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# SEISMIC BEARING CAPACITY OF SHALLOW FOUNDATIONS IN MEXICO CITY DURING THE 1985 MICHOACAN EARTHQUAKE

G. AUVINET\*, A. PECKER\*\*, J. SALENÇON\*\*\*

\*Instituto de Ingeniería, UNAM, Cd. Univ., Apdo. Postal 70-472, Coyoacan, 04510, México, D.F.

\*\*Géodynamique et Structure, 157 Rue des Blains, 92220, Bagneux, France

\*\*\*Laboratoire de Mécanique des Solides, Ecole Polytechnique, 91128, Palaiseau Cedex, France

## ABSTRACT

During the 1985 Michoacan-Guerrero earthquake, a significant number of foundation problems were reported in Mexico City. Large permanent movements and, in some cases, actual bearing capacity failures affected floating piles foundations as well as shallow foundations. These observations were the motivation of an extensive research. The objective was to understand the behavior of foundations during earthquakes, to derive new bearing capacity formulae with due account of the soil inertia forces and to propose modifications to Mexico City seismic code.

On the one hand, the bearing capacity of strip shallow foundations has been derived within the framework of the yield design theory in which four independent loading parameters are applied to the foundation: the vertical force, the shear force, the overturning moment and the inertia forces within the soil mass. Simple analytical formulae were established which reflect the influence of the four parameters. Moreover, two simplified methodologies, based on different assumptions, were derived to compute the foundation permanent displacements.

On the other hand, information on buildings that experienced foundation failures was gathered and their seismic responses during the 1985 earthquake was computed with finite element analyses. These analyses yielded the applied seismic forces which were used together with the theoretical results to evaluate the seismic behavior of their foundations.

It appears that the effect of the soil inertia forces can be disregarded, provided the foundation exhibits an adequate safety with respect to static loads, and that permanent displacements cannot be tolerated because the foundation behavior becomes unstable.

## KEYWORDS

Shallow foundations. Yield design theory. Load eccentricity. Seismic bearing capacity. Permanent displacements. Case histories. Building code.

Failure or large permanent movements of a number of foundations on soft soil in Mexico City during the 1985 Michoacan earthquake caused deep concerns among soil engineers (Auvinet and Mendoza., 1986). It was considered necessary to thoroughly review the accepted analysis and design procedures and to introduce in the building code some new requirements guaranteeing a higher degree of safety for foundations. This proved more difficult than expected since it appears that while our knowledge concerning topics such as linear dynamic soil-structure interaction has greatly improved in recent years, the situation is much less satisfactory concerning failure or near failure behavior of soil-foundation systems. In the present paper, this problem is addressed. The results of some recent theoretical developments regarding seismic bearing capacity and permanent deformations are presented and their validity is evaluated analyzing several case histories. Some recommendations based on parametric studies are suggested for inclusion in the building code.

### THEORETICAL EVALUATION OF BEARING CAPACITY

Up to a very recent time, the seismic bearing capacity of surficial foundations was checked using classical bearing capacity formulae in which the seismic action is regarded as an equivalent static force. Load eccentricity and inclination were accounted for through coefficients modifying the usual bearing capacity factors or through reduction of the foundation width. Inertia forces within the soil mass were ignored. Recently, methods based on limit equilibrium analysis taking into account the soil inertia forces have been developed. Probably the most complete and rigorous solution to the problem has been given, at least for a cohesive soil, with or without tensile strength, by Pecker and Salençon (1991) and Salençon and Pecker (1995). Their solutions, which were recently extended to anisotropic materials (Pecker *et al* 1995, Pecker, 1996), provide upper and lower bounds estimates of the true bearing capacity. The loading parameters N (vertical force), T (shear force), M (overturning moment) and  $F_x$  (inertia force in the soil) are considered as independent loading parameters acting simultaneously on the foundation. Many different kinematic mechanisms, used in the framework of the yield design theory are studied to reach the best lower bound evaluation; some of these mechanisms can account for partial uplift of the foundation (Fig 1).

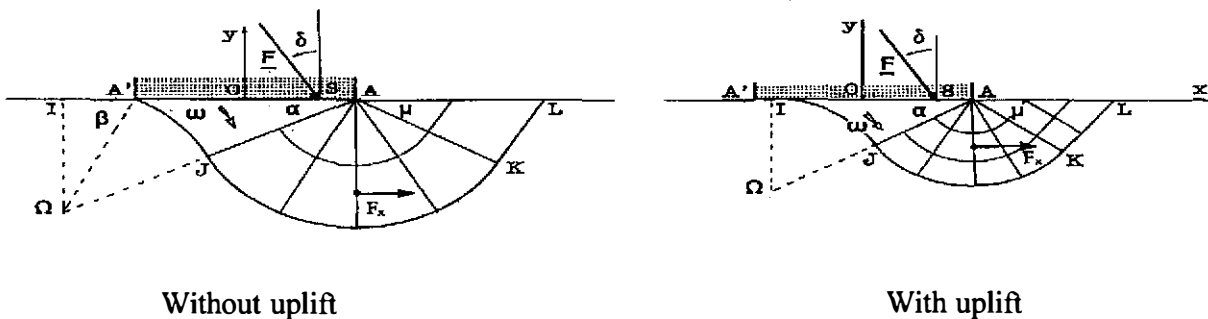


Fig 1. Examples of kinematic mechanisms

A convenient way to present the results is to normalize the loading parameters of the problem, that can be assembled in a vector:

$$\underline{F} = \left\{ \begin{array}{cccc} N & T & M & F_x B \\ BC & BC & B^2 C & C \end{array} \right\}$$

where B is the foundation width, C is the undrained strength of the soil and M is the overturning moment that can be replaced by the product of normal force N by load eccentricity e.

It could be established that for foundation for which  $N/CB$  is less than 2.5, i.e. for a foundation with a safety factor higher than 2 under a vertical centered load, the effect of the soil seismic forces can be neglected without loss of accuracy. On the contrary, for foundations with lower safety factors, these forces induce a dramatic reduction in bearing capacity.

In Fig 2, cross sections of the bounding surface containing the admissible loading parameters are presented for different  $e/B$  values. These results were obtained for a cohesive soil obeying Tresca's strength criterion without tensile strength. The surface was defined based on the kinematic approach of the yield design theory in the case where the condition ensuring that the effect of inertia forces are negligible is fulfilled.

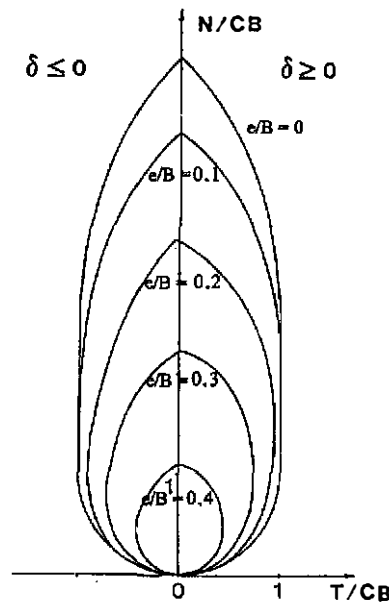


Fig 2. Cross sections of bounding surfaces

## ASSESSMENT OF FOUNDATION PERMANENT DISPLACEMENTS

A first approach consists of assuming that the soil-foundation system behaves as an elastic plastic system and that the boundary of the admissible loads defined previously, can be adopted as the boundary for the onset of plastic deformations. Using the theorem of kinetic energy, it can then be shown (Salençon and Pecker, 1991) that the angular velocity of the foundation around point  $\Omega$ , the center of rotation of the kinematic mechanism (Fig 1) can be written:

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

where  $K$  is a factor related to the geometry of the kinematic mechanism used,  $\rho$  is the mass density of the soil,  $B$  the foundation width,  $T^+$  is the maximum admissible load and  $T(\tau)$  is the time history of the shear force applied to the foundation. Integrating the above equation between  $t = t_0$ , time at which  $T(t_0) = T^+$  and  $t = t_1$ , such that  $\omega(t_1) = 0$ , the maximum permanent rotation of the foundation can be estimated.

A different approach was developed by Romo and García (1995) assuming that development of permanent displacements of a foundation during an earthquake is an accumulative process. When the soil-structure system is submitted to an irregular cyclic displacement history, failure surfaces can develop during short periods of time in opposite directions and the structure presents alternate rigid body rotations (Fig 3). The foundation thus presents a zigzag movement with accumulation of total and differential movements. If a

convenient shape of the failure surface is assumed, permanent displacements can be estimated integrating the dynamic movement equation in the time domain by Newmark's Method.

