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CODE-ORIENTED STUDIES ON THE BEHAVIOUR OF SHALLOW FOUNDATIONS UNDER SEISMIC LOADING

E. FACCIOLI[1], R. PAOLUCCI[1] AND A. PECKER[2]

[1] Department of Structural Engineering, Politecnico di Milano, P.zza L. da Vinci 32, 20133 Milano, Italy
[2] Géodynamique et Structure, 157 rue des Blains, 92220 Bagneux, France

ABSTRACT

Recent research on the seismic behaviour of shallow foundations, carried out by the authors in the framework of a project sponsored by the European Commission, is summarized in this paper. Illustrated are a pseudo-static approach, based on the yield design theory, and a dynamic approach, based on finite element simulations of simple soil-structure-foundation systems and taking into account the non-linear soil behaviour by a sophisticated constitutive model. The effects of the seismic actions transmitted by the structure to the foundation and of the soil inertia on the ultimate bearing capacity are discussed and simple formulas are derived for code-oriented applications. The results of finite element simulations are subsequently presented, and some practical indications for code-oriented purposes are derived.

KEYWORDS

Foundation seismic bearing capacity; finite element simulations; kinematic approach; permanent settlements; soil constitutive models.

INTRODUCTION

We summarise here the main developments occurred from the end of 1994 to that of 1995 in the part of the research Project "Prenormative research in support of Eurocode 8" devoted to geotechnical earthquake engineering topics, specifically to the seismic bearing capacity of shallow foundations. Thus, this paper can be viewed as the continuation of an earlier contribution on the same subject (Faccioli *et al.*, 1994) .

The principal factors affecting the seismic behaviour of shallow foundations are thoroughly discussed in a recent state-of-art review by one of the authors (Pecker, 1994). Because the dynamic forces can temporarily exceed the resistance of the soil-foundation system without causing a general failure of the foundation, and because of the soil nonlinearity, one key aspect of the seismic case is that the variation in time of such forces need generally be considered. Hence, to evaluate how conservative the pseudo-static bearing capacity verifications recommended in codes (including Eurocode 8) may be in the presence of seismic actions, it is crucial to assess the relationship (if any) between the permanent deformations induced in the soil-foundation system by dynamic loading and the collapse condition under static loads.

This requires, on one hand, the knowledge of rigorous solutions for the collapse loads in the presence of load inclination and eccentricity on the foundation, obtainable by the upper and lower bound approaches of the yield design theory. On the other hand, data from carefully controlled laboratory tests are needed , or parametric dynamic analyses of soil-foundation systems by a rigorous finite element approach with at least a simplified model of the structure and a realistic nonlinear constitutive description of the soil.

Based on those, one can e.g. determine whether the combined action of the pseudo-static horizontal load and overturning moment transmitted by the superstructure correspond to the general failure threshold beyond which a fast accumulation of irreversible displacements occurs. This seems to be implied by the examples discussed by Faccioli *et al.* (cit.) for granular soils and by Pecker (cit.) for cohesive soils, and by a dynamic failure definition tied to the generation of excessively large displacements for the structure.

Especially critical appears the need to assess whether under dynamic conditions the influence of the eccentricity of the load is so severe as in pseudo-static verifications of the bearing capacity . Consistently with the foregoing conceptual outline, we illustrate first some theoretical results for the static collapse loads both for cohesive and granular soils, and the design-oriented provisions and formulas derived thereby. Subsequently the results of nonlinear dynamic finite element analyses of a soil-structure system are presented, where a substantial effort has been made at simulating the drained experimental behaviour of a real soil (Hostun sand) and at analysing the influence of the soil density upon the permanent deformations of the foundation. The implications with respect to the pseudo-static verifications are discussed, and also the feasibility of simplified dynamic analyses derived from the method of Newmark.

PSEUDO-STATIC APPROACHES

Yield design theory

The yield design theory belongs to the category of limit analysis methods and was originally introduced by Salençon (1983). Like any limit analysis method, the derivation of upper bound and lower bound solutions allows to bracket the exact solution, and possibly to determine it exactly when the two bounds coincide. A proper application of the yield design theory requires the knowledge of:

- the *problem geometry*; in the following, unless otherwise stated, the foundation is assumed to be a strip footing resting on the surface of a homogeneous halfspace ;
- the *material strength*; in this section, the soil will be represented by a Tresca (cohesive soil) or a Mohr-Coulomb strength criterion (cohesionless soil), with a soil-foundation interface without tensile strength ;
- the *loading parameters*; five independent loading parameters are considered in the derivation of the foundation bearing capacity (Fig. 1): the normal force V, the horizontal shear force H, the overturning moment M=Ve and the soil inertia forces f_x and f_y in the horizontal and vertical direction. The effect of the lateral overburden will not be considered for simplicity, although it can be easily taken into account in the present approach.

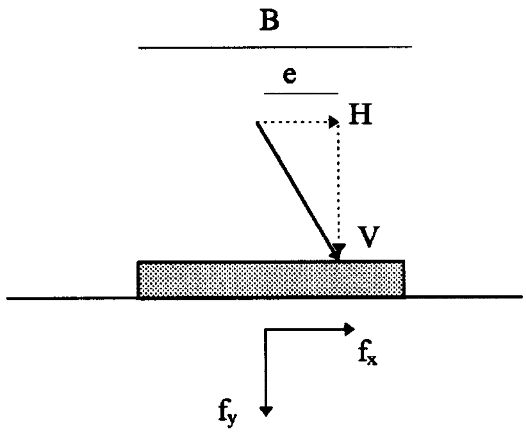
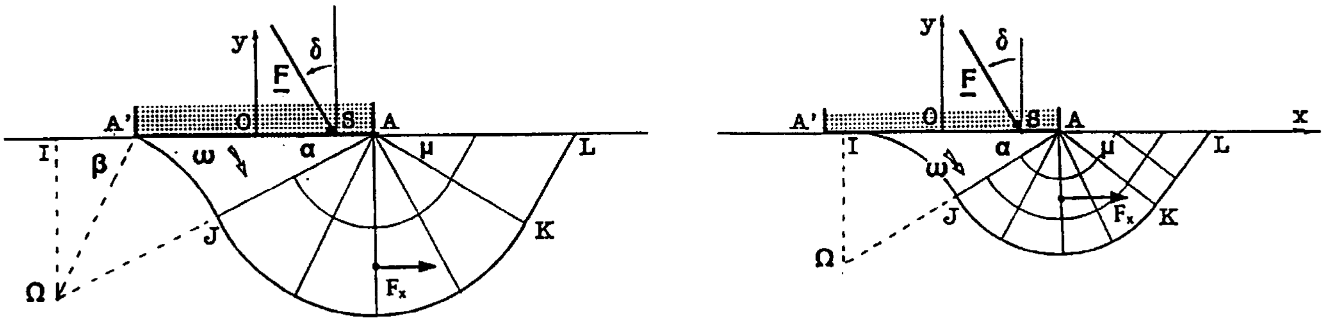


Fig. 1. System under study

Cohesive soils. Solutions to the bearing capacity of shallow strip footings resting on the surface of a cohesive halfspace have been obtained by Salençon and Pecker (1995 a, b) for a soil with or without tensile strength. These solutions were derived from the static and the kinematic approaches of the theory and it was shown that the lower bound and the upper bound solutions were very close to each other, giving therefore an almost exact solution to the problem. The most prominent kinematic mechanisms used are presented in Fig. 2 in two situations: without uplift of the foundation and with uplift. The first situation is prevailing for small load eccentricities or inclinations whereas the second one governs when these two parameters become significant. These mechanisms depend upon three geometric parameters for which the optimum values, which minimise the maximum resisting work, are numerically determined. In the loading parameters space, the set of admissible loads is located within the so-called bounding surface, the upper part of which, corresponding to M>0, is represented in Fig. 3 for a soil without tensile strength and $f_x=f_y=0$.



a/ Without uplift   b/ With uplift

Fig. 2. Kinematic mechanisms used for upper bound calculations on cohesive soils.

