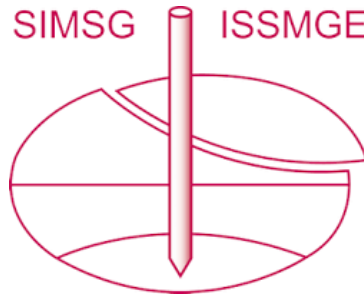


INTERNATIONAL SOCIETY FOR SOIL MECHANICS AND GEOTECHNICAL ENGINEERING



This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:

<https://www.issmge.org/publications/online-library>

This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.

Finite element modelling of the driving process of piles

La modélisation par éléments finis du battage des pieux

A.CIVIDINI, Politecnico di Milano, Milan, Italy

G.GIODA, University of Udine, and Politecnico di Milano, Italy

SYNOPSIS: A finite element procedure is outlined for the evaluation of the stress and strain states induced in a soil deposit by the driving process of a pile. The analysis is carried out in axisymmetric conditions, considering the pile as a rigid object advancing into the ground in quasi static regime. The development of large strains is accounted for, through an updated Lagrangian approach, as well as the non linear material behaviour of soil and of the interface between pile and soil. The results obtained in the analysis of the penetration of a pile in a cohesive soil are discussed.

1 INTRODUCTION

The determination of the states of stress and strain caused in a soil deposit by the penetration of cylindrical object is one of the problem frequently met in the design of deep foundations or in the interpretation of CPT or CPTU tests, see e.g. [ISSMFE, 1988]. The complex characteristics of the problem, related to the non linear mechanical behaviour of soils, to the changes in geometry during the advancing process and to the consequent development of large strains make numerical, computer oriented methods particularly suitable for its solution.

The majority of the numerical studies on this topic can be subdivided into four groups, depending on the specific aspect of the problem that they consider in details:

- 1) initial phase of the penetration process;
- 2) behaviour at a constant depth of penetration;
- 3) cavity expansion problems;
- 4) "flow" of soil around a rigid obstacle.

The early stages of penetration, when the pile tip is close to the surface, involve failure of the soil close to the tip and heave or settlement of the surrounding ground, depending on the tip geometry and on the mechanical behaviour of soil. These phenomena have been studied through techniques used in the limit analysis of shallow foundations, see e.g. [Houlsby and Wroth 1982], obtaining useful information about the influence of the shape and roughness of the tip on the force required for initiating penetration. Other studies consider a pile already installed at a known depth, and range from limit analysis approaches [Durgunoglu, Mitchell, 1975; De Simone and Golia, 1988] to small and large strain elasto-plastic finite element analyses [De Borst and Vermeer, 1982; Van Der Berg and Vermeer, 1988]. With a few exceptions, see e.g. [Kioussis et.al, 1988], they neglect the effects related to further penetration of the pile.

To improve the simulation of the continuous advancing process, various approaches have been proposed based on the solution of cavity expansion problems in one dimensional (plane strain and axisymmetric) and two dimensional (axisymmetric) conditions [Banerjee and Fathallah, 1979]. Usually they cannot take into account the shear

interaction between pile and soil and, hence, cannot indicate the value of the force required for penetration. The numerical procedures of the last group make the penetration of the pile into the ground equivalent to the flow of fluid against a fixed obstacle [Baligh, 1985]. The state of strain in the soil is first determined on the basis of the fluid velocity field, then the constitutive equations are used to work out the stress distribution. In general this analysis leads to stresses which do not fulfill completely the equilibrium equations on infinitesimal volume. This drawback can be eliminated through a finite element approach proposed by Houlsby et al. [1985].

The mentioned studies show that the following main effects characterizing the penetration process should be considered in a consistent analysis of this problem:

- a) development of a yielded zone in the soil close to the pile tip;
- b) displacement in the horizontal direction of the ground in vicinity of the pile (involving in general large strain conditions);
- c) relative vertical movement, and consequent shear interaction, between pile and soil;
- d) vertical deformation of the soil due to the above shear interaction with the pile.

