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Capacity design principles for shallow foundations in seismic areas

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Keywords: Capacity design, shallow foundation, soil reinforcement, seismic bearing capacity

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1 INTRODUCTION

Following the 1985 Michoacan Guerrero earthquake and the foundation failures reported 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 1995a, b) and later continued through a collaboration with Mexican colleagues (Pecker et al. 1995; Auvinet et al. 1996) and European colleagues within the framework of the TMR program (Training and Mobility of Researchers) financed by the European Commission (Paolucci & Pecker 1997a, b; PREC8).

The general theoretical framework for the evaluation of the seismic bearing capacity has been set forth in these studies introducing the concept of a bounding surface in the loading parameters space to define the set of allowable loads, recognizing the limitations of a pseudo-static approach and developing a methodology to compute the permanent, irreversible displacements. These results have been validated by comparison with observed foundations behaviors during earthquake, experimental results on model tests and numerical sophisticated finite element analyses.

These theoretical tools are the basis of the analysis of a new, innovative at least in seismic areas, design concept implemented to improve the seismic bearing capacity of a shallow foundation (Pecker & Salençon 1998). 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.

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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 requires 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 supertructure 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.

This is clearly the approach which is favored in aseismic design building codes, like Eurocode 8 - Part 5, which states that "the bearing capacity of the foundation should be calculated using appropriate graphs and formulae which include the load inclination and eccentricity arising from the inertia forces of the structures as well as the possible effects of the inertia forces in the supporting soil itself". It is further required that the design action S_d be smaller than or equal to the design strength:

$$S_d \leq R_d \tag{1}$$

In the preceding equation, the design action must be interpreted as any combination of the design vertical force N_d, horizontal force T_d, overturning moment M_d (and possibly inertia forces in the soil mass):

$$S_d = F\,(N_d\,,\,T_d\,,\,M_d) \tag{2}$$

and the design strength as the bearing capacity equation:

$$R_d = \psi\left(\frac{C}{\gamma_m}, \frac{\tan\phi}{\gamma_m}, B, L\right) \tag{3}$$

where C and ϕ are the soil cohesion and friction angle, γ_m a material partial safety factor and B, L the foundation dimensions.

Equation (1) explicitly requires that at any time, the design action be smaller than the design strength and precludes the possibility of having the available resistance of the soil-foundation system being exceeded for short periods of time.

As pointed out by Newmark (1965) in his classical work on the seismic stability of earth dams, short periods of exceedance of the available resistance are not associated with a general failure but are accompanied by the development of permanent, irreversible displacements. This philosophy

Figure 2 presents a three-dimensional view of the bounding surface determined for a cohesive soil without tensile strength; only the upper part corresponding to $M \geq 0$ is presented in figure 2.

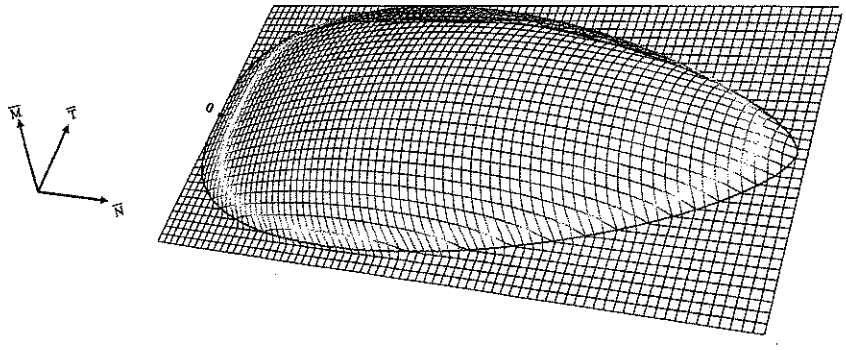


Figure 2. Skeletal view of the bounding surface for a cohesive soil without tensile strength.

Simplified analytical formulae are available to define the bounding surface for a cohesive or a cohesionless soil (Pecker 1997). It is worth noting that experimental evidence of the existence of a bounding surface has been given by Butterfield & Gottardi (1994) and Kitazume & Terashi (1994).

2.2 *Determination of permanent displacements*

One possibility which is offered by the theory, to propose tentative values for the "safety factor" γ_c in equation (4), is the computations of permanent displacements which take place once the bounding surface is reached.