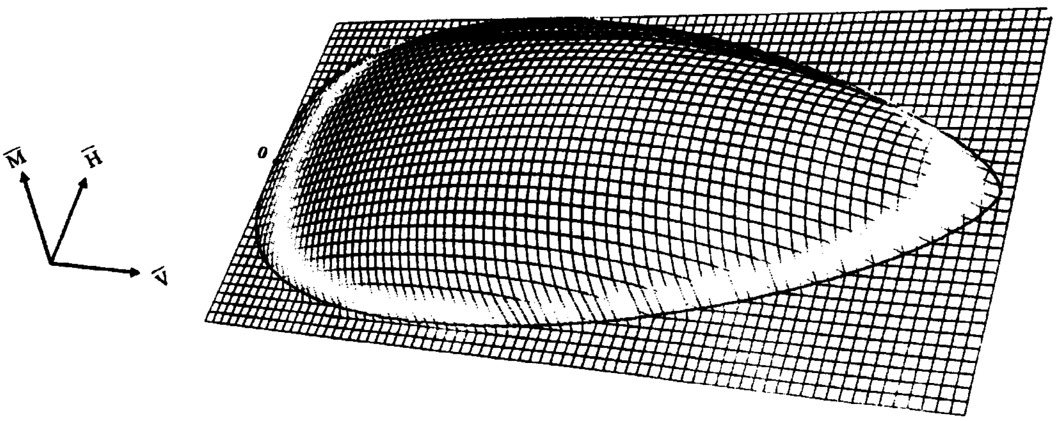


Fig. 3. Bounding surface in the $\overline{V},\overline{H},\overline{M}$ space ($\overline{M}>0$) derived by the yield design theory for a cohesive soil without tensile strength and $f_x=f_y=0$.

With respect to the seismic bearing capacity of foundations on cohesive soils, the following conclusions were derived by Pecker and Salençon (1991) and by Pecker et al (1995):

- denoting as C the soil undrained shear strength and as B the foundation width, for foundations with $\overline{V}$=V/CB≤2.5, i.e. for foundations with a safety factor higher than 2 under a vertical centered load V, and for commonly used values of the seismic coefficient k_h ($f_x = \rho g\ k_h$), the effect of the soil seismic forces can be neglected without loss of accuracy. For a foundation with a lower safety factor, the soil seismic forces induce a dramatic reduction in the bearing capacity. This is illustrated in Fig. 4 which presents cross sections of the bounding surface for various values of the non-dimensional parameter f_xB/C.

- for situations in which the soil seismic forces can be neglected, the equation of the bounding surface can be written (soil without tensile strength) in the form:

$$\frac{(\beta\,\overline{H})^2}{(\alpha\,\overline{V})^a\,[1-\alpha\,\overline{V}]^b} + \frac{(\gamma\,\overline{M})^2}{(\alpha\,\overline{V})^c\,[1-\alpha\,\overline{V}]^d} - 1 = 0 \qquad (1)$$

with : $\overline{V}=\dfrac{V}{CB}$, $\overline{H}=\dfrac{H}{CB}$, $\overline{M}=\dfrac{M}{CB^2}$, a=0.70, b=1.29, c=2.14, d=1.81, $\alpha=\dfrac{1}{\pi+2}$, β=0.5, γ=0.36, under the constraints $0<\alpha\overline{V}\le 1$, $|\overline{H}|\le 1$.