Here a finite element analysis is discussed in which the above effects are taken into account within the same solution procedure. In the following the formulation of the large strain problem is first recalled. Then some details of the finite element program are illustrated, related in particular to the technique adopted in the simulation of the advancing process. Finally the results obtained in the analysis of a pile driven into a cohesive soil are discussed and compared with those obtained with a cavity expansion approach.

2 GEOMETRICALLY NON LINEAR ANALYSIS

Two alternative formulations can be adopted in large strain analyses [Bathe, 1982]: the so called Lagrangian and Eulerian approaches. The first one considers a reference volume of material and describes its displacements, and chan-

ges in stresses and strains, during the non linear process. The second one considers a fixed region in space and describes the behaviour of the material moving through it. The Eulerian formulations are particularly suited when dealing with fluid flow problems. The Lagrangian approaches are preferable for the analysis of deformable bodies and can be subdivided into Total and Updated Lagrangian formulations. They adopt, respectively, a fixed (initial) reference configuration or a reference configuration varying during the non linear process.

Here an Updated Lagrangian formulation of the finite element problem is adopted. The incremental form of the equilibrium equations is obtained through the principle of virtual work where the stress and strain states are expressed, respectively, by the second Piola-Kirchhoff σ^{PK} and Green-Lagrange tensors.

$$\int_{V_t} \underline{B}^T \Delta \sigma_{t+\Delta t}^{PK} dV + \int_{V_t} \Delta \underline{B}^T \sigma_t^{PK} dV = \Delta \underline{f}_{t+\Delta t} \quad (1)$$

In eq.(1) $\underline{B}$ is the matrix of the shape function derivatives, Δ denotes the increments from time t to $t+\Delta t$, $\underline{f}$ are the nodal forces. The integrals are over the volume at the initial time t , since the volume at time $t+\Delta t$ is not a priori known.

By introducing the constitutive equations and the strain-displacement relationship, with some manipulations the following incremental form between nodal displacements and nodal forces is arrived at,

$$(\underline{K}_0 + \underline{K}_\sigma) \Delta \underline{u}_{t+\Delta t} = \Delta \underline{f}_{t+\Delta t} \quad (2)$$

where $\underline{K}_0$ is the customary small-strain stiffness matrix and $\underline{K}_\sigma$ is the so called initial stress stiffness matrix, which depends on the state of stress at the beginning of the increment. Note that if a Total Lagrangian approach were used, the stiffness matrix would have involved an additional term: the so called initial displacement stiffness matrix.

The solution of the above system of non linear equations could be obtained through a series of very small load increments. However, this in general leads to an exceedingly large computer time for analysis. To limit the computational cost, and to increase the accuracy of results, the use of an iterative procedure is advisable. This procedure, based on the following relationship, permits to take into account also the non linear material behaviour and the changes in boundary conditions which are necessary to simulate the advancing process, as it is shown in the next section.

$$(\underline{K}_0 + \underline{K}_\sigma) \Delta \underline{u}_{t+\Delta t}^i = \Delta \underline{f}_{t+\Delta t} - \int_{V_{t+\Delta t}} (\underline{B}^T \underline{\sigma}^{i-1})_{t+\Delta t} dV \quad (3)$$

The term on the right hand side of eq.(3) represents the unbalanced nodal forces due to both geometrical and material non linear behaviour. At the end of each load increments the nodal coordinates and the matrices $\underline{K}_0$ and $\underline{K}_\sigma$ are updated on the basis of the calculated displacements.

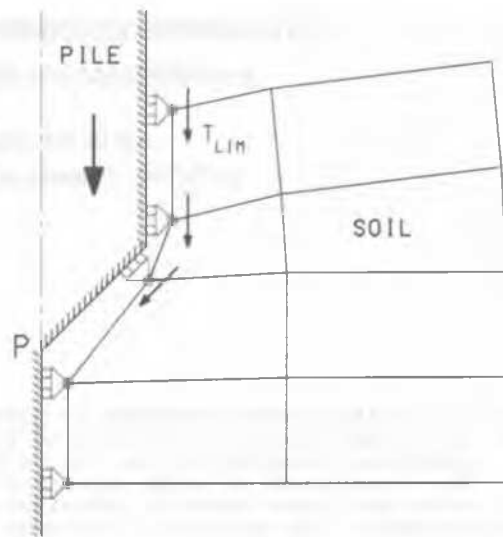


Fig. 1 Finite element scheme adopted in the simulation of the advancing process.