This method is an extension of Newmark's original rigid blocks method, which has been implemented for shallow foundations by Sarma & Iossifelis (1990) and Richards et al. (1993), to deformable bodies.

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):

$$\omega(t) = \frac{K}{\rho B^3} T^+ \int_{t_0}^{t_1} \left[\frac{T(\tau)}{T^+} - 1 \right] d\tau \tag{6}$$

where K is a factor related to the geometry of the optimum mechanism, ρ the soil mass density, T^+ the maximum admissible load and $T(\tau)$ the time history of the applied force, computed from an independent soil-structure interaction analysis. Integrating (6) 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 (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.

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, has been successfully applied to the seismic design of earth dams and gravity retaining walls and should be extended to shallow foundations.

It could be incorporated in code-like format simply through a slight modification of equation (1)

$$F\,(N_d\,,T_d,\,M_d)\;\leq\;\frac{1}{\gamma_c}\,\psi\left(\frac{C}{\gamma_m},\frac{\tan\phi}{\gamma_m},B\,,L\right) \tag{4}$$

where γ_c is a partial "safety" factor which can take values greater than 1.0 to allow for small exceedance of the available resistance. Clearly, the value of γ_c depends on the failure mode of the structure, on its consequences with regards to the overall behavior and on the soil behavior, whether it exhibits a post peak softening or a ductile behavior.

Tentative values of γ_c have been proposed in PREC8: 1.0 for shallow foundations on clay and dense sand; 1.2 for shallow foundations on medium dense dry sand. However, additional studies based on the computations of permanent displacements and on the observed behavior of structures during earthquakes, are needed to prescribe definite values.

2.1 *Determination of the equation for the bounding surface*

The yield design theory (Salençon 1983, 1990), which belongs to the category of limit analysis methods, constitutes the appropriate theoretical framework for the evaluation of the system available resistance through equation (1). Alike any limit 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. In the following, only the kinematic approach is used to derive an upper bound estimation of the bearing capacity; for the validity of the derived solution, one can refer to Salençon & Pecker (1995a, b).

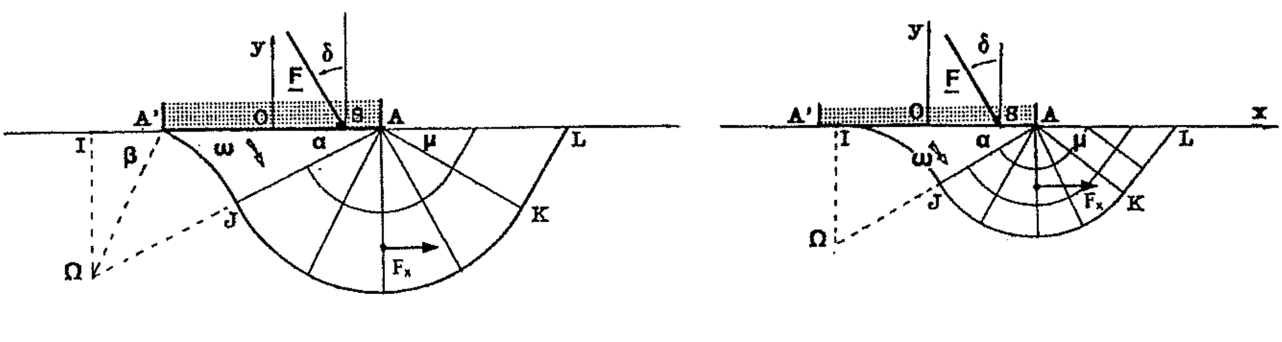
The kinematic approach of the yield design theory states that, for any virtual, kinematically admissible, velocity field $\underline{U}$, the following inequality holds:

$$P_e\,(\underline{U}) \leq P_m\,(\underline{U}) \tag{5}$$

where P_e is the work of the external forces (N, T, M) and P_m the maximum resisting work of the system which only depends on the system strengths and geometry.

Equation (5) is similar to equation (1) and constitutes the theoretical background for limit state design calculations (Salençon 1992). Consideration of different kinematic mechanisms, like those presented in figure 1 for a cohesive soil, allows the right hand side of inequality (5) to be minimized and the system resistance to be approximated (by excess).