The preceding results, valid for a 2D plane strain problem, have been extended by Paolucci and Pecker (1996a) to account for a rectangular shape of the footing; the modification is expressed as a correction factor S_c to the 2D ultimate vertical load and is written as:

$$S_c = 1+0.32\left(1-\frac{2e}{B}\right)\frac{B}{L} \tag{2}$$

where e is the load eccentricity and L is the foundation length. In most practical situations, the correction is small.

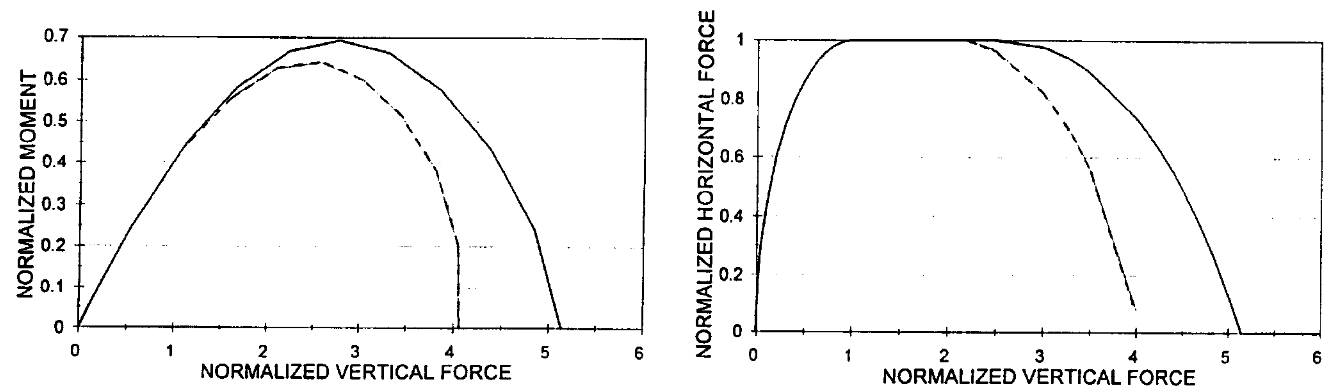


Fig. 4. Cross-sections of the bounding surface at H=0 (left) and M=0 (right), showing the effects of the soil inertia. Continous line: $f_x B/C=0$; dashed line: $f_x B/C=1$.

Granular soils. The kinematic mechanisms illustrated in Fig. 5 have been considered, similar to those used for cohesive soils. The definition of the velocity fields in the various soil blocks and the analytical determination of the power of external and internal forces for the upper bound calculations are given by Paolucci and Pecker (1996b). The bounding surface can be described in this case by the following equation:

$$h = 0.85v\left(1-\sqrt[3]{\frac{v}{(1-2e/B)^{1.8}(1-k_h/\tan\phi)^{0.35}}}\right) \tag{3}$$

where $h=H/V_{max}$, $v=V/V_{max}$, V_{max} = ultimate bearing capacity under vertical centered load and ϕ = angle of shearing resistance of soil. With the previous equation, the effects of the load inclination and eccentricity, as well as that of the soil inertia, can be easily evaluated. In agreement with the case of cohesive soils, it is found that the effect of the soil inertia can be neglected in the most common design situations. However, the combined effect of the horizontal load and overturning moment can dramatically reduce the bearing capacity of the foundation. If we assume $H=k_hV$ and $e/B=k_h\zeta/B$, ζ being the height of the point of application of the horizontal force on the superstructure, and we substitute in (3), the curves plotted in Fig. 6 are obtained, showing the relation between the seismic coefficient k_h and the static safety factor $F_s=V/V_{max}$ required for the safe seismic design of the foundation. This indication is at variance with the general good behaviour of shallow foundations observed during past earthquakes. Indeed, the exceedance of the pseudo-static failure load for a short time interval will merely lead to the development of permanent displacements, whose magnitude strongly depends on the non-linear behaviour of soil and on the severity of the ground shaking.

