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INNOVATIVE CONCEPTS IN FOUNDATION ENGINEERING

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ABSTRACT: Following the 1985 Michoacan Guerrero earthquake, a significant amount of work has been devoted to the evaluation of the foundation bearing capacity under earthquake loading. After a review of the previous studies which have been initiated in France, this paper presents a new, innovative, design concept to improve the seismic bearing capacity of a shallow foundation under difficult soil conditions. Aside from improving the bearing capacity, it introduces the capacity design philosophy in foundation engineering.

1. INTRODUCTION

Following the 1985 Michoacan Guerrero earthquake and the foundation failures observed in Mexico City, a significant amount of work has been devoted to the evaluation of the foundation bearing capacity under earthquake loading. Although restricted to shallow foundations, these studies represent a significant improvement on the previous situation, clarifying some of the key aspects such as relative contribution of the inclination and eccentricity of the structural loads on the one hand, and of the inertia forces within the soil on the other hand.

The theoretical studies mentioned above have been initiated in France (Pecker - Salençon, 1991; Salençon - Pecker, 1995 a-b) and later continued through a collaboration with Mexican colleagues (Pecker et al, 1995; Auvinet et al, 1996) and European colleagues (Paolucci - Pecker, 1996; Paolucci - Pecker, 1997, PREC8).

After a review of the previous studies, this paper presents a new, innovative, design concept to improve the seismic bearing capacity of a shallow foundation under difficult soil conditions. This scheme, which is presently being implemented for the foundations of the

Rion Antirion bridge, in Greece (Pecker - Teyssandier, 1998), aside from improving the bearing capacity, introduces the capacity design philosophy in foundation engineering. The capacity design principles, which are well known to structural engineers since Paulay's pioneering work (see for instance Paulay, 1993), have never been, to the best of the authors' knowledge, introduced in foundation design.

2. GENERAL FRAMEWORK FOR THE EVALUATION OF THE FOUNDATION BEARING CAPACITY

The dynamic bearing capacity of foundations can be examined from two different approaches.

The probably most rigorous approach would be to develop a global model (finite element model) including both the soil and the structure. Obviously, if the analysis is meant to be significant, a realistic non-linear constitutive soil model must be used. Owing to this constraint, to computer limitations and also to the fact that development of a global model requires competence in geotechnical engineering, structural engineering, soil-structure interaction and numerical analysis, such an approach is seldom used in every day practice. In addition, it is not well suited for

the development of design which require that various alternatives be tested before achieving a final design.

The alternative approach, which represents the state of practice, is to uncouple the evaluation of dynamic loads (a structural engineer task) from the verification of the bearing capacity (a geotechnical engineer task). This is a so-called substructure approach, which suffers the following limitations, which, up to now, have not clearly been evaluated:

- the evaluation of the dynamic loads is based on an elastic analysis of the soil-structure system; at most, some degrees of non-linearities can be accounted for in an approximate manner, but how the dynamic loads are affected by yielding of the foundation is usually not evaluated. Recently, Paolucci (1997) has shown that the base shear transmitted by the superstructure may differ from that predicted from a classical linear elastic soil-structure interaction analysis, if soil yielding is accounted for;
- the bearing capacity is checked using a pseudo-static approach, in which only the maximum loads acting on the foundations are considered.

2.1 Pseudo-static approach

Up to very recently, the seismic bearing capacity of shallow foundations was checked using classical bearing capacity formulae in which the seismic action is regarded as an equivalent static force and load eccentricity and inclination are treated as correction factors (S and i) to the N_γ , N_c and N_q bearing capacity factors.

The ultimate bearing capacity is written:

$$q = \frac{1}{2} \gamma B S_\gamma i_\gamma N_\gamma + C S_c i_c N_c + q S_q i_q N_q \quad (1)$$

Recently, methods based on limit equilibrium analyses (Chen, 1990; Salençon, 1983, 1990) have emerged, taking into account the soil inertia forces (Sarma-Iossifelis, 1990; Budhu-Al Karni, 1993; Richards et al, 1993). All these methods, although they present significant improvements on the preceding bearing

capacity equation, suffer limitations which restrict the significance of their findings:

- the mechanics assumed in these analyses do not allow for foundation uplift which can be significant for high horizontal accelerations;
- these methods are upper bound solutions to the true bearing capacity problem and no indication of their accuracy is given by comparison with lower bound estimates;
- the same horizontal seismic coefficient is applied to the structural loads and to the soil weight which leads the authors to conclude that the incorporation of the soil inertia forces results in a dramatic reduction in the foundation bearing capacity. However, the major reduction outcomes from the load inclination and eccentricity; the incorporation of the soil inertia forces only contributes for an additional small reduction (Dormieux - Pecker, 1995).