Once the best approximation is obtained, inequality (5) defines a surface, called the bounding surface, which delimits the set of allowable loads for the system: any combination (M, N, T) falling inside the surface is a stable state of forces, whereas those combinations falling outside cannot be withstood by the system.



Without uplift **With uplift**

Figure 1. Kinematic mechanisms - Cohesive soil.

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 general framework outlined in the previous paragraph may however be used to design shallow foundations in such a way that:

- permanent displacements are allowed,
- the development of these permanent displacements do not impede a propre functioning of the structure; this can be achieved by a careful control on the failure mode.

To illustrate these possibilities, the theory is applied to a new concept in foundation engineering (Pecker & Salençon 1998), which, to the best of the author's knowledge, has never been proposed or implemented before in seismic areas. This concept is presently being designed and implemented 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 3 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 as a square grid of 7 m x 7 m below and outside the footprint of the foundation.

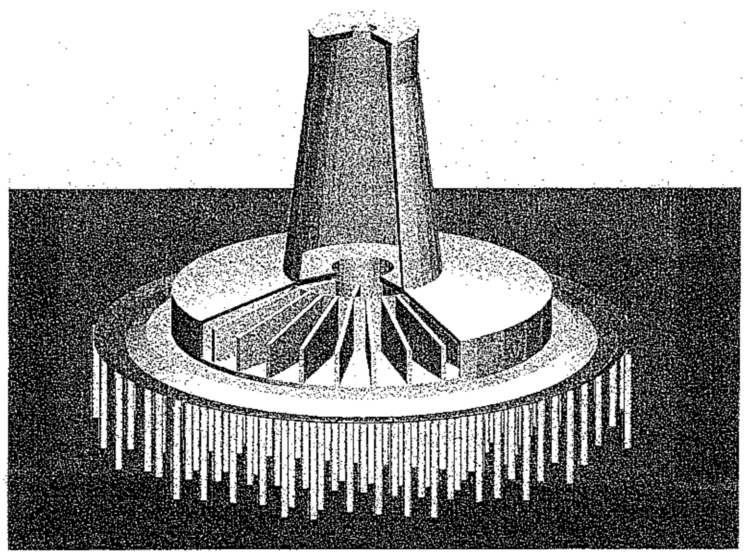


Figure 3. View of a reinforced soil foundation (Dumez-GTM).

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). 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 (figure 4).

Referring to equation (5), only the maximum resisting work P_m ($\underline{U}$) is modified with respect to the case without inclusions; it must include the contribution of the inclusions to the overall resistance of the soil-foundation system.

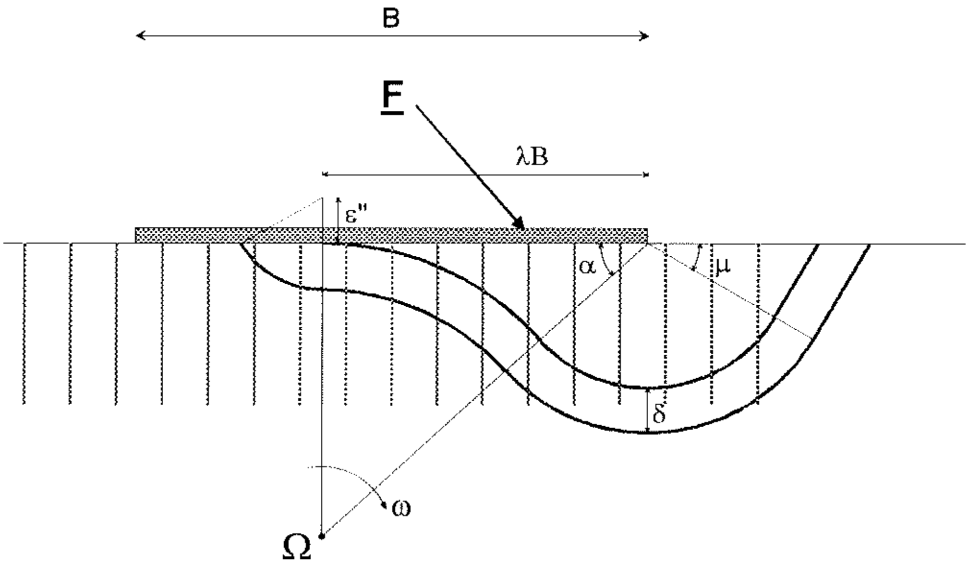


Figure 4. Kinematic mechanisms - Yield design theory.