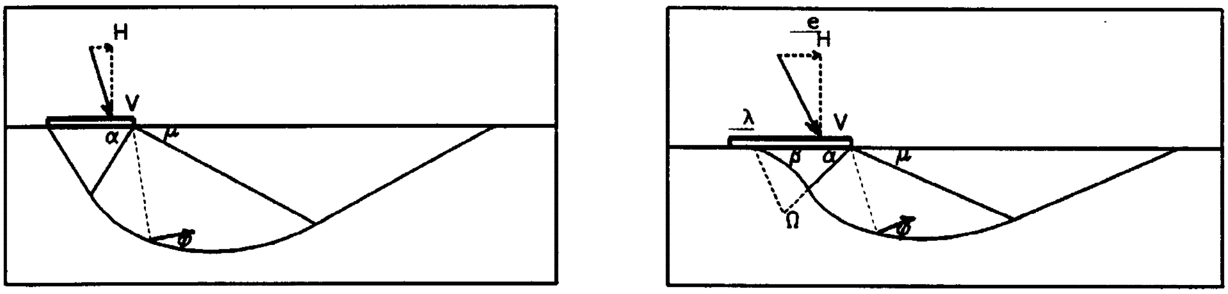


Fig. 5. Left: Prandtl type kinematic mechanism defined by two geometric parameters (α and μ). Right: Four parameters (α, μ, λ and μ) kinematic mechanism with foundation uplift allowed.

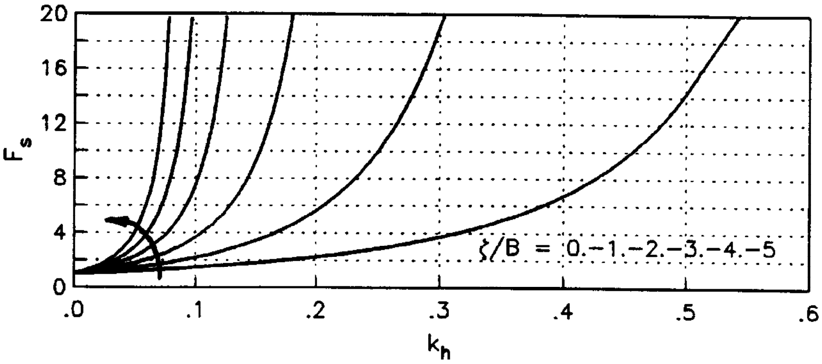


Fig. 6. Chart for the simplified evaluation of the seismic effects on the bearing capacity of a shallow foundation designed with a static safety factor F_s. The curves, plotted as a function of ζ/B, separate the safe region (upper side) from the unsafe (lower side).

The problem of the evaluation of the permanent displacements of the soil-foundation system is addressed in this paper by a finite element (FE) approach that incorporate an accurate description of the non-linear soil-behaviour. A simpler approach has been presented by Paolucci and Faccioli (1996), who show that a simple 4 degrees of freedom system with non-linear soil-structure interaction may explain most of the salient features of the seismic response of the system.

DYNAMIC APPROACH BY FINITE ELEMENT SIMULATIONS

Free-field response by 1D simulations

The contribution of the free-field soil settlements to the total settlement of shallow foundations has been studied by dynamic FE analyses on a one-dimensional (1D) soil column idealised as a shear beam. A representative soil profile was considered, consisting of a medium dense sand (relative density $D_r \cong 65\%$) with unit weight γ=19kN/m^3 and shear modulus G varying with depth as $G = G_0 (p'/p_0)^{0.5}$, where p′ is the effective confining pressure and the parameters G_0 and p_0 are chosen so that the shear wave velocity (v_s) varies from ~85m/s at the surface, up to ~220m/s at a depth of 13.5m. The Hujeux (1985) constitutive model has been used, with a set of parameters calibrated on data from static and cyclic triaxial tests (Fig. 7) available for the Hostun RF sand (from France). This model is incorporated in the non-linear code GEFDYN (Aubry *et al.*, 1986) used for the numerical analyses. A comparative study of the results of the Hujeux model and that of Mohr-Coulomb has been performed by Paolucci (1995). Further numerical simulations with a set of 10 representative real accelerograms have led to the following conclusions:

- a good correlation exist between the computed permanent vertical settlements and the Arias intensity $I_a = \frac{\pi}{2g}\int a^2(t)dt$, a(t) being the acceleration time history (Fig. 8a);

- the numerical predictions obtained by the present analyses are in good agreement with those provided by the method of Tokimatsu and Seed (1987), at least for peak ground accelerations (PGA) ≤ 0.3g (Fig. 8b).

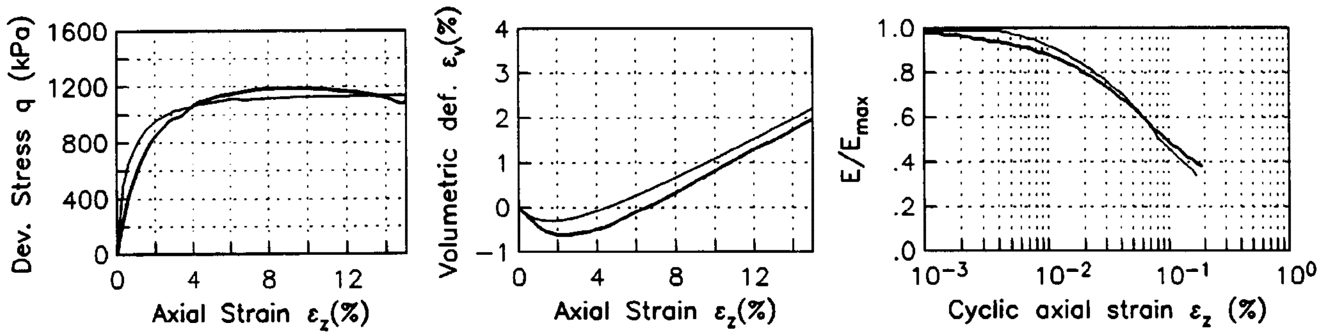


Fig. 7. Comparison of experimental results on the Hostun medium dense sand (thick line) with best fits obtained by the Hujeux constitutive model (thin line). a) and b): Static drained triaxial tests, with σ_0=0.4 MPa. c): Decay of the normalised Young modulus E/E_{max} as a function of cyclic axial strain amplitude.

