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CONTENTS OF VOLUME II

II. Structural Mechanics. Structural Engineering. Earthquake Engineering

The correct choice of the structure may significantly affect the mentioned factors. It is necessary to pay attention to the designing of details in the structure to allow for the redistribution of influences and prevent the collapse of the structure. The safety factors adopted in designing provide the reserve in dimensions of the elements, which may satisfy the limit states of bearing capacity and serviceability and increase the integrity. The corrosion of concrete elements can considerably accelerate failure of the structure, unless the damaged elements are rehabilitated during the maintenance..

This paper analyzed the structure with equal spans in both orthogonal directions, and the redistribution is uniform. The further analysis could comprise analysis of redistribution where the spans are different in two directions, or by including the failure of some additional elements.

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COMPARATIVE ANALYSIS OF THE METHOD FOR DETERMINATION OF PILES FOUNDATION LATERAL LOAD CAPACITY

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Abstract: *In this paper a single pile and pile groups (batter piles) under horizontal loading are considered using finite element method (FEM) and approximate methods. Both rigid and flexible pile is treated for single piles: Some of the approximate methods, and among them the Winkler method, are applied to the calculation of piles and pile group they are described and the results are discussed. The typical value for subgrade reaction k (soil modulus) is used and it depends on the soil conditions. In analysis the value in the range 5000-30000 kNm^{-3} was used for cohesive soil. A comparative analysis of those results with those obtained by more precise methods was done. Particular recommendations are formulated based on those results.*

Key words: *Piles foundation, lateral load capacity, approximate methods, FEM*

1. Introduction

Piles are often exposed to lateral forces caused by wind, earth pressure or earthquakes. Most of existing theories dealing with the dynamic behaviour of soil-pile system assume perfect contact between pile and soil. However, during strong motion soil surrounding the piles behaves nonlinearly causing the separation between soil and pile [1]. This problem leads to necessary analyses of the material nonlinearity of the soil as well as of the geometrical nonlinearity arising due to separation. In design practice this method is not appropriate, because the goals of designers are to determine deflections and stresses in the selected soil/pile system that they may be controlled within tolerable limits [2]. In general, laterally loaded piles can be divided into two groups: short piles and long piles. Most piles are relatively flexible and can be analysed as if they are infinitely long. Only short rigid piles are likely to require consideration of the lower boundary conditions in an analysis. The goals of designers are to determine deflections and stresses in the soil-pile system so that they may be controlled within tolerable limits [3]. For instance, horizontal load capacity may be limited by: ultimate soil capacity which if exceeded results in very large horizontal movements and foundation failure; bending moments which may generate excessive bending stresses in the piles resulting in their structural failure; pile head deflections. The primary function of the piles is to provide resistance

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Analysis of piles under lateral loading, which is characteristically of the force of earthquakes, is a rather complex task. The most widely used method is the finite element method (FEM). However, for the purposes of conceptual design the approximate approaches that are recommended in technical regulations are also commonly used. In this paper, the behaviour of piles under horizontal loading is examined using the FEM, with its adjustment to conditions of pile-soil interactions [4], [5] and approximate approaches [6]. A pseudostatic approach for estimating the deformation behaviour and internal forces of single piles and pile groups subjected to seismic excitation was presented [7]. The method is capable of accounting for both inertial and kinematic effects (required EN 1998).

2. Methods of single pile analysis

The lateral load on long pile can be analyzed using the concept of subgrade modulus or considering soil as an elastic medium. Idealize the soil like a spreading medium or the continuum elastic half space, which allows the study of the mutual interaction among piles through the soil. Soil strength can be defined from the coefficient of subgrade reaction between load and deformation y, i.e. $p=k\,y$, where k is the coefficient of subgrade reaction. A simplified approach consists of replacing the soil adjacent to the pile by a Winkler-type springs. According to the Winkler model, an elastic medium (soil) can be replaced by a series of infinitely close, independent elastic spring. Broms [7] developed a simplified solution for laterally loaded piles based on the assumption of: shear failure on soil, which is the case of a short pile, and bending of the pile governed by plastic yield resistance of pile section, which is applicable on a long pile. The idealised soil stiffness profiles are: constant stiffness assumed for consolidated clay, parabolic curve typical for sand, and linear stiffness for soft clay [3] and [4]. An extensive review of application *p-y* relationship in cohesionless soil is given in [8], and for analysis of laterally loaded piles in soft clay is given in [10]. The broadest analysis of different piles is presented in [11], and for the rigid pile in [12].

A laterally loaded pile can be calculated using FEM by modelling the pile with beam elements and using bars to represent the uniaxial soil resistance. Standard beam and beam - bar FE program can be used to design a laterally loaded pile (Fig. 1). A bar can be represented using a beam element and supporting the translational degrees of freedom. Using the FEM, the standing pile with fully fixed head that is built into clay soil, described by the Winkler's coefficient of soil reaction, is computed similar as in [8] and [9]. The constant value of soil reaction is adopted both excluding the surface layers of soil due to plasticization and without excluding the plasticized zone. These solutions were compared with the results obtained using the approximate methods (equivalent cantilever method Fig. 2) that are used in engineering [5] and [6]. A wider explanation of those procedures was given in [4]. Relative stiffness factors for constant (left) and for linear stiffness profiles (right) are:

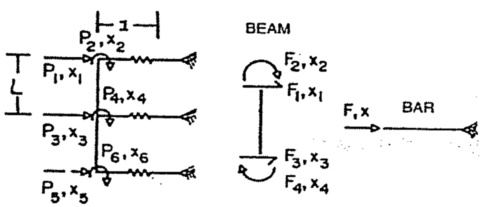
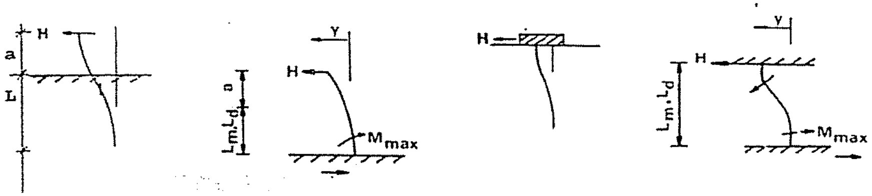


Fig.1. FEM formulation and internal force deformation notation [9]

$$(2.1)\quad R_c=\left(\frac{E_pI_p}{k}\right)^{1/4};\quad R_c=\left(\frac{E_pI_p}{n_S}\right)^{1/5},$$

where $k=k_sD_p$; [kN/m²]; D_p is the diameter of the pile in [m]; $n_S=\dfrac{k_SD_P}{z}$ [kN/m³].



$$M_{max}=H(L_m+a);\; y=\frac{H(L_d+a)^3}{3EI}\qquad M_{max}=\frac{HL_m}{2};\; y=\frac{HD_d^3}{12EI}$$

Fig.2. Case for equivalent cantilever method, [4] and [6]

3. The results and their analysis

The bending moments and horizontal displacement for the piles built into coherent and cohesionless soil was computed applying the described method (equivalent cantilever method) and the expressions for computing. For the purpose of numerical analysis, a pile having a length of 10m and a circular base of 60cm diameter was taken (L=10 m, D=60 cm). Several models were analyzed under horizontal loading, H=10kN, that acts at soil level, and in case of horizontal loading that acts beneath the soil surface (including M). Analyses of pile emended into soil for varying stiffness. The value of soil subgrade was varied in range from 5000 to 30 000 kN/m³. Values of maximum bending moment in pile (Models 1 and 2) were calculated using the software Tower (Panel pro), based on FEM [4].

The length of finite element is 1m, and the springs by which the soil is modelled were concentrated in the nods. The soil is modelled as having constant stiffness in its depth. In the first model of spring calculation, there is a 50% of stiffness in soil level in regard of spring stiffness in the soil. The pile in this model is a standing one, and it is fixed in its base. This model has been calculated according to several methods whose results are provided further in the text. The stiffness of soil in models 1) and 2) is constant. In the second model of calculation according to the FEM the reduction of bearing capacity was carried out, the soil level spring is excluded and the stiffness of the other spring is corrected according to exclusion of the upper zone of soil in depth to 1.5D (90cm), so the other spring has 60% of stiffness of other springs in the soil (Fig.3). The pile is a standing one and is fixed in its base. Models 3, 4, 5, 6 and 7 were calculated according to the equivalent cantilever method. Models 4 and 7 are restrained, and models 3, 5 and 6 are freely supported in head of the pile. On model 5 the force has eccentricity beneath the soil of a=1m. In models 6 and 7, the soil is introduced with linearly variable stiffness, that is, sand.

When soil reduction is introduced into model 2), and not in model 1), the maximum moment in pile, for the given example, is increased depending on soil quality, from 48 to 81%. With the reduction, the maximum deflections in case of FEM are increased from 54 to 98%. The results for various values of K are given in tables for the variation of the value K from 5000 to 30000 kN/m³ (Tab. 1), according to [10] and the soil modulus Es (Tab. 2) is introduced in the calculation, according to SAP 2000 (Tab. 3). In Tab. 3 are presented the values of the stiffness factor Rc and relation L/Rc as a rank of the pile (rigid smaller than 2, and flexible greater than 4 [3]. The values in Tab. 3 are calculated using Ec=24.86 GPa (Mmax=12.22), and diagrams using 30.0 GPa (Mmax=12.81). It would be more adequate, for the low values of Es to introduce a linear variation by depth, similar to sand, instead of constant rigidity [13]. This would yield a more realistic result for values K' from 3000 to 9000 kN/m.

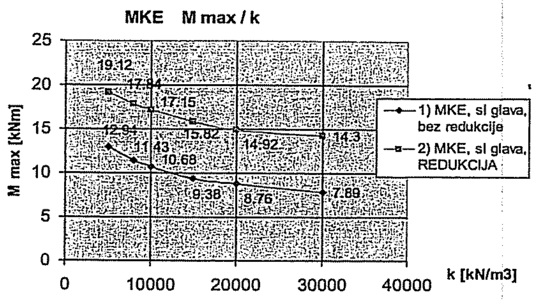


Fig.3. Diagram 1 FEM comparison of results of M_{max} with reduced and not reduced stiffness of the upper 90 cm of soil (1.5D)

Table 1. Clay – constant rigidity by soil depth along the pile

K[kN/m3]	5000	8000	10000	15000	20000	30000
K'[kN/m]	3000	4800	6000	9000	12000	18000
Mmax[kNm]	12,94	11,43	10,68	9,38	8,76	7,89
U [mm]	1,6	1,12	0,95	0,7	0,56	0,41
Rc=	2,7	2,4	2,27	2,05	1,91	1,72
L/Rc=	3,70	4,17	4,41	4,88	5,24	5,81

Table 2. Results according to Budhu & Davis [10]

Es[kN/m2]	2730	4368	5460	8190	10920	16380
K'[kN/m]	3000	4800	6000	9000	12000	18000
Mmax[kNm]	8,65	7,61	7,16	6,41	5,93	5,31
U [mm]	1,51	1,03	0,86	0,61	0,49	0,35

The soil elasticity model is determined according to the approximate formula: $E_S \approx k \cdot b(1-\nu^2)$ even though:

$$K = \frac{E_S}{B(1-\nu^2)}$$

(3.1) $$k = 0.65 \cdot \left(\sqrt[12]{\frac{E_S b^4}{E_1 I_1}} \right) \cdot \frac{E_S}{1-\nu^2} \quad \text{(A. Vesić)}$$

Table 3. Model 1) SAP2000

Load	1	2	3	4	5	6
Mmax [kNm]	12,22	10,65	9,89	8,89	8,27	7,39
U [mm]	1,69	1,19	1,01	0,74	0,59	0,43
B(kN)	1,92	1,21	0,9	0,43	0,18	**-0,05**

In Tabl. 3 B(kN) is the horizontal reaction in the pile base with fixed support. The minus operator means that the reaction has an opposite direction in regard to the active force at the top (diagram Tr. Force intersects 0 twice, i.e. changes the sign). For the practical purposes, it is a long pile. The results in Tables contain maximal bending moments and deflections Y. Calculation, with Ec=24.86 in SAP give Mmax=18.52; in Panel pro give Mmax=19.12 kNm.