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.

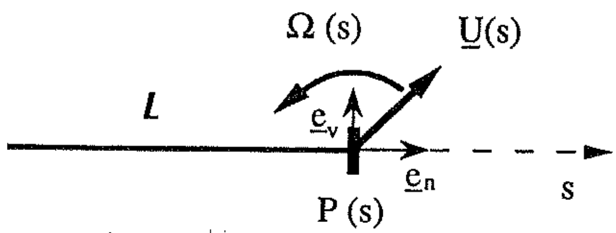


Figure 5. Virtual motion of an inclusion

The strength criterion for the inclusion is given by:

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

where n, v, m are the normal force, shear force and bending moment and n_ℓ, v_ℓ, m_ℓ are the ultimate values of n, v, m.

The following expression for the maximum resisting work per unit length of an inclusion, derived from (7) is:

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

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:

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

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 [5]) and minimization is performed on the geometric parameters defining the mechanisms to find the best upper bound.

These calculations can easily be programmed on a PC; numerical minimization on the five geometric parameters (α, μ, δ, λ, ε'') defining the kinematic mechanisms is efficiently performed with a downhill simplex method (Nelder & Mead 1965). Determination of an ultimate load, for a given configuration, does not take more than few tens of seconds.

3.2 *Example of application*

Let us take for illustration purposes the example presented in figure 3. The soil profile below the foundation consists of a clay layer with a shear strength increasing linearly with depth:

$$Su = 35 + 2.8\,z \qquad (10)$$

where Su 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.

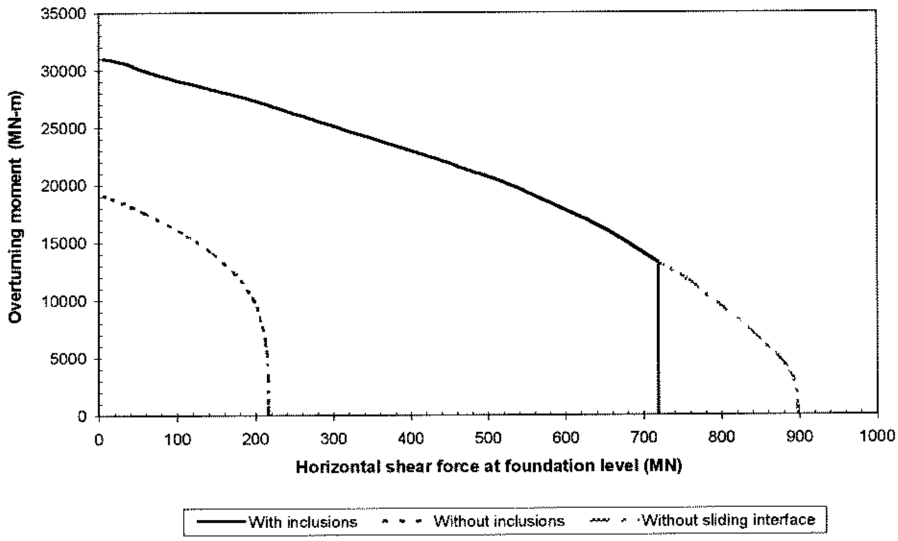


Figure 6. Bounding surface for a reinforced soil.

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:

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

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.

3.3 *Experimental and numerical validation of the concept*

The concept and theoretical evaluations presented above have been validated with model tests performed in a centrifuge and with non-linear finite element analyses.

3.3.1 *Numerical analyses*

Non-linear finite element analyses using the computer code DYNAFLOW (Prevost 1981) have been run under monotonic increasing loads up to failure. An elastoplastic Von Mises constitutive model, with kinematic hardening, for the clay and a Mohr Coulomb model for the ballast layer, have been used. Special contact elements with a limited shear capacity and no tensile strength connect the raft and the inclusions to the soil. All the runs confirmed, within ± 10%, the limit loads computed from the limit analyses, as shown for one particular example in figure 7.