3 SIMULATION OF THE ADVANCING PROCESS

Among various alternative schemes applicable to the finite element simulation of the penetration process [Cividini and Gioda, 1988], the one here adopted represents the pile by means of a rough, rigid "rail" moving downward by small increments (cf. fig.1). The rail consists of a series of straight segments to which the soil is constrained by means of rollers that cannot support tensile normal forces exceeding the tensile resistance of soil. The shear interaction between pile and soil is taken into account by applying to each roller on the pile wall a tangential force evaluated through the constitutive equations of the pile-ground interface.

At the beginning of the calculations all rollers are located on the axis of symmetry (coinciding with the pile axis) and no tangential forces are applied to them. The pile is then moved downward by a small increment and new positions of the rollers are determined by means of a non linear analysis at the end of which the nodal coordinates are updated. The process is continued until reaching the desired depth of penetration.

Some provisions have to be introduced when a roller reaches the corner between two adjacent segments of the rigid rail (point P in fig. 1). It could happen, in fact, that the roller remains "trapped" in the corner for some subsequent steps of analysis, during which it behaves as a hinge subjected to known vertical displacements. This node is freed again when the tangential force on the hinge overcomes the shear resistance of the pile-ground interface.

4 AN ILLUSTRATIVE EXAMPLE

The finite element procedure summarized in the previous sections has been applied to the analysis of the penetration of a pile in a homogeneous deposit of cohesive soil. The results of calculations are summarized in figs.2, 3 and 4.

Fig.2 shows a detail of the axisymmetric finite element mesh close to the symmetry axis and some

stages of the advancing process, from the initial undeformed state up to a penetration depth equal about to $15r_p$ (r_p being the radius of the pile). In order to limit the cost of calculation this illustrative analysis has been based on a relatively coarse mesh. In fact, the finite elements discretizing the soil in the zone close to the pile have size equivalent to that of the pile tip. The accuracy of results could be improved by reducing the height of these elements, so that several nodes can be simultaneously in contact with the tip at every stage of the advancing process.

The semilog diagrams in figs.3 and 4 refer to two limit cases with respect to the shear interaction between pile and soil. Those in fig.3 were obtained considering the penetration of a perfectly "smooth" pile. They show the variation with the radial co-ordinate r (at constant depth $z=4.0r_p$) of the mean component of the normal stresses σ_m , divided by the cohesion c , and of the second invariant of the deviatoric stresses J_2 . The solid curves a, b and c represent the stress distribution for different depth of penetration, while the dashed curves CE represent the results obtained from the cavity expansion analysis.

Fig.4 reports the diagrams obtained considering an upper value of the pile-soil adhesion equal to the cohesion of soil.

The results of analyses show the influence of the depth of penetration and of the pile-soil adhesion on the stress state in the soil. An aspect of these analyses which seems worthwhile mentioning is the non negligible difference between the stresses obtained with the proposed procedure and with the cavity expansion approach. The difference reaches its maximum in the vicinity of the pile wall, where the non linear effects are predominant, and is particularly large for the volumetric stress. This indicates that simplified analyses based on cavity expansion approaches, when applied to the analysis of two-phase soils, could lead to non negligible error in the determination of the pore pressure induced by the penetration process.

5 CONCLUDING REMARKS

A finite element procedure has been discussed for the analysis of the stress and strain states induced in a soil deposit by the advancing of a pile or a penetrometer.

The calculations are carried out in large strain regime, adopting a Updated Lagrangian approach and accounting for the non linear behaviour of the soil and of the pile-soil interface. This solution technique and a standard cavity expansion approach have been applied to the analysis of a pile advancing into a cohesive soil, considering the two limiting cases of perfectly "smooth" pile and of pile-soil adhesion equal to the soil cohesion. In both cases the results of calculations show the limited approximation of the cavity expansion approach. In particular, this approach tends to overestimate the volumetric stress at the end of penetration and, consequently, could lead to non correct values of the pore pressure if applied to the analysis of saturated two-phase media.