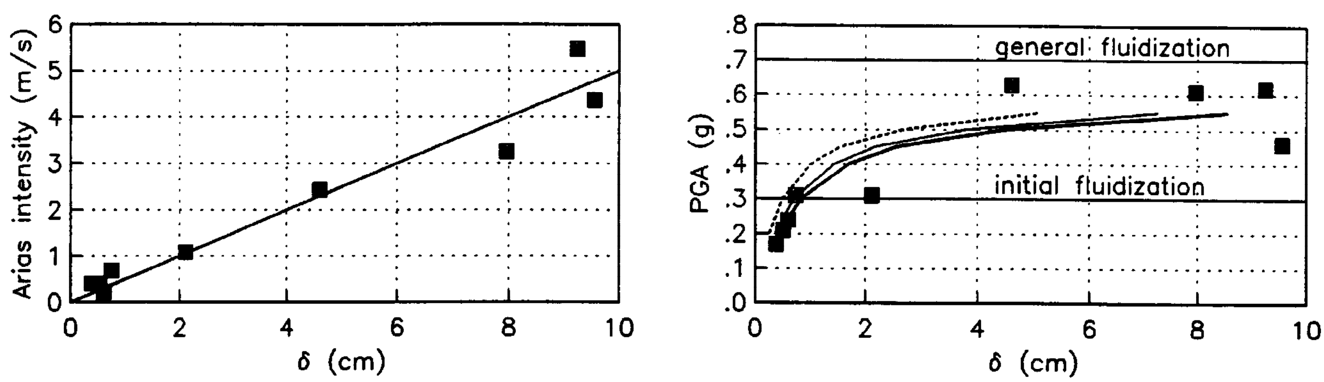


Fig. 8. a) Correlation between Arias intensity of excitation (I_a) and permanent vertical settlements of a soil column calculated by 1D dynamic finite element analyses. The least-squares best-fit line is also shown. b) Same as a) but using PGA instead of I_a. The curves have been obtained by the Tokimatsu and Seed method for magnitude M=6 (dotted line), M=6.5 (thin line) and M=7 (thick line). The initial and general fluidization levels (Richards *et al.* 1990) are also shown.

2D simulations

The 2D dynamic analyses, presented in a previous study (Faccioli *et al.*, 1994) for dense soil conditions, have been extended to deal with the looser granular soil considered in the 1D simulations. The model under study consists of a shallow strip foundation of width B=4m supporting a simple shear beam structure (Fig. 9), with fundamental period T_0=0.5s. The height of the point of application of the seismic shear forces on the superstructure is ~10m. Although several base excitations were considered, we will present herein some results for the case of the relatively severe accelerogram, from the September 1976 Friuli (Italy) earthquake sequence, with PGA=0.63g (Fig. 10).

In Fig. 11 the base shear forces calculated by FE analyses for the two soil conditions of dense and medium sand are compared, clearly showing the effects of the non-linear dynamic interaction; in the case of medium sand, the base shear is significantly reduced and the period of vibration of the structure increased, because of relative motion between the foundation and the soil due to the attainment of the failure load. In the latter case, the foundation acts as an energy dissipator, reducing the seismic action effects on the superstructure. However, the displacements calculated for the medium sand are much greater than for the denser soil conditions (Fig. 12). The rocking, in particular, reaches a final value of ~16mrad corresponding to a permanent displacement of ~25 cm at the top of the structure.

Because of the dynamic nature of the problem and the complicated interaction of the loading actions H, M, V, it is difficult at this stage of the research to determine the relation between pseudo-static and dynamic approaches. However, the studies of Pecker *et al.* (1995) for cohesive soils and Paolucci and Faccioli (1996) for granular soils, together with the present results of the FE analyses, suggest that the development of large permanent displacements in the soil-foundation system is strictly related to the attainment of the bounding surface determined by pseudo-static methods. The amount of these displacements has been found to depend mainly on the relative density of the soil.

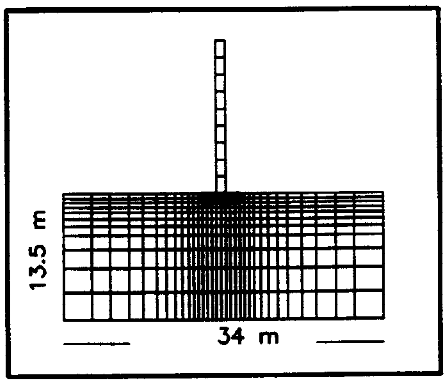


Fig. 9. Model for finite element calculations.