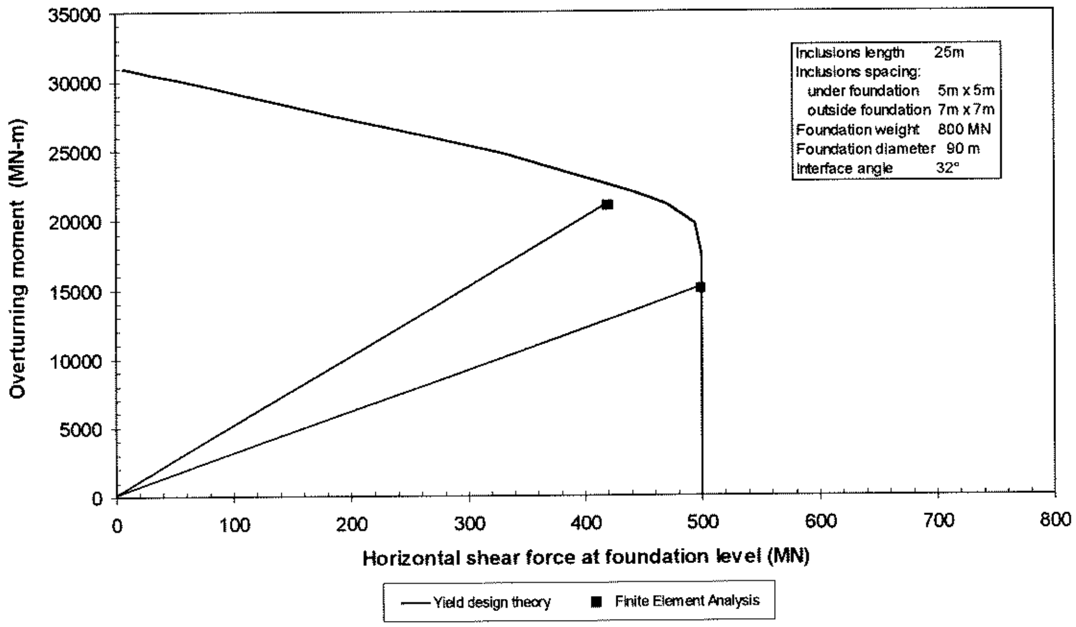


Figure 7. Comparison of finite element analyses and limit analyses.

In addition, the failure mechanisms found in the finite element analyses are alike the kinematic mechanisms of figure 4.

3.3.2 *Centrifuge model tests*

These tests have been run at 100 g on a modeled circular foundation with a diameter 0.30 m. Clay material coming from the site has been used for the soil model and reconsolidated using either a hydraulic gradient technique or a 1D compression test with a piston.

Two series of tests have been run:

- the first one is intended to check the validity of the theory; therefore, tests set up departing, even significantly from the foreseen one, have been investigated. The specimens have been loaded to failure under a monotonically increasing horizontal load and overturning moment;
- the second one is intended to check the behavior of the proposed reinforcement scheme under cyclic loads (overturning moment and horizontal load).

For both series of tests, comparison has been made with numerical evaluation obtained either from limit analyses or from finite element analyses.

Figure 8 compares the results of the five tests in the first series with numerical evaluations (limit analyses) in terms of ultimate loads. The computed ultimate loads are always smaller than the measured ones but the trend in the results is very consistent between tests and analyses. The discrepancies which, except for test n° 2, do not exceed 20% to 30% can be explained by the different hypotheses underlying both data: in the limit analyses, the ultimate load is computed assuming that changes in the initial geometry are negligible; in the tests, the ultimate load defined as the plateau of the load-displacement curve is attained for large displacements after considerable changes in the initial geometry have taken place (soil bulging in front of the foundation).