An alternative approach to the evaluation of the seismic bearing capacity has been developed within the framework of the yield design theory (Salençon, 1983, 1990) and lends itself to the definition of a bounding surface in the loading parameters space: any combination of the loading parameters (normal force N , shear force T , overturning moment M acting on the foundation, soil inertial forces acting within the soil), located outside the bounding surface, corresponds to an unstable condition, whereas any combination inside the bounding surface is a stable state.

The yield design theory belongs to the category of limit analysis methods. Alike any analysis method, the derivation of upper and lower bound solutions allows to bracket the exact solution and, possibly, to determine it exactly when both bounds coincide. A proper application of the theory requires the knowledge of:

- the problem geometry; in the following, the foundation is assumed to be a strip footing resting on the surface of an homogeneous half-space;
- the material strengths; they refer to the soil strength which is represented by a Tresca strength criterion with or without tensile strength (cohesive soil) or a Mohr Coulomb criterion (dry cohesionless soil); the soil foundation interface is characterized by a Tresca (or Mohr Coulomb) strength criterion

with zero underbase suction to allow for separation between the soil and the foundation (foundation uplift);

- the loading parameters; five independent loading parameters are considered in the derivation of the bounding surface: the normal force N , the horizontal shear force T , the overturning moment M and the soil inertia forces $F_x (= \rho g k_H)$ and $F_y (= \rho g k_V)$ in the horizontal and vertical directions.

The set of admissible loads is located within a surface, defined in the loading parameters space, and called the bounding surface:

$$\Phi(N, T, M, F_x, F_y) \leq 0 \quad (2)$$

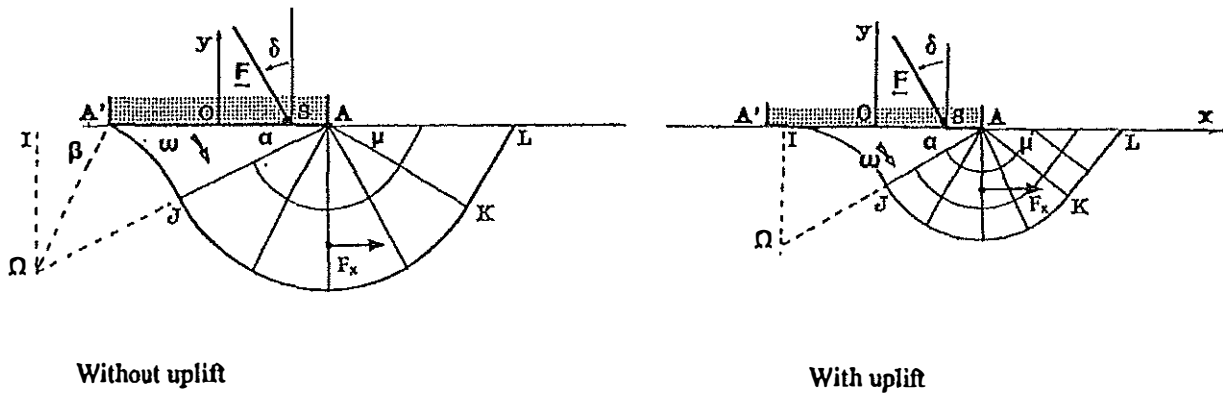


Fig. 1 - KINEMATIC MECHANISMS - COHESIVE SOIL

These mechanisms depend upon three parameters for which the optimum values, which minimize the resisting work, are numerically determined.

A simple dimensional analysis shows that the results can be expressed in terms of the adimensional parameters

$$\bar{N} = \frac{N}{CB}, \bar{T} = \frac{T}{CB}, \bar{M} = \frac{M}{CB^2}, \bar{F} = \frac{F_x B}{C},$$

$$F_y = \frac{F_y B}{C}$$

where C is the soil undrained shear strength and B the foundation width.