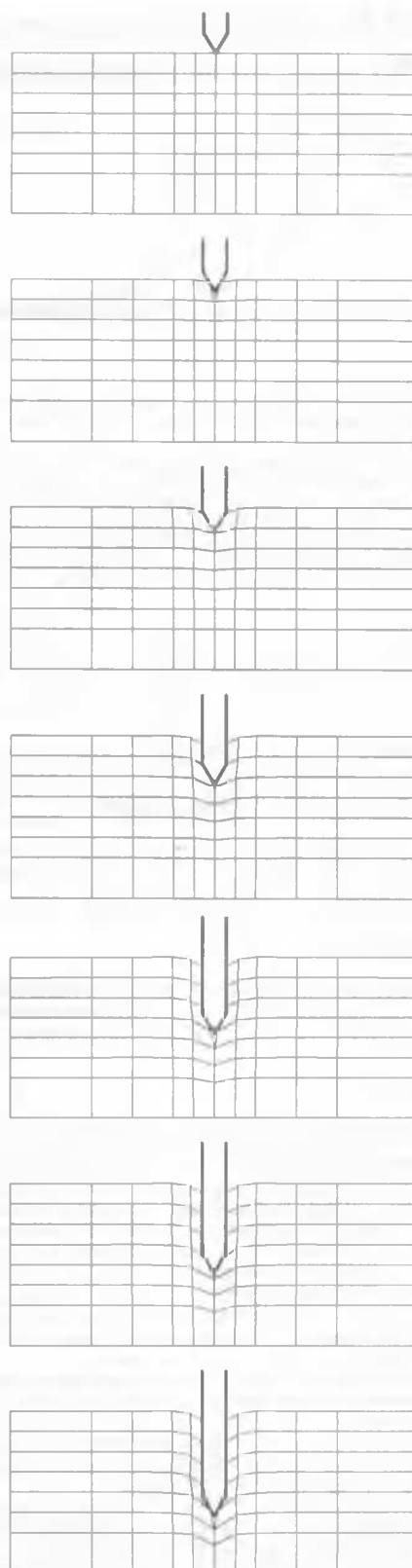


Fig. 2 Details of the deformed mesh during the advancing process of the pile.

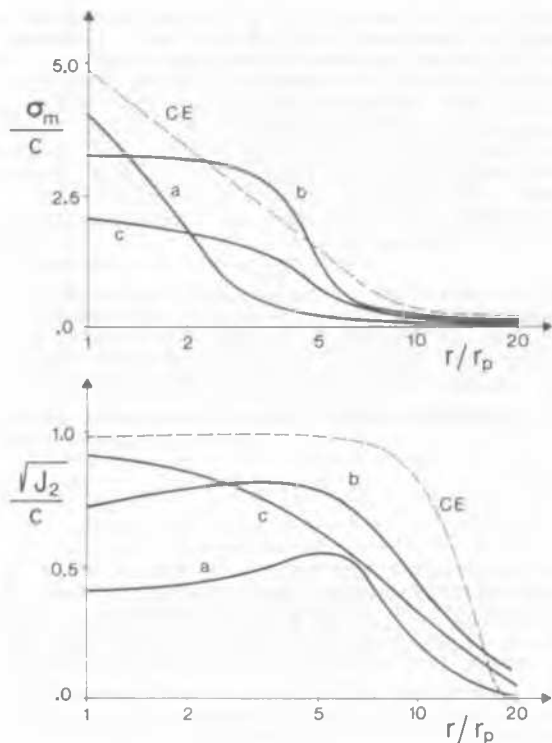


Fig. 3 Smooth pile example. Variation of the mean normal stress and of the second invariant of the deviatoric stresses with the radial coordinate, at depth $z=4r_p$, for depth of penetration equal to: a) $3r_p$; b) $6.5r_p$; c) $10r_p$. CE denotes the results of cavity expansion analysis.

ACKNOWLEDGEMENTS

This research is supported by the National Research Council (CNR) and by the Ministry of Education (MPI) of the Italian Government.

REFERENCES

- Baligh, M.M. (1985). Strain path method, J. Geotech. Engng. Div. ASCE (111), GT9, 1108-1136.
- Baligh, M.M. (1985). Fundamentals of deep penetration: I, soil shearing and point resistance; II, pore pressures, Research Reports R85-9, R85-10, Dept. of Civil Engineering, MIT.
- Banerjee, P.K., Fathallah R.C. (1979). An eulerian formulation of the finite element method for predicting the stresses and pore water pressures around a driven pile, Proc. 3rd ICONMIG, 1053-1060, Aachen.
- Bathe, K.J. (1982). Finite element procedures in engineering analysis, Prentice-Hall, Englewood Cliffs.
- Cividini, A., Gioda, G. (1988). A simplified analysis of pile penetration, Proc. 6th ICONMIG, 1043-1049, Innsbruck.
- De Borst, R., Vermeer, P.A. (1982). Finite element analysis of static penetration tests, Proc. 2nd ESOPT, 457-462, Amsterdam.
- De Simone, P., Golia, G. (1988). Theoretical analysis of the cone penetration test in sands, Proc. 1st ISOPT, 729-735, Orlando.

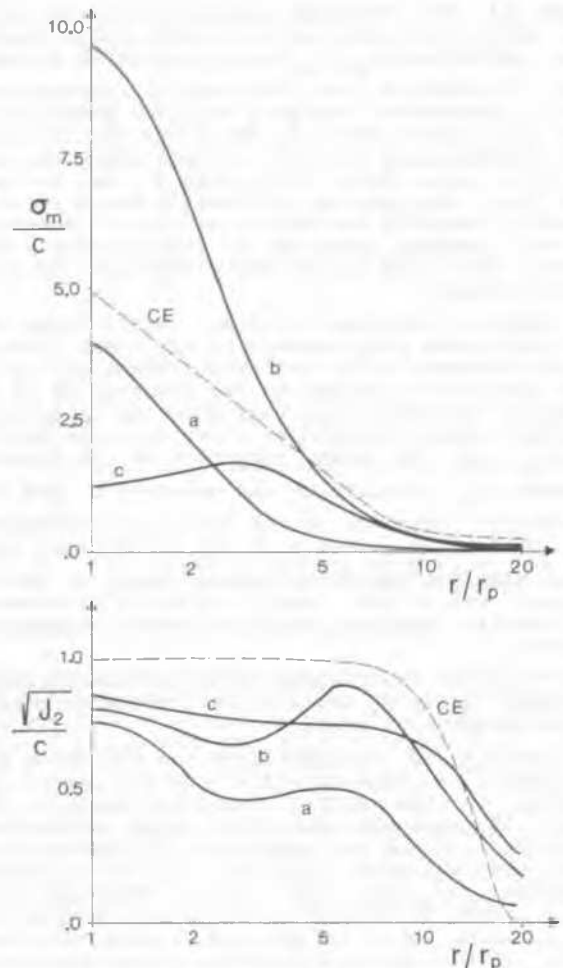


Fig. 4 Stress distribution due to penetration assuming the pile-soil adhesion equal to the cohesion of soil. Other characteristics as in fig. 3.

- Durgunoglu, H.T., Mitchell, J.K. (1975). Static penetration resistance of soils: I - Analysis; II - Evaluation of theory and implications for practice, Proc. ASCE Conf. on In situ Measur. of Soil Properties, 151-189, Raleigh.
- Houlsby, G.T., Wheeler, A.A., Norbury J. (1985). Analysis of undrained cone penetration as a steady flow problem. Proc. 5th ICONMIG, 1767-1773, Nagoya.
- Houlsby, G.T., Wroth, C.P. (1982). Determination of undrained strengths by cone penetration tests, Proc. 2nd ESOPT, 585-590, Amsterdam.
- ISSMFE Technical Committee on Penetration Testing (1988). Cone penetration test (CPT): International reference test procedure, Proc. 1st ISOPT, 27-51, Orlando.
- Kiousis, P.D., Voyiadjis, G.Z. and Tumay M.T. (1988). A large strain theory and its application in the analysis of the cone penetration mechanism, Int. J. Numer. Anal. Methods Geomech. (12), 45-60.
- VanDenBerg, P., Vermeer, P.A. (1988). Undrained strength from CPT and finite element computations, 6th ICONMIG, 1095-1100, Innsbruck.