Longitudinal reinforcement 6#6 (6R19.1mm), transversal reinforcement. #3/15cm (9.5mm/15cm).

In order to analyse the behaviour of the pile under higher degrees of load, a calculation according to the *non-linear theory* was conducted, whereby the piles behave in a non-linear manner, since the plastic joints were introduced, and the linear behaviour was adopted for the soil. The load was from 200kN, increased to 300 kN, and then to 400 kN, and the results are presented in Tab. 4. Under force of 200 kN reach yield. Under force of 300 kN appear plastic hinge and collapse (See Appendix).

Table 4. Displacement under increased horizontal force (nonlinear analisys)

K[kN/m3]	5000	8000	10000	15000	20000	30000
K'[kN/m]	3000	4800	6000	9000	12000	18000
H=200[kN]	H=200[kN]	H=200[kN]	H=200[kN]	H=200[kN]	H=200[kN]	H=200[kN]
U [mm]	35.03	21.82	17.44	12.81	10.27	7.5
H=300[kN]	H=300[kN]	H=300[kN]	H=300[kN]	H=300[kN]	H=300[kN]	H=300[kN]
Y[mm]	116.11	72.94	58.49	38.72	29.04	19.38

In Fig. 4 are comparatively presented the maximum bending moments determined using FEM and according to the proposition Budhu and Davis [10] for the soft clay and in Fig. 5 the comparative diagram of horizontal pile displacement. The agreement of displacement results is considerably better. The differences of values of maximum moments would be smaller if, as already mentioned, instead of the constant value, the linear change by depth, similar to sand, would be introduced. Laterally loaded rigid piles in cohesive solids based on kinematic approach were given in [13].

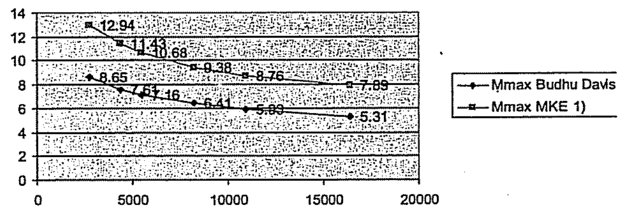


Fig. 4. Diagrams of results for Mmax determinate using FEM and according to [10]

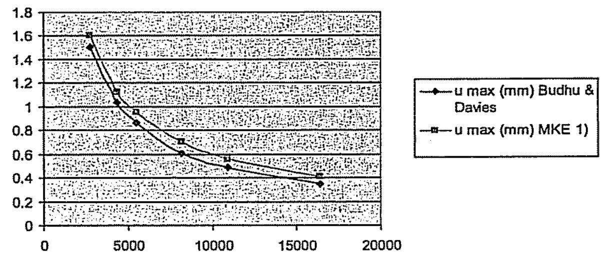


Fig. 5. Diagrams of results for displacement determinate using FEM and according to [10]

The load displacement relation - curves are nonlinear under increased horizontal force Tab. 4. When the maximum moment values according to the FEM, model 1) in regard with model 3) Fig. 6 is compared, then the moment decreases to value from 91.5 to 87.5%. For the freely resting head of the pile, when an eccentricity for 1m is introduced, model 5) with regard to the model 3), the maximum moment inside the pile increases, depending on the soil quality, from 71 to 110%. The maximum deflection for the given example increases, depending on the soil quality, from 83 to 145% (Diagrams Fig. 7 and model 6).

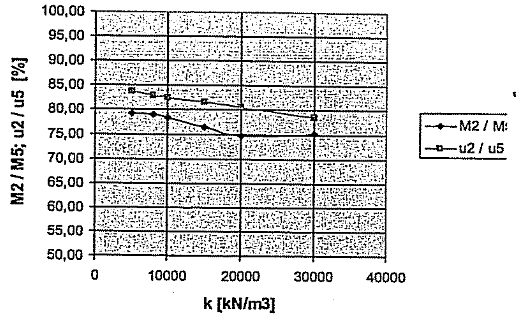


Fig. 6. Comparison of the results determined by FEM (model 2), and equivalent cantilever method (model 5) with the reduced stiffness of the upper 90 cm of soil.