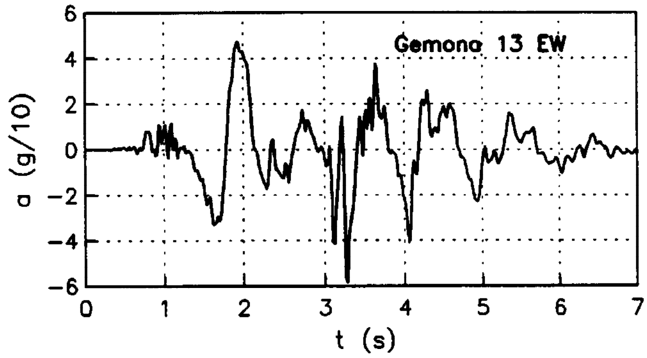


Fig. 10. Base excitation

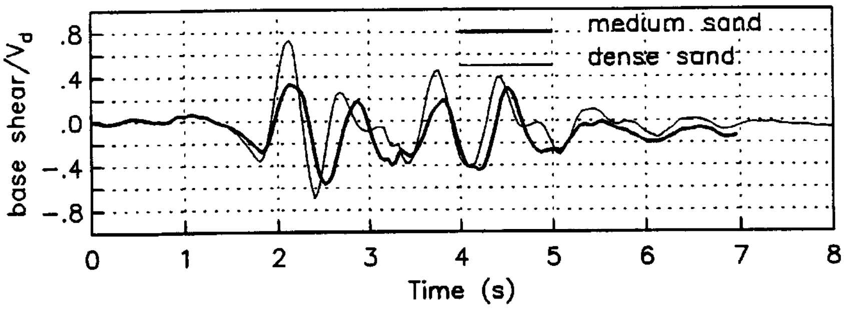


Fig. 11. Comparison of the shear actions at the base of the superstructure of Fig. 9 (normalised by the vertical static load V_d) calculated by 2D finite element analyses for the cases of a medium sand (thick line) and of a dense sand (thin line).

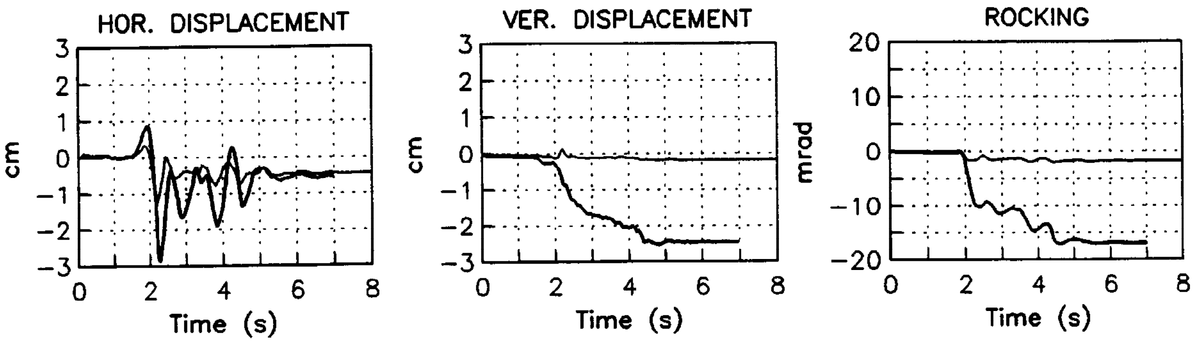


Fig. 12. Same as Fig. 11, but showing the comparison of the two displacement components and rocking of the foundation.

CONCLUSIONS

The pseudo-static approach for both cohesive and granular soils has shown that the effects of the soil inertia on the bearing capacity of shallow foundations can be disregarded, if the foundation is designed with an adequate safety factor ($F_s \geq 2.5$). On the other hand, the detrimental effects of the seismic actions transmitted by the superstructure may be very important, especially if the load eccentricity is taken into account.

The 'free-field" contribution, due to soil compaction, to the total settlement of shallow foundations cannot be disregarded. For small levels of excitation (PGA ≤ 0.3 g) it can be evaluated by standard geotechnical methods (e.g. the Tokimatsu and Seed method), whereas for higher levels the realistic description of the non-linear soil behaviour becomes more important.

In the most unfavorable conditions of shallow foundations on medium dense sands or cohesive soils, such as the soft clays of Mexico City, the attainment of the limit load, determined by pseudo-static approaches, may lead to an unacceptable development of permanent deformations of the soil-foundation system. For denser soil conditions, these deformations were found to be less important, even for high levels of seismic excitation.

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