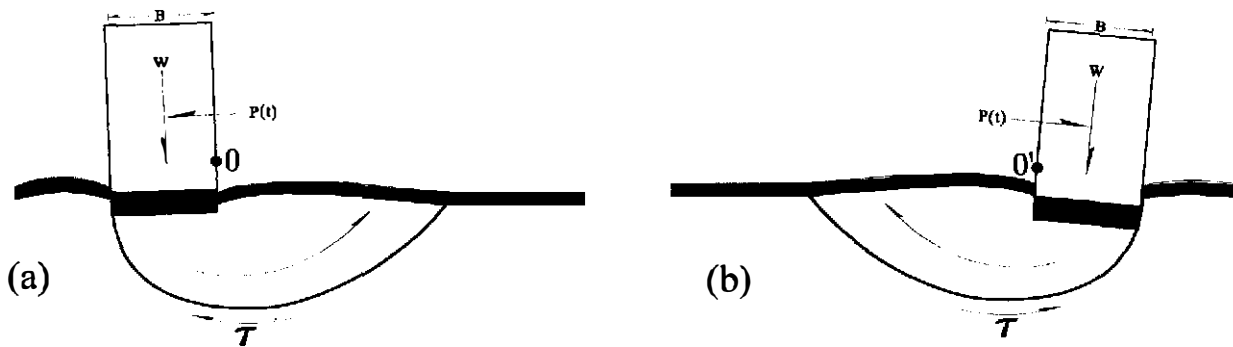


Fig 3 Concept of permanent settlements as an accumulative process

### APPLICATION TO CASE HISTORIES OF FOUNDATIONS FAILURES IN MEXICO CITY

Data concerning several case histories of foundation failures were collected after the 1985 earthquake (Auvinet and Mendoza, 1986). Two of these cases were selected for analysis and testing of the theoretical models.

#### a) CASE 1. School building

This building is a long reinforced concrete structure with partially compensated foundation located near Mexico City airport. In-plane dimensions of the construction are 54.5m (East-West) and 11.6m (North-South). The foundation is a continuous slab at a depth of 3.0m. The water table is found at a depth of 1.6m. This box type foundation was found practically filled of water after the earthquake. The average net pressure transmitted to the ground in static conditions was 25 kPa, with a uniform distribution along the building. However, at the West end of the building, two cisterns containing 15 cubic meters of water each, water tanks on the roof, and other equipment, induced an additional load and the stress probably reached  $q = 30\text{kPa}$ .

As a consequence of the 1985 earthquake, notable differential settlements as well as north-westward tilting were observed. In the transversal direction, differential settlement was 0.52m at the West end and 0.32m at the East end; in the longitudinal direction, surveys showed a differential settlement of 0.26m. The maximum tilt in the north-west corner was 2.9%. When the building was demolished, the rigid-body rotation of the foundation box around the longitudinal axis could be clearly appreciated.

The supporting soil is a very soft clay with a conventional static shear strength of 20 kPa. According to laboratory tests (Romo and Auvinet, 1991) the "dynamic" undrained strength is thus likely to be in the range 25-30kPa. Consequently the factor of safety is of the order of 3.0 ( $SF=(\pi+2)C/q$ ) and according to the results commented previously, soil seismic forces should not bear any significant influence on the foundation.

The seismic forces were computed from a dynamic finite element soil structure interaction model for the North-South component. The acceleration time history at the centre of gravity of the building was determined. The maximum value is 0.37g. The horizontal shear force was obtained multiplying the accelerations by the mass of the building. The maximum force is 0.32MN/ml. The overturning moment is equal to the horizontal shear force multiplied by the lever arm, i.e. the elevation of the centre of gravity of the building,  $H = 7\text{m}$ . Its maximum value is 2.26MN.m/ml. To perform the pseudo-static bearing capacity

analysis, the maximum horizontal force, applied at elevation +7m, that the 11.6m wide foundation can sustain was computed as a function of the soil undrained shear strength  $C$  using the theoretical model. Seismic inertia forces in the soil corresponding to a seismic coefficient of 0.15 were included but, as expected, their influence proved to be negligible (less than 3%) from an engineering standpoint. The applied load becomes equal to the maximum allowable load for a soil strength  $C = 31.5\text{kPa}$ .

