SEISMIC BEARING CAPACITY OF SHALLOW STRIP FOUNDATIONS ON CLAY SOILS

A. Pecker[1] and J. Salençon [2]

ABSTRACT

The seismic bearing capacity of shallow spread foundations is determined within the framework of the yield design theory. The supporting medium obeys Tresca's isotropic strength criterion with a cohesion C and with or without resistance to traction. The load vector is composed of four independent components N (vertical force), T (horizontal force), M (moment), fx (inertia forces in the soil).

At first, the static extreme loads for the foundation-soil system are determined from the static and the kinematic approaches of the yield design theory. New kinematic mechanisms are proposed, which are used for the evaluation of the seismic bearing capacity. For the computation of the latter, the foundation-soil system is then assumed to behave as an elastic perfectly plastic system. The dynamic bearing capacity is determined in terms of allowable residual displacements for the foundation. Charts are presented in terms of dimensionless parameters for the static and seismic bearing capacities of foundations.

INTRODUCTION

Little attention has been paid to-date to the ultimate bearing capacity of foundations under seismic excitation. The main justification is that bearing capacity failures are not very often observed during earthquakes, provided that liquefaction of the soil foundation does not occur. However, in seldom cases, such as in Mexico city during the 1985 Michoacán earthquake, failures did occur with dramatic consequences for the supported superstructures.

The state of practice in evaluating the bearing capacity of foundations during earthquakes consists in applying conventional bearing capacity formulae with reduction coefficients accounting for the load excentricity and inclination arising from the inertial forces developing in the super-structure. In such an approach, fundamental aspects are ignored [3]: inertial forces developed in the soil by the passage of the seismic waves are not included in the analyses; earthquake loads are treated as permanent loads, either in space or in time, although a temporarily overload on the foundation does not mean failure but rather permanent displacements.

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It is the purpose of this paper to present the general framework for analyzing the dynamic and seismic bearing capacity of strip shallow foundations resting on a homogeneous clay layer whose strength criterion obeys Tresca criterion ($\phi=0$ concept) with or without resistance to traction. Solutions are derived on the basis of the yield design theory ([5], [6]), mainly its kinematic approach.

First, new results obtained for the bearing capacity under excentric and inclined static loads are briefly summarized; these results are extended to include the effect of the inertial forces in the soils; finally, the permanent displacements of the foundation are evaluated to define "failure".

DEFINITION OF THE PROBLEM AND METHOD OF ANALYSIS

Generalities

An infinite rigid foundation of width B lies on the surface of a homogeneous halfspace. The foundation is subjected to three independent external forces defining the load vector Q: the axial force N, the tangential force T and the overturning moment M (fig.1). This load vector represents the external forces transmitted from the super-structure onto the foundation, for instance, the inertial forces developing during an earthquake. The problem to be solved is the determination of all possible combinations of N, T and M which the foundation can withstand.

The solution is searched for in the framework of the yield design theory ([5], [6]). The supporting halfspace is modelled as a three-dimensional continuum characterized only by its strