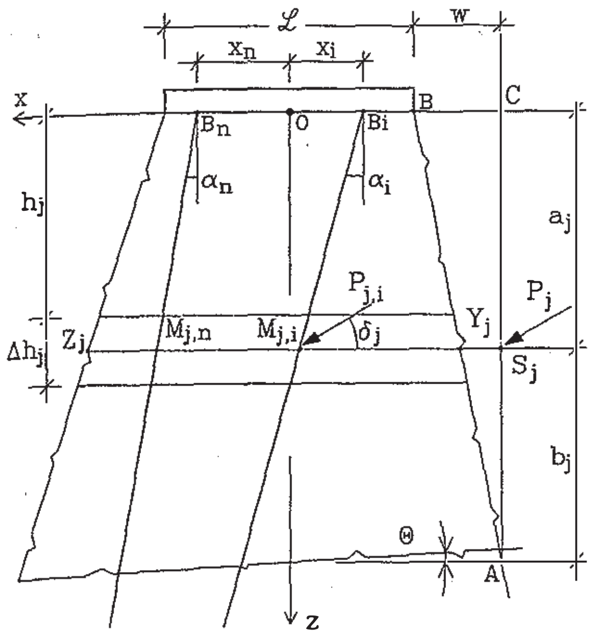

Fig. 3 Scheme for the loads sharing between the piles

The hypothesis of a Winkler-type behaviour simplifies notably the calculations; on the other hand a continuum model could allow a more realistic modelling of the interaction phenomena.

As far as soil modification is concerned, the method allows to take into account improved mechanical parameters, both in terms of soil strength (γ, c, ϕ) and in terms of the stiffness of the pile cross-section (E·A and E·J).

The active pressures having as resultants the forces Q and G give rise to the force P_j , that acts at the depth a_j (Fig. 3). As previously noted this is shared out between the N piles as forces P_{ji} , depending on the stiffness supplied to the i-th pile by the soil in front of it (zone Z_jM_{ji}).
This sort of "shadow effect" allows to state that the nearest is the pile to the applied load (zone $M_{ji}S_j$), the higher will be the applied stresses. This criterion leads to:

$$Z_jM_{ji} = (\frac{\mathcal{L}}{2} - X_i) + a_j(tg\alpha_s - tg\alpha_i) \qquad (2)$$

$$P_{ji} = \frac{P_j[(\frac{\mathcal{L}}{2} - X_i) + a_j(tg\alpha_s - tg\alpha_i)]}{\sum_{i=1}^{N}[(\frac{\mathcal{L}}{2} - X_i) + a_j(tg\alpha_s - tg\alpha_i)]} \qquad (3)$$

The stabilizing forces R_ϕ and R_c, taken into account considering partial factors of safety F_ϕ and F_c, are applied at the interface (z = 1 ; Fig. 1), along the base of the improved nucleus having breadth

$$B = \mathcal{L} + l(tg\alpha_s - tg\alpha_d) \qquad (4)$$

The forces R_ϕ and R_c result:

$$R_\phi = 0.5\,\gamma\,(B + \mathcal{L})\,l \cdot S \cdot tg\phi \cdot cos\theta \qquad (5)$$

$$R_c = c \cdot S \cdot B \cdot cos\theta \qquad (6)$$

The soil parameters can be considered the original or the modified ones as well.

The resistant forces are shared out, on the i-th pile, depending on an influence interval Δh_j and, moreover, following the same criterion as the forces P_j.
The solution is so obtained from the Eqns. (7) (8) (9). (App. I).

Obviously the simplicity of the solution is only formal; it is infact very difficult to express the stiffness coefficients related to the supported section in B_i, when B_i is considered one end of the beam A_iB_i (Fig. 2). At the lower end the support is the entire element A_iD_i, embedded into the sub-soil, and all the forces acting along l_i have to be taken into account. It is impossible, due to space limitation, to report here the expressions of these coefficients as well as the complex procedures used to obtain them.

Once the unknowns Uo Vo Φo have been computed, the support reactions in B_i , the stress-strain states along the pile, the loads applied to the soil, can be derived.

3. ANALYSIS OF THE BEHAVIOUR OF THE RPS

The geometry of a RPS can assume several configurations, especially for the rake of the piles; the proposed approach allows to analyze the behaviour only in the plane (xz), Fig. 1, while a real structure is likely to have piles raked in plane and out of plane.

The influence that some factors have on the choice of the more effective geometrical configuration is pointed out in the following , having performed analyses varying some characteristics of the considered RPS's.
They are constituted (Fig. 4) by a rigid cap having dimensions $\mathcal{L}$ x S = 3x3 meters, N = 8 - 12 - 16-piles (n = 2.67 - 4 - 5.33 piles/m, respectively), diameter d = 22 cm. The piles rake ranges from 0° to 20° (downhill direction Iv) and from 0° to 30° (uphill direction Im) .
The sliding soil mass extends to a depth l = 8 m from the ground surface; it has unit weight γ = 19 Mg/m^3, cohesion c = 15 kPa, friction angle ϕ = 22°.

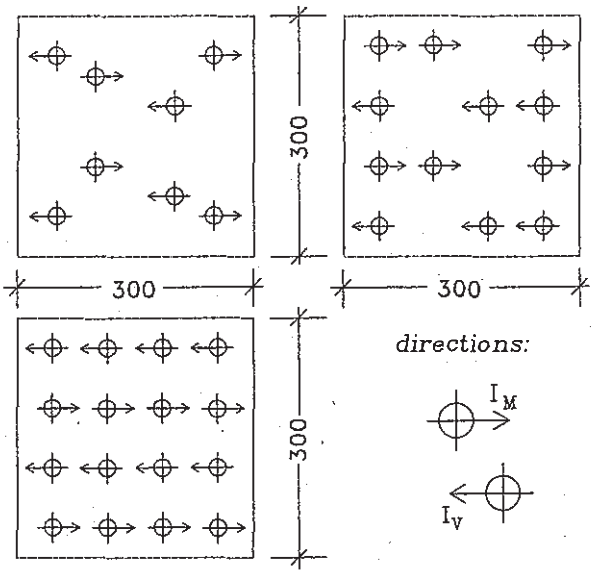

Fig. 4 Lay-out of the considered RPS's

The piles are embedded to a depth of 5 m into the stable soil; the horizontal subgrade reaction coefficient is assumed to vary linearly with depth from a value K_{ho} = 80 N/cm^3 at the sliding interface to a value K_h = 470 N/cm^3 at L = 5 m (gradient n_h = 17 N/cm^3).

The coefficient β (Eqn. 1) is assumed = 0; no anchors have been considered (forces I) to highlight the influence of the number and the rake of the piles. In this context particular relevance assumes the case when Iv = 0; this situations may happen if the RPS is used to reinforce and retain a high excavation, when other design solutions (i.e. anchored diaphragm walls) are not allowed.

The Figs. 5 -6 -7 report some of the obtained results, in terms of the horizontal displacement Uo and of the bending moment on the piles.

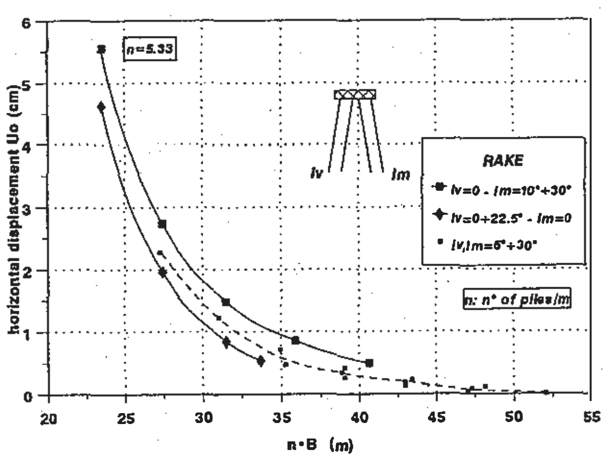

Fig. 5 Horizontal displacements of the pile cap

Analyses previously performed (Berardi & Berardi 1995) have already shown the influence of the number of piles; obviously when it increases the displacements and the stresses decrease with an asymptotic trend.

In Figure 5 the horizontal displacement Uo is reported vs. the factor n·B , as far as the larger considered RPS is concerned; it is possible to observe that passing from arrangements having (Iv ≠ 0 ; Im = 0) to (Iv = 0 ; Im ≠ 0) the horizontal displacement increases, and the principal factor in the limitation of the displacement is assumed by the increase in the downhill rake Iv (" rafter effect ").

The analysis of the results in Fig. 6 leads to the following remarks:

- the horizontal displacements decreases when the uphill rake Im increases;
- it is convenient to rake the piles towards the uphill direction ("tie-back effect "): if Iv = 0 larger inclinations are required;
- raking the piles in the two directions (Iv, Im >0) allows to introduce fewer piles;
- when Iv = 0 and Im = 10° ÷ 20° (common design parameters used in excavations), it is necessary to create a " resisting cage " of many reinforcing piles well penetrating into the stable soil.

Analogous considerations can be done as far as the maximum bending moment on the piles is concerned; In Fig. 7 some results are resumed in relation to the bending moment Mm(o) at the fixed end of the rear piles raked uphill: for these, infact, in that section, the moment has always been the maximum one. As a consequence, the knowledge of the distribution of the stresses between the piles allows, in the design phase, to optimize the geometry of the RPS.

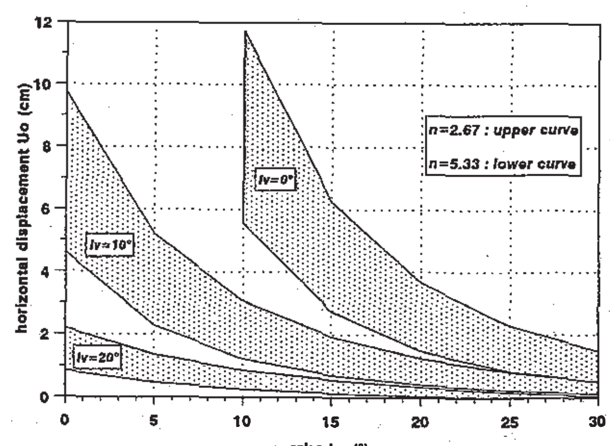

Fig. 6 Influence of the pile rake

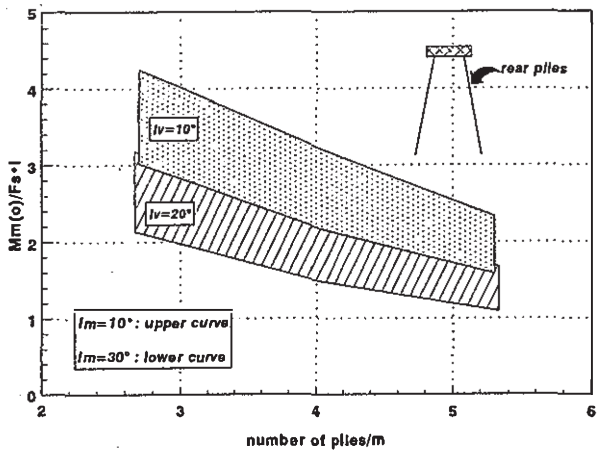

Fig. 7 Envelope curves of the normalized bending moment

COMPARISON OF THE RESULTS

As it has been already pointed out, there is scarce evidence of experimental measurements to allow a comparison supporting the effectiveness of the proposed method.

Since the relevance of the piles arrangements, on the performance of the RPS, has stood out, the comparison has been performed referring to a well documented numerical study (Poulos & Davis 1980), Poulos & Randolph 1983).

The six groups of piles shown in Fig. 8 have been analyzed; even if they differ from the adopted scheme assumed for the RPS, the method can be used assuming a very small value of the sliding soil thickness (l) and considering proper parameters for the "stable " soil.

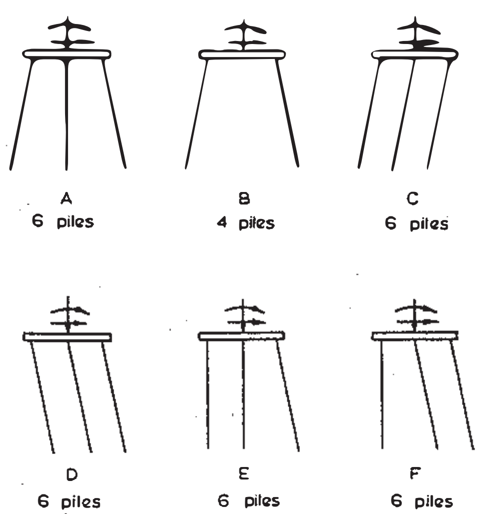

Fig. 8 Configurations of the pilegroups (rake = 15°) (from Poulos & Davis 1980)

The parameters supplied by Poulos and Davis are:

- L = 10 m
- d = 0.4 m
- E_s = 7000 kPa (soil modulus)
- E/E_s = 1000 (moduli ratio)
- No = 1200 kN
- To = 400 kN
- Mo = 600 kNm

From these data the other input quantities have been derived. In particular the soil has been considered mechanically homogeneous, with a constant subgrade reaction coefficient K_h = 40 N/cm^3, obtained as a function of the pile radius and of the modulus Es (Tornaghi 1974).

The comparison between the results supplied by Poulos and Davis (1980) and the ones obtained with this approach, are illustrated in Fig. 9 , as regards the pile cap displacements.

The comparison shows a good agreement between the two methods, although they are basically different.

The comments already made in relation to the effectiveness of a piled structure with a " rafter effect " as regards one with a " tie-back effect " are confirmed.

CONCLUSIONS

A rational procedure to design complex piled structure such as the "reticulated pile structure" has been established. The method allows to take into account piles-soil interaction through simple hypotheses and well known soil models.

The required soil parameters to perform the analyses are few and the usual ones the designer can rely on.

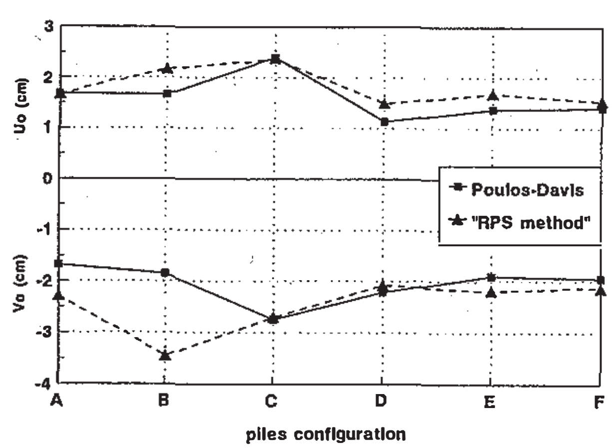

Fig. 9 Comparison of the results

Although the procedures to get the solutions are rather complicated, the computer code is very fast and easy to handle, being the method based on a close-form approach.

The knowledge of the stresses and of the displacements allows to check the limit states of the soil , of the structure and of the single elements that compose it.

The performed analyses have shown the influence of various factors on the more effective geometrical assembly of a reticulated pile structure.

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APPENDIX I

Equilibrium equations: $\left(\Sigma = \sum_{i=1}^{N} \right)$

$$\Phi o \cdot (\Sigma m_{\Phi\Phi} + \Sigma V_{V\Phi} \cdot Xi + \Sigma m_{\Phi V} \cdot Xi + \Sigma V_{VV} \cdot Xi^2) + Vo \cdot (\Sigma m_{\Phi V} + \Sigma V_{VV} \cdot Xi) + Uo \cdot (\Sigma m_{\Phi U} + \Sigma V_{VU} \cdot Xi) = Mo - (\Sigma M_{Pi} + \Sigma R_{Pi} \cdot Xi) \quad (7)$$

$$\Phi o \cdot (\Sigma V_{V\Phi} + \Sigma V_{VV} \cdot Xi) + Vo \cdot \Sigma V_{VV} + Uo \cdot \Sigma V_{VU} = No - \Sigma R_{Pi} \quad (8)$$

$$\Phi o \cdot (\Sigma U_{U\Phi} + \Sigma U_{UV} \cdot Xi) + Vo \cdot \Sigma U_{UV} + Uo \cdot \Sigma U_{UU} = To - \Sigma T_{Pi} \quad (9)$$

APPENDIX II

Variation of horizontal subgrade reaction coefficient with depth z:

mode 1: $K_h(z) = n_h \cdot z/d$ (10)

mode 2: $K_h(z) = K_{ho} + n_h \cdot z/d$ (11)

mode 3: $K_h(z) = K_{ho}$ (12)