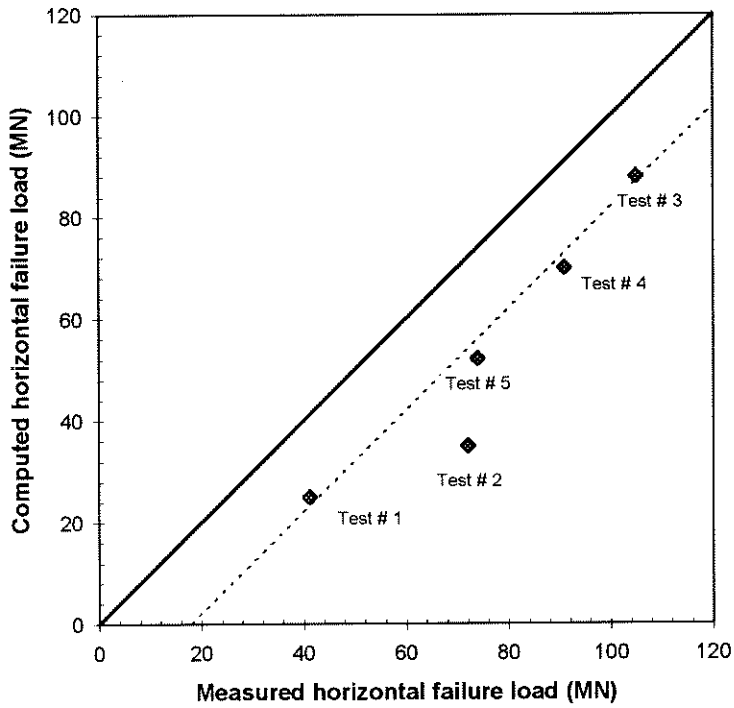


Figure 8. Comparison between measured and computed ultimate loads.

If the ultimate load is taken at the onset of the significant geometry changes, the agreement becomes very good (within 10%).

Figure 9 presents the results of a cyclic test with a force amplitude equal to 70% of the failure load. It is worth noting that, despite the large cyclic load, no degradation occurs, even after ten cycles. In addition, fat hysteresis loops develop, which reveal a high potential for energy dissipation. The equivalent damping ratio computed from these hysteresis loops as,

$$\beta = \frac{1}{4\pi}\frac{\Delta W}{W} \tag{12}$$

where:
ΔW: area of hysteresis loop,
W: elastic energy stored under the backbone curve,

is equal to 22%. Finite element analyses have predicted an equivalent damping ratio of 18%.

Figure 10 presents the monotonic load displacement curve obtained by loading the model to failure after the cyclic test. The first plateau on the curve, at 48 MN (prototype value) corresponds to a sliding at the soil foundation interface; for increasing displacements, the curve reaches a second plateau at 52 MN when the inclusions yield in bending. Significant changes in the geometry then take place, explaining the further load increase up to 56 MN.

Both values (48 MN and 52 MN) are accurately predicted by the theory. Note that, up to a horizontal displacement of 1.5 m, pure sliding takes place, without foundation rotation, which is a definite superiority of the concept over foundations on unreinforced soil, as explained in the following paragraph.

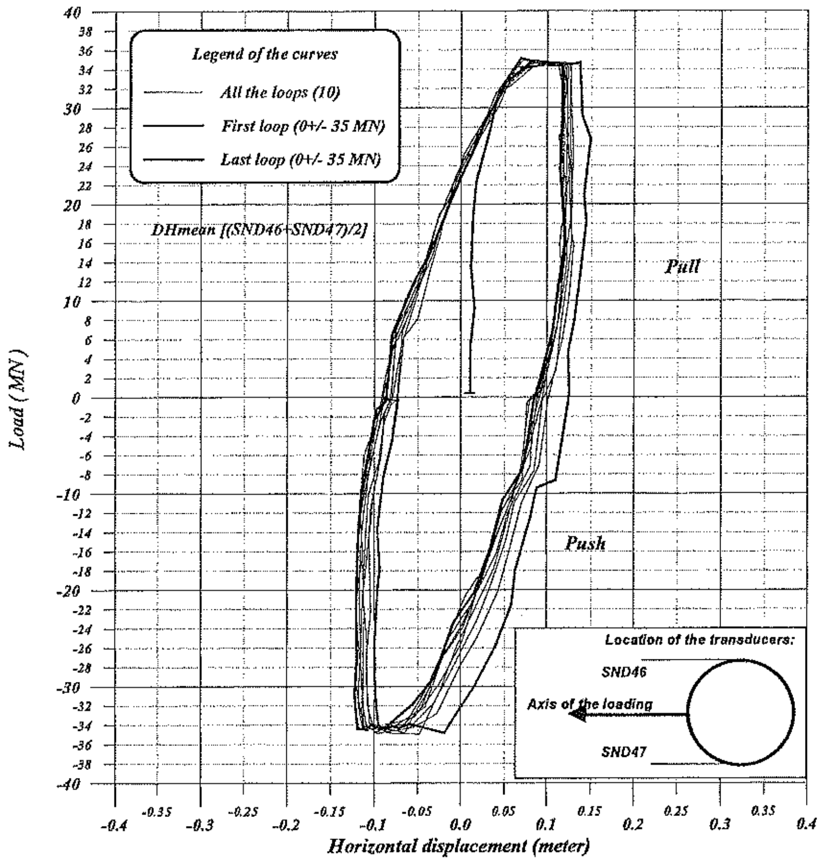


Figure 9. Hysteresis loops for cyclic load tests.