For a cohesive soil with tensile strength, \bar{F}_y does not play any role; for a cohesive soil without tensile strength, neglecting \bar{F}_y gives a conservative estimate of the bearing

In the case when $F_x = F_y = 0$, experimental evidence of equation (2) has been given by Butterfield - Gottardi (1994) and Kitazume - Terashi (1994).

Based on the kinematic and static approaches of the yield design theory, Pecker - Salençon (1991) and Salençon - Pecker (1995a-b), have derived the bearing capacity of a strip footing resting on the surface of cohesive halfspace.

The most prominent kinematic mechanisms used are presented in figure 1 for two situations: without uplift of the foundation and with uplift.

capacity. The following equation has been found appropriate to define the bounding surface in this latter case:

$$\frac{[(1-e\bar{F})\beta\bar{T}]^2}{(\alpha\bar{N})^a [1-\alpha\bar{N}-e\bar{F}\bar{g}]^b} + \frac{[(1-f\bar{F})(\gamma\bar{M})]^2}{(\alpha\bar{N})^c [1-\alpha\bar{N}-e\bar{F}\bar{g}]^d} - 1 = 0 \quad (3)$$

with the following values for the parameters: $a = 0.70$, $b = 1.29$, $c = 2.14$, $d = 1.81$, $e = 0.21$, $f = 0.44$, $g = 1.22$, $\alpha = \frac{1}{\pi+2}$, $\beta = 0.5$, $\gamma = 0.36$

Equation (3) is valid under the constraints:

$$0 < \alpha N \leq 1 \quad |\bar{T}| \leq 1$$

In the case when $\bar{F} = 0$, the bounding surface is presented in figure 2; only the upper part of the surface, corresponding to $M \geq 0$ is presented in figure 2.

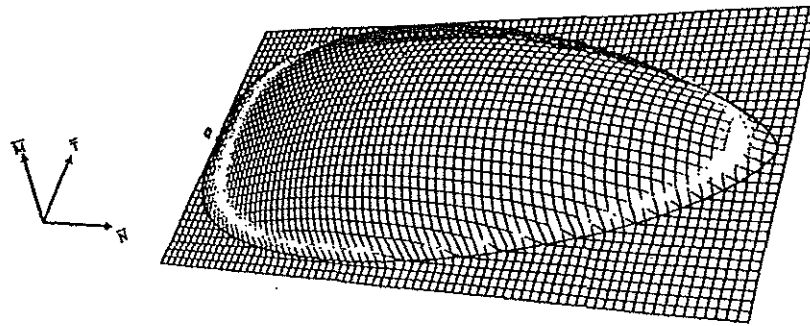


Fig. 2 - SKELETAL VIEW OF THE BOUNDING SURFACE FOR A COHESIVE SOIL WITHOUT TENSILE STRENGTH

With non zero values of \bar{F} , the following conclusions have been derived (Pecker - Salençon, 1991; Pecker et al, 1995):

- for commonly encountered ground accelerations, characterized by a value $\bar{F} \leq 2$ and for foundations for which $\bar{N} \leq 2.5$, i.e. for foundations with a safety factor higher than 2.0 under a vertical centered load, the effect of the soil inertial forces can be neglected without loss of accuracy. For foundations with lower safety factors, the soil inertia forces induce a dramatic reduction in the bearing capacity.

Solutions for a dry cohesionless soil obeying a Mohr-Coulomb strength criterion have been obtained along the same line by Salençon - Josserson (1994) and Paolucci - Pecker (1997).

Introducing the adimensional variables

$$\bar{N} = \frac{N}{N_{\max}}, \quad \bar{T} = \frac{T}{N_{\max}}, \quad \bar{M} = \frac{M}{B N_{\max}}, \quad \bar{F} = \frac{k_H}{\tan\phi}$$

in which N_{\max} is the ultimate load under a vertical centered load, the following equation has been derived for the bounding surface, whichever the value of ϕ :

$$\frac{[(1-e\bar{F})\beta\bar{T}]^c}{(\bar{N})^a [(1-g\bar{F})^d - \bar{N}]^b} + \frac{[(1-f\bar{F})\gamma\bar{M}]^c}{(\bar{N})^a [(1-g\bar{F})^d - \bar{N}]^b} - 1 = 0 \quad (4)$$

in which:

$$a = 0.92, \quad b = 1.25, \quad c = 1.14, \quad d = 0.39, \\ e = 0.41, \quad f = 0.32, \quad g = 0.96, \quad \beta = 2.90, \\ \gamma = 3.95$$

Equation (4) is valid under the following constraint:

$$0 < \bar{N} \leq (1-g\bar{F})^d$$

In equation (4), opposite to the case of cohesive soils, the gravity force and vertical inertia force in the soil do affect the bearing capacity. They are accounted for in the soil unit weight used to compute N_{\max} .