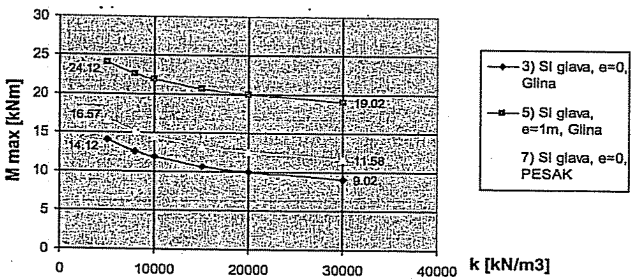


Fig. 7. Value M_{max} for the free head, with and without eccentricity. The soil stiffness in model 3) and 5) is constant – clay, and in model 7) varies linearly – sand.

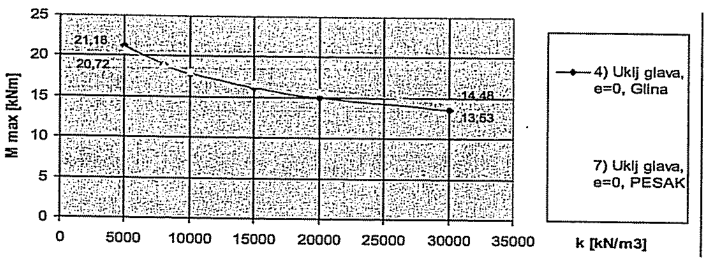


Fig. 8. Comparison of results of M_{max} for the restrained head, with and without eccentricity. The model 4) is clay, and model 7) is sand - Equivalent cantilever method

When soil with sand is introduced instead of clay, the model 6) with regard to the model 3), the maximum moment inside the pile, for a given example, increases, depending on the soil quality, from 17 to 28%, and maximum displacement increases, depending on the soil quality, from 2.5 to 33.3% (Diagrams Fig. 7 and Fig. 10). On diagram (Fig. 9) the best agreement of results occurs for the comparison of deflection of FEM model 1) and equivalent cantilever method model 3). The comparison of the equivalent cantilever method model, for the clay model 3) and the sand model 6), for weaker soil types the results almost coincide (2.5%), and with the increased compaction-stiffness of the soil this difference also increases and reaches 33%.

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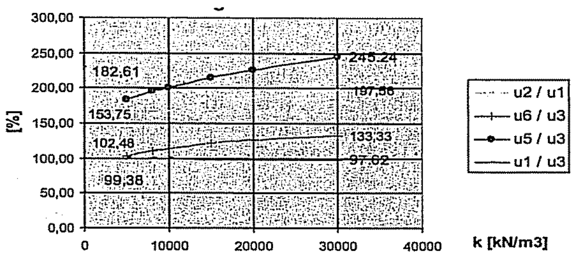

Fig. 9. Comparison of maximum deflections for all models

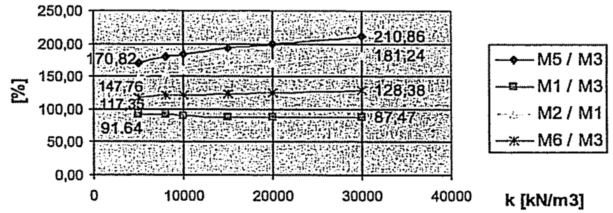

Fig. 10. Comparison of maximum moments

4. Pile group

The Winkler model has no interaction between adjacent piles in the group. The deformation of the pile under load is accompanied by deformations of the surrounding soil which decrease with distance from the pile. The application of load to one pile will cause movement of adjacent piles [11]. Interaction factors are needed for pile-pile interaction under vertical, horizontal, and moment loading. The ultimate lateral capacity of a pile group depends on the length to diameter ratio of pile, pile friction angle, pile group geometry, spacing of piles in a group and soil placement density. The quantitative and qualitative influence of those parameters was investigated [11], [14]. The ultimate resistance per pile increases with an increase in pile spacing. A method for evaluation the distribution of the external loads within a pile group was presented in [14]. The model is based on the imposition of the rotation compatibility condition between the piles cap and the piles heads. It includes the soil-foundation interaction which depends on two main parameters. The result is a set of clear formulas similar to those provided by the statically determinate method. The basic assumptions are: rigid pile cap; mechanical and geometric characteristics equal for each pile; independent behaviour of each pile; resultant external loads acting along one principal direction of the pile plan.

Conclusion

The analysis conducted in the text indicates that in the conceptual phase of design of pile foundations, also the approximate methods can be used. Comparing the results calculated on some models it could be concluded that the differences of maximum moments are within tolerant limits and that the calculation by using of equivalent cantilever method is reliable. For the values of modules of soil reactions between 5000 and 30 000 kN/m^3, the differences in curvature of analysed diagrams are somewhat more stressed than for soils of larger stiffness. Nevertheless, it is important to choose the adequate soil parameters, whether the soil reaction coefficient or soil deformation modulus are used. It is particularly important for the low bearing capacity soils. In case of sandy soil it is necessary to introduce the FEM model with linearly variable stiffness of soil as a function of depth. The Winkler model, as it has been demonstrated with the presented analysis provides the pile behaviour data (for service load) useful for practice.

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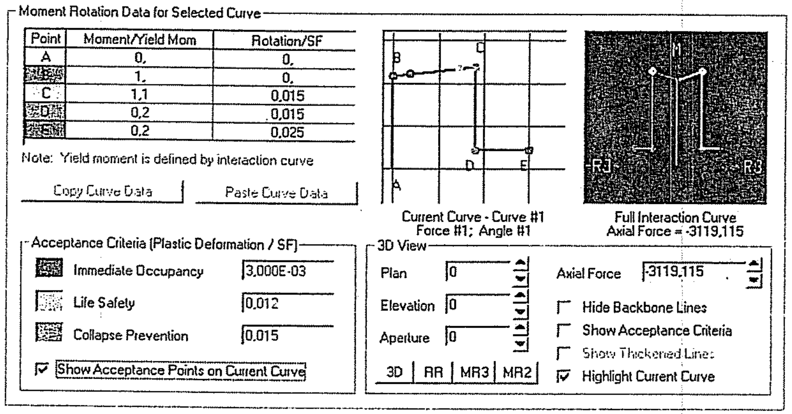
Apendix: Nonlinear analysis



APPROXIMATIVE PROCEDURE TO HOMOGENIZATION OF MICROSTRUCTURED COMPOSITE MATERIALS

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Abstract: *The paper proposes a numerical method for the homogenization of microstructural elastic materials. The method is based on numerical experiments involving macro loads applied to preliminarily chosen representative volume element (RVE). Load type and combination are found via experiment planning. FEM is used on micro-level to find the macro elastic characteristics by means of averaging. The capabilities of the method to produce satisfactory results are analyzed. Comparison to some known models of composite materials is also made.*

Key words: *microstructural elastic materials, homogenization, FEM*

1. Introduction

During the second half of the previous century new materials proving exceptional mechanical properties have been invented with the development of the Material Science. These are intermetalics, ingredient materials, nanocompostes, geocements etc. This new class of materials possesses complex composite structures, and thereby corresponding models treating the composite at its nano-, micro-, mezzo- and macro-structural levels should be applied in the research and determination of constitutive relations [1]. In the bibliography, a number of elaborations could be found in the three basic types of research fields: analytical, numerical and experimental.

For microcomposite materials the transition between the models at the different structural levels through different methods of homogenization brings difficulties of theoretical and computational character [1], 3, 5-7]. In this elaboration aiming an easy performance of such transition a numerical procedure, although that it is approximative, is proposed. This approach realizes a transition from a lower to a higher structural level imitating real physical experiments numerically. The approximation of the procedure is in the ranges of the assumptions during the working out of physical experimental data [8]. We assume that the mechanical characteristics and the material's model at the lower level are known, and after we look for the mechanical characteristics and corresponding model at the higher level by homogenization. The procedure eliminates the need of carrying out expensive experiments at the highest level. If the process of composite preparation is controllable, this procedure, as a numerical, permits variations in the solutions for the composite content at the lower level with aims to design a composite with optimal properties for a given application at the higher level.

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