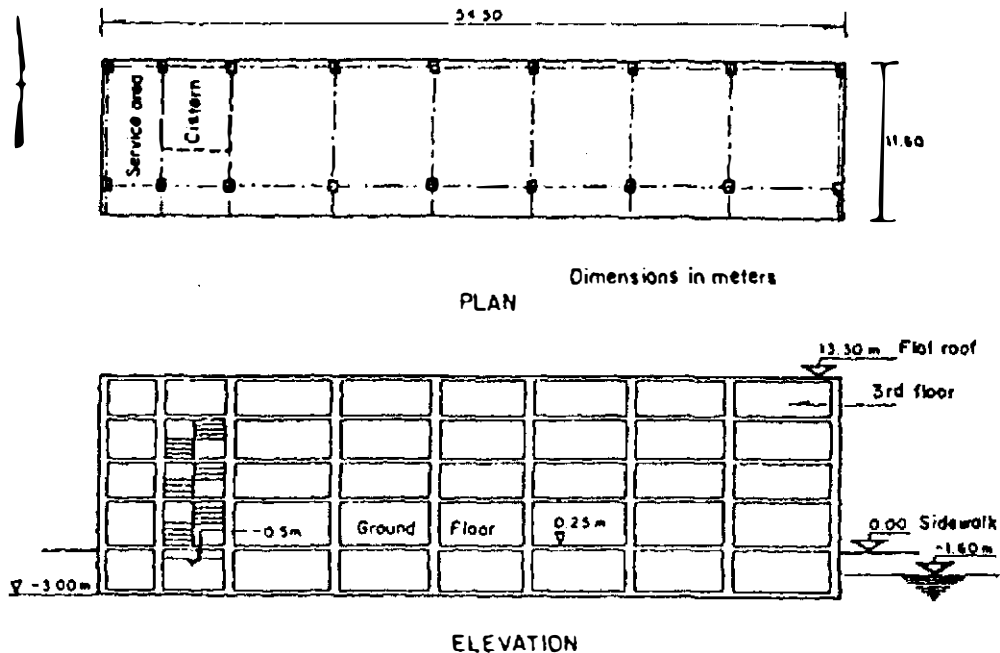


Fig 4 Case 1

The permanent displacements of the foundation were computed from the shear force history  $T(\tau)$ , following the first methodology exposed. The foundation tilt could be accurately predicted while the foundation settlement was slightly underestimated.

#### b) CASE 2. Residential Building, Colonia Roma

This is a 18.6m high six stories apartment building. The dimensions of the construction are 14.10m along the North-South direction and 16.75m along the East-West direction. The foundation consists of a concrete slab, 0.2m thick, at a depth of 1.2m below the sidewalk level. The building transmits to the soil an average net pressure of 55kPa. The original load distribution was fairly uniform, no significant eccentricity was detected. No record of the initial configuration is available; however, it can be estimated that the Northeast front of the building already presented a settlement of 0.65m before the earthquake.

The total settlement measured on the same side of the building after the earthquake was 1.57m; a settlement of the order of 0.92m can thus be attributed to the seismic event. The total Eastward tilt was 5.2%.

The upper layer of the soil profile is a 4m-thick fill. Below this material, a volcanic clay layer extends down to 32m. The water content of this soft soil varies between 250 and 380%. The average conventional “static” undrained shear strength is 25kPa. The “dynamic” shear strength can be assumed to lie between 30 and 35 kPa.

Disregarding the fill layer, the safety factor against a static bearing capacity failure is of the order of 2.3 and consequently, the soil seismic forces should not have a pronounced effect on the ultimate horizontal load.

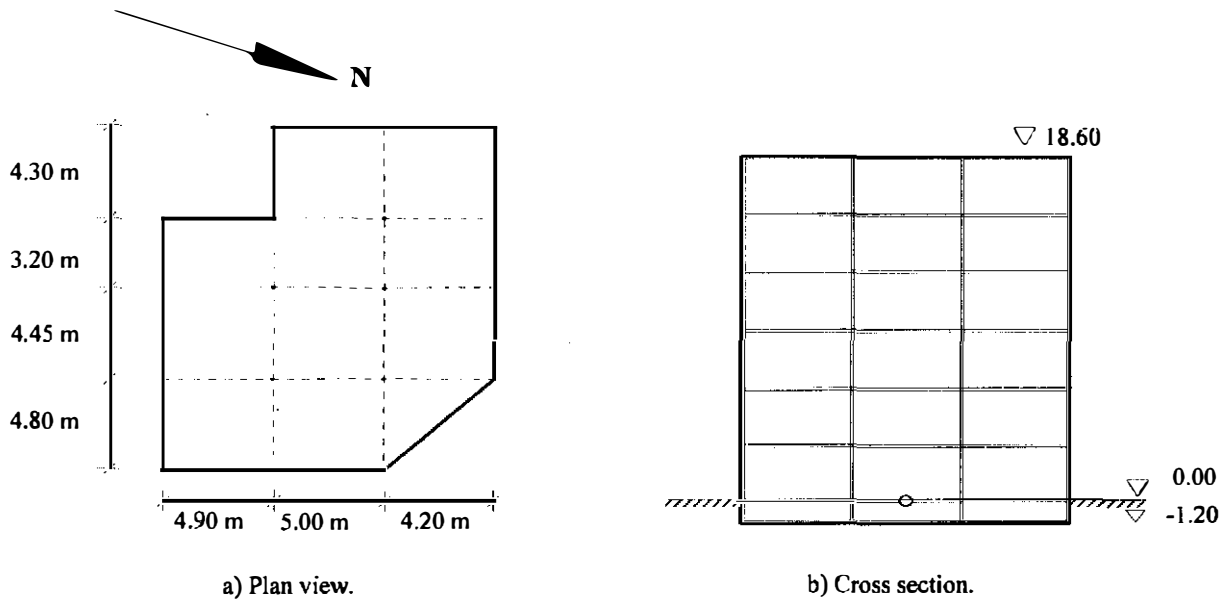


Fig 5 Case 2

This case history does not strictly conform to the theoretical model of a strip foundation. However, it is known that for a square or circular footing on a clay soil, the increase in the bearing capacity is rather limited, of the order of 1.2 for a vertical centred load. For an inclined, eccentric load, the increase is even smaller. The analysis was performed along the East-West direction.

The seismic forces were computed from a dynamic finite element soil structure interaction model for the North-South component. The acceleration time history at the centre of gravity of the building was shown to reach a maximum value of 0.48g. The horizontal shear force was obtained multiplying the accelerations by the mass of the building. The maximum force is 0.58MN/ml. The overturning moment, equal to the horizontal shear force multiplied by the lever arm, i.e. the elevation of the centre of gravity of the building, reaches a maximum value of 7.54 MN.m/ml. Again, to perform the pseudo-static bearing capacity analysis, the maximum horizontal force, applied at elevation of the centre of gravity, that the 16.75m wide foundation can sustain was computed as a function of the soil undrained shear strength  $C$  using the theoretical model. Seismic inertia forces in the soil corresponding to a seismic coefficient of 0.15 were also included; in this case their influence also proved to be small (less than 5%) but larger than in the previous case due to the lower safety factor. The applied load becomes equal to the maximum allowable for a soil strength clearly larger than the estimated value, which can explain why this particular building presented such a dramatic failure.

The same procedure as in Case 1 was used to estimate permanent displacements. The foundation tilt was slightly overestimated, whereas the foundation settlement (middle of foundation) was significantly underestimated.

This same building was analyzed, together with four additional constructions, by Romo and García (1995) using the methodology described previously. To cover the uncertainty on cohesion, a parametric study of the displacements in the North and South edges of the building was performed. Good agreement was obtained for the expected value of cohesion.

It should be recognized that permanent displacements result from a highly chaotic process that depends on secondary factors such as duration of the earthquake, initial tilt, soil heterogeneity, and others and, as such, cannot be accurately predicted. However, the models described in this paper are useful to evaluate in which conditions these movements can become significant. They can be used for parametric studies from which simple rules are derived to include provisions in design to avoid permanent movements.

The first results of this kind of studies show that design for a limited amount of permanent displacements is not acceptable since the behavior of the building becomes highly unstable. It also appears that load eccentricity should never exceed 0.3 B since for higher values large permanent movements associated to overturning of the building can be expected.

## CONCLUSIONS

Models describing the limit behavior of soil-foundation systems in seismic conditions have been developed and their predictions were compared to actual performance of buildings that presented large permanent movements during the 1985 Michoacan earthquake in Mexico City. A good agreement was obtained.

According to the models, it appears that inertia forces in the soil can be neglected in bearing capacity analyses, whenever the static factor of safety is reasonably large ( $>2$ ). For a foundation with a lower safety factor, these forces induce a dramatic reduction in the bearing capacity.

Parametric studies also show that it is possible to define a design domain in terms of variables such as static safety factor, pseudo-static factor and eccentricity within which permanent displacements of the foundation can be avoided. The incidence of such results on future regulations in Mexico City are expected to be significant.

## ACKNOWLEDGMENT

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