With respect to the influence of the soil inertia forces, the same conclusions as for the cohesive soils are reached; in a range of reasonable values for the pseudo-static seismic coefficient ($k_H \leq 0.3$), the reduction in the bearing capacity is small and does not exceed 15% to 20%.

2.1.2 Dynamic approach

During a seismic excitation, the forces acting on the foundation or within the soil mass vary with time. They can exceed the available resistance of the foundation soil-system for short periods without leading to a general failure of the foundation. Failure can therefore be no longer defined as a situation in which the safety factors drop below 1.0. It must rather be defined as excessive permanent displacements which impede the proper functioning of the supported structure. This definition, first

supported structure. This definition, first introduced by Newmark (1955) has been successfully applied to the design of dams, gravity retaining walls assimilating the potential unstable soil mass to a rigid sliding block. It has also been used for the bearing capacity of foundations (Sarma - Iosseff, 1990; Richards et al, 1993).

This method has been further extended, relaxing the condition of rigid soil blocks and considering a deformable body, as it is actually assumed in the computed kinematic mechanisms of figure 1. The soil foundation system is assumed to behave as a rigid perfectly plastic system, for which the bounding surface defined previously is adopted as the boundary for the apparition of plastic deformations. Using the kinetic energy theorem, the angular velocity of the foundation around point Ω in figure 1 is computed as (Pecker - Salençon, 1991):

$$\omega(t) = \frac{K}{\rho B^3} T^+ \int_{t_0}^{t_1} \left[\frac{T(\tau)}{T^+} - 1 \right] d\tau \quad (5)$$

where K is a factor related to the geometry of the optimum mechanism, ρ the soil mass density, T^+ the maximum admissible load computed from (3) and $T(\tau)$ the time history of the applied force, computed from an independent soil-structure interaction analysis. Integrating (5) between $t = t_0$, such that $T(t_0) = T^+$ and $t = t_1$, such that $\omega(t_1) = 0$, gives the permanent rotation of the foundation. This methodology has been successfully applied to actual case histories of foundation failure, as shown in figure 3 (Auvinet et al, 1996; Pecker et al, 1995).

Under the assumptions spelled above, this method permits a rigorous definition of failure in terms of unacceptable permanent displacements.

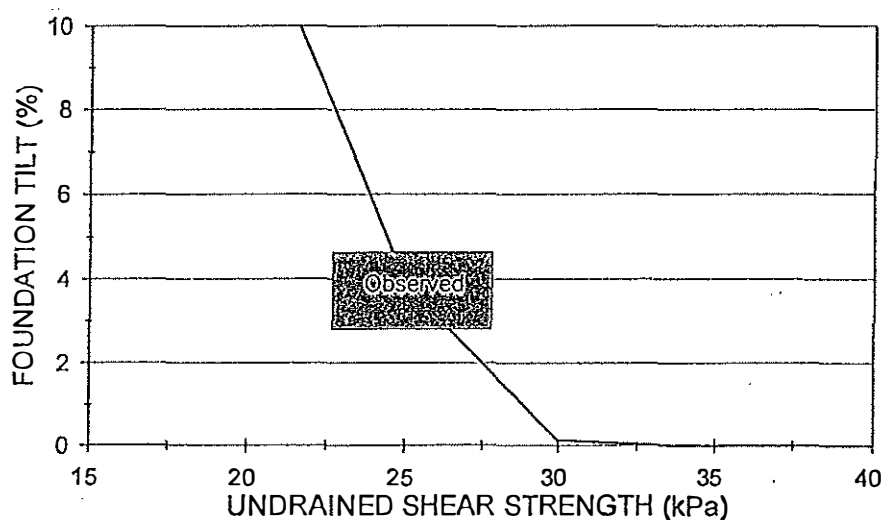


Fig. 3 - EXAMPLE OF COMPUTED FOUNDATION TILT

3. NEW CONCEPTS IN FOUNDATION ENGINEERING

Although the concepts introduced in the previous paragraph lead to a more rational approach of the seismic bearing capacity of foundations and often result in a significant cost saving of the design, the bearing capacity of foundations may still be of concern in difficult environmental

conditions characterized by poor soil conditions and high seismic intensities.

Under such circumstances, alternative foundation designs must be investigated and their relative merits, in terms of economy, feasibility and technical soundness, must be weighted before a final choice is made. When shallow foundations prove to be unsatisfactory or inadequate, a classical alternative is to resort to piled foundations,

although piled foundation failures during or after earthquakes have also been reported.

The concept which is proposed herein has, to the best of the authors' knowledge, never been proposed nor implemented before in seismic areas. It is presently being designed for a large bridge structure in Greece, the Rion Antirion bridge (Pecker - Teyssandier, 1998). It consists of reinforcing the existing soil strata with stiff inclusions at a close spacing and to lay a

shallow foundation on top of the reinforced soil through a transition, gravel layer.

For illustration purposes, figure 4 presents an example of this concept: the foundation is a gravity caisson, 90 m in diameter, and the inclusions consist of steel hollow cylinders, 2 m in diameter, 20 mm thick and 25 m long, spaced at a square grid of 7 m x 7 m below and outside the footprint of the foundation.

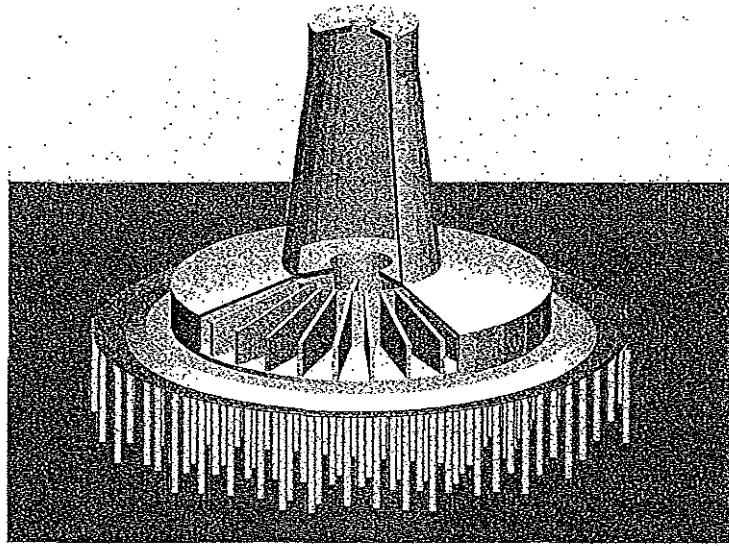


Fig. 4 - VIEW OF A REINFORCED SOIL FOUNDATION (Dumez-GTM)

Although the foundation looks like a piled foundation, it does not at all behave as such: no connection exists between the inclusions and the raft, thereby allowing for the foundation to uplift or to slide with respect to the soil; the density of inclusions is far more important and the length smaller than usually employed in piled foundations.

Aside the merits of its simplicity and economy, this technique allows for the implementation of a seismic design philosophy very similar to the capacity design principles used in structural engineering.

3.1 Theoretical analysis of a reinforced soil

The yield design theory which has been used for the evaluation of the bearing capacity on an unreinforced soil can be extended to account for the presence of inclusions or

nails (de Buhan - Salençon, 1993). In the present study, only the kinematic approach is used, thereby leading to an upper bound approach of the exact bearing capacity. A mixed modeling approach is used for the reinforced soil in which the soil is modeled as a 2D continuum and the inclusions as beams. The kinematic mechanisms shown in figure 1 have been adopted to account for the presence of inclusions.

The kinematic approach of the yield design theory states that, for any virtual, kinematically admissible, velocity field \underline{U} , the following inequality holds:

$$P_e(\underline{U}) \leq P_m(\underline{U}) \quad (6)$$

where P_e is the work of the external forces (N, T, M) and P_m the maximum resisting work of the system. P_e is unchanged with respect to the case without reinforcement

and P_m has to be modified to account for the resistance of the inclusions.

The construction of the virtual motion of an inclusion, modeled as a beam loaded within the plane of the figure consists in assigning a couple of independent vectors (\underline{U} , $\underline{\Omega}$) to any point along the inclusion (figure 5). When perfect adhesion between the soil and the inclusion is assumed, U is

defined by continuity with the virtual motion in the soil and represents the virtual velocity of the beam model of the inclusion while Ω is the virtual rotation of the cross-section at the same point. When the soil - inclusion interface presents a limited shear capacity, there exists a velocity discontinuity between the soil and the beam model of the inclusion.

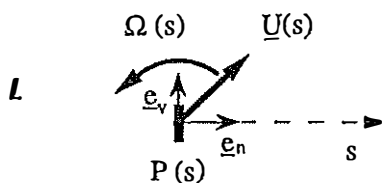


Fig. 5 - VIRTUAL MOTION OF AN INCLUSION

The strength criterion for the inclusion is given by:

$$f(n, v, m) = \left(\frac{n}{n_\ell} \right)^2 + \left(\frac{v}{v_\ell} \right)^2 + \left| \frac{m}{m_\ell} \right| - 1 \leq 0 \quad (7)$$

which yields the following expression for the maximum resisting work per unit length of an inclusion:

$$\pi = \text{Sup} \left\{ n(s) \frac{dU(s)}{ds} \cdot \underline{e}_n + v(s) \left(\frac{dU(s)}{ds} \cdot \underline{e}_v - \Omega(s) \right) + m(s) \frac{d\Omega(s)}{ds}; f(n, m, v) \leq 0 \right\} \quad (8)$$

Assuming the virtual motion of the inclusion to comply with the Navier - Bernoulli condition (i.e. the beam cross-section remains perpendicular to the axis), makes the second term in the above expression of π vanish to zero so that the maximum resisting work per unit length of the inclusion does not include any contribution from the shear force and is given by:

$$\pi = \text{Sup} \left\{ n(s) \frac{dU}{ds} \cdot \underline{e}_n + m(s) \frac{d\Omega(s)}{ds}; f(n, m, v) \leq 0 \right\} \quad (9)$$

where \underline{e}_n is the unit vector oriented along the beam axis, and s the abscissa along the inclusion.

The contribution of all the inclusions are added to the maximum resisting work of the soil (right hand side of equation [6]) and minimization is performed on the geometric parameters of the mechanism to find the best upper bound.

3.3 Example of application

Let us take for illustration purposes the example presented in figure 4. The soil profile below the foundation consists of a clay layer with a shear strength increasing linearly with depth

$$S_u = 35 + 2.8 z \quad (10)$$

where S_u is expressed in kPa and z is the depth below the ground surface in m. The normal force acting on the foundation (dead

weight) is equal to 860 MN, corresponding to a vertical stress of 135 kPa.

Without the inclusions, the cross-section of the bounding surface by the plane

$N = 860$ MN is shown as a dotted line in figure 6 in the lower left corner.

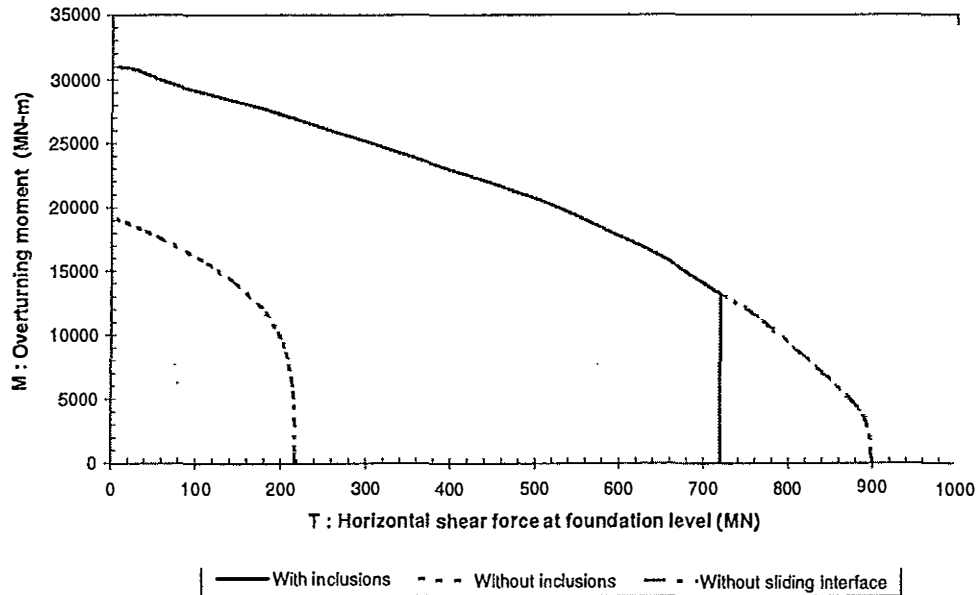


Fig. 6 - BOUNDING SURFACE FOR A REINFORCED SOIL

If the reinforced scheme described in the previous paragraph is implemented, the bounding surface is considerably expanded, as represented by the soil line. The maximum allowable horizontal shear force, corresponding to the vertical ascending line to the right of the figure is associated to horizontal sliding at the soil - foundation interface; this sliding occurs in the transition gravel layer placed on top of the inclusion

$$T = N \tan \phi = 860 \tan 40^\circ = 721 \text{ MN} \quad (11)$$

If one moves on the bounding surface from the point ($M = 0$, $T = 721$ MN), sliding at the interface does occur until the overturning moment reaches a value of 15 000 MN.m; for higher overturning moments, rotational mechanisms prevail and the maximum allowable horizontal force decreases.

4. CAPACITY DESIGN PHILOSOPHY IN FOUNDATION ENGINEERING

The capacity design philosophy used in structural engineering consists in

establishing a suitable strength hierarchy between the components of the system (Paulay, 1993). The structural system is rationally and deterministically chosen so as to be able to mobilize energy dissipating regions which will have ample reserve deformation capacity to accommodate significant departures from the initial estimates. Paulay notes that "the strategy invites the designer to tell the structure where plastic hinges are desirable or convenient and practicable at the ultimate limit state and to proscribe plastification in other regions".

Clearly, this statement is relevant to the proposed reinforcement concept.

Referring to figure 6, aside from significantly improving the resisting capacity of the foundation, the reinforcement concept enforces this philosophy:

- without reinforcement, the maximum horizontal force corresponding to a sliding at the gravel - clay interface, decreases from the beginning for increasing overturning moments; this decrease becomes more significant for overturning moments larger

than 7 000 MN.m and the failure mechanism involves rotation of the foundation;

- with reinforcement, pure sliding prevails over a large range of overturning moments (up to 15 000 MN.m in that case, more than twice the previous value). In addition, would the interface have a large strength capacity (case of inclusions connected to the raft), the vertical line at 720 MN would move to the right and the bounding surface would be represented by the dotted line joining the horizontal axis at 900 MN; in such a case, the domain of the allowable forces is extended, but as soon as the bounding surface is reached, failure modes involve a foundation rotation.

Therefore, the effect of the combined gravel layer and soil reinforcement is to improve the bearing capacity, but moreover, to enforce and control the failure mode:

- the fuse provided by the gravel layer (which is a well-controlled material) plays the role of the energy dissipating region: it limits the maximum shear force at the interface, dissipates energy by sliding and forces the foundation "to fail" according to a failure mode which is not detrimental to its overall behavior,

- the reinforcement increases the strength capacity with respect to undesirable failure modes, like rotational failure modes especially for tall structures. This increase obviously depends on the reinforcement layout and can be adjusted to the desired value.

5. CONCLUSIONS

Based on the yield design theory, a rational approach to the evaluation of the seismic bearing capacity of shallow foundations has been developed. This approach accounts for the essential features of the problem: the loading parameters (N, T, M and soil inertial forces) are treated as independent parameters, leaving to the designer the choice of the most approximate combination of them; failure is no longer defined with reference to a pseudo-static safety factor, and a methodology to compute the permanent displacements has been derived and tested against case histories.

This approach has been extended to a new design concept for foundation engineering in seismic areas. This concept based on an in-situ reinforcement of the existing soil with stiff, closely spaced, inclusions overlaid by a well-controlled gravel layer allows for the use of shallow foundations, even in difficult environmental conditions (poor soil conditions, high level of seismicity). Even more important is the fact that this foundation concept enforces a design capacity philosophy in foundation engineering. It looks therefore very promising for increasing the safety of the structure and presents the advantage of being simple and rather economical.

6. ACKNOWLEDGMENT

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The design concept presented herein is patented by Dumez-GTM.

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