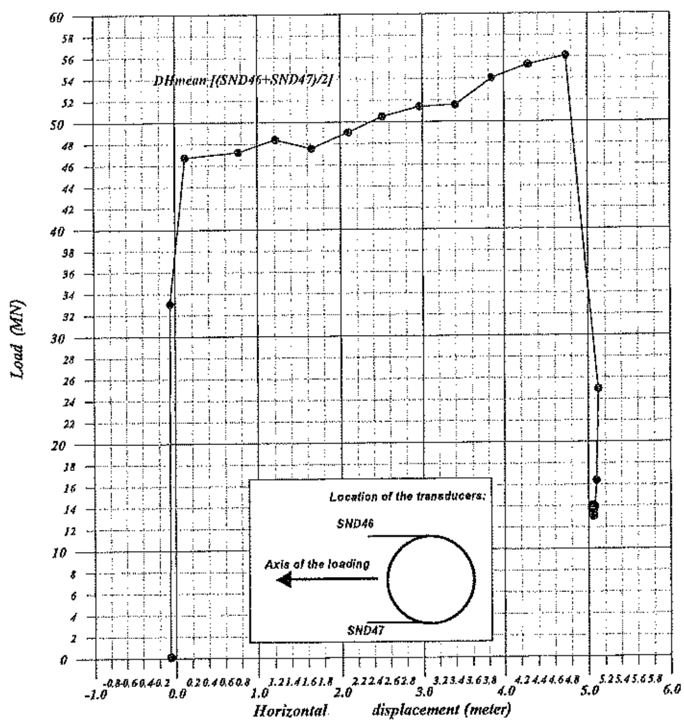


Figure 10. Load displacement curve for centrifuge test.

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 design 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 from the very beginning;
- 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, the vertical line at 720 MN would move to the right and the bounding surface would be represented by the dashed 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,

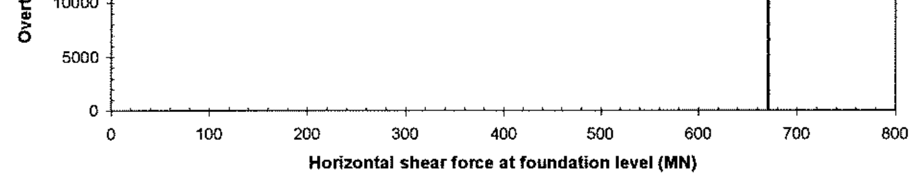


Figure 11. Influence of inclusion spacing on the shape of the bounding surface.

- the reinforcement increases the strength capacity with respect to undesirable failure modes, like rotational failure modes especially for tall structures. In addition, it provides an efficient means of dissipating energy, as evidenced by the results of cyclic centrifuge tests.

This increase in the foundation bearing capacity and the shape of the bounding surface is a function of the reinforcement scheme. Therefore, with a proper choice of the inclusions strength, spacing and length, the capacity design principle described above can be enforced for a wide range of load-moment combinations. Figure 11 shows, for instance, the influence of the inclusion spacing on the shape of the bounding surface: decreasing the spacing increases the moment for which rotational failure dominates over horizontal sliding failure from 15 000 MN.m to 18 000 MN.m.

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 capacity design philosophy in foundation engineering. It looks therefore very promising for increasing the safety of the structures 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.

7 REFERENCES

Auvinet, G., Pecker A., Salençon J. 1996. Seismic bearing capacity of shallow foundations in Mexico city during the 1985 Michoacan earthquake. *Proc. 11th World Conference on Earthquake Engineering*, Acapulco, paper n° 1966.

Budhu, M., Al-Karni, A. 1993. Seismic bearing capacity of soils. *Geotechnique*, 33, pp. 181-187.

de Buhan P. Salençon J. 1993. A comprehensive stability analysis of soil nailed structures - *European Journal of Mechanics A/Solids*, 12, n° 3, pp. 325-345.

Butterfield R., Gottardi G. 1994. A complete three-dimensional failure envelope for shallow footings on sands. *Geotechnique*, vol. 44, n° 1, pp. 181-184.

Kitazume M., Terashi M. 1994. Operation of PHRI Geotechnical Centrifuge from 1980 to 1994. *Technical note of the Port and Harbour Research Institute*, Ministry of Transport, Japan.

Nelder J.A., Mead R. 1965. Minimiztion in multidimensions with a downhill simplex method - *Computer Journal*, vol. 7, pp. 308-310.

Newmark N. 1965. Effect of earthquakes on dams and embankments - *Geotechnique*, vol. 15, n° 2, pp. 139-160.

Paolucci R. 1997. Simplified evaluation of earthquake induced permanent displacements of shallow foundations. *Journal of Earthquake Engineering*, vol. 1, n° 3, July 1997, pp. 563-579.

Paolucci R., Pecker A. 1997. Soil inertia effects on the bearing capacity of rectangular foundations on cohesive soils. *Engineering Structures*, vol. 19, n° 8, Elsevier Science Ltd., pp. 637-643.

Paolucci R., Pecker A. 1997. Seismic bearing capacity of shallow strip foundations on dry soils - *Soils and Foundations*, vol. 37, n° 3, pp. 95-105.

Paulay T. 1993. Simplicity and Confidence in Seismic Design - *The Fourth Mallet-Milne Lecture*, J. Wiley.

Pecker A., Salençon J. 1991. Seismic bearing capacity of shallow strip foundations on clay soils. CENAPRED - *Proceedings of the International Workshop on Seismology and Earthquake Engineering*, Mexico, pp. 287-304.

Pecker A., Auvinet G., Salençon J., Romo, M.P., Verzura L. 1995. Seismic bearing capacity of foundations on soft soils. Report to the European Commission. Contract CI1* CT92-0069.

Pecker A. 1997. Analytical formulae for the seismic bearing capacity of shallow strip foundations. *Seismic Behavior of Ground and Geotechnical Structures*, 1997, Seco e Pinto Editor. Balkema.

Pecker A., Salençon J. 1998. Innovative concepts in foundation Engineering. *Proceedings of the 2nd Japan-UK Workshop on Implications of Recent Earthquakes*. Technical report TIT/EERG 98-6. April 1998.

Pecker A., Teyssandier J.P. 1998. Seismic design for the foundations of the Rion-Antirion Bridge. *Proc. Institution of Civil Engineers, Geotechnical Engineering*, 131, pp. 4-11, paper n° 11311.

PREC8 - Prenormative Research in Support of Eurocode 8 - European Consortium of Earthquake Shaking Tables - Seismic behavior and design of foundations and retaining structures - Report n° 2. Editors E. Faccioli, R. Paolucci (July 1996).

Prevost J.H. 1981. DYNAFLOW - A finite element analysis program for the static and transient response of linear and non linear two and three-dimensional systems" - Department of Civil Engineering - Princeton University.

Richards R., Elms, D.G., Budhu, M. 1993. Seismic bearing capacity and settlements of shallow foundations. *ASCE Journal of Geotechnical Engineering*, 119, pp. 662-674.

Salençon J. 1983. Calcul à la rupture et analyse limite. *Presses de l'Ecole Nationale des Ponts et Chaussées.*

Salençon J. 1990. Introduction to the yield design theory. *European Journal of Mechanics A/Solids* 9, n° 5, pp. 477-500.

Salençon J. 1992. Approche théorique du calcul aux états limites ultimes. *Texte rédigé en hommage au Professeur Dautray.*

Salençon J., Pecker A. 1995. Ultimate bearing capacity of shallow foundations under inclined and eccentric loads. Part I: purely cohesive soil. *European Journal of Mechanics A/Solids*, 14, n° 3, pp. 349-375.

Salençon J., Pecker A. 1995. Ultimate bearing capacity of shallow foundations under inclined and eccentric loads. Part II: purely cohesive soil without tensile strength. *European Journal of Mechanics A/Solids*, 14, n° 3, pp. 377-396.

Sarma S.K., Iossifelis I.S. 1990. Seismic bearing capacity factors of shallow strip footings. *Geotechnique*, 40, pp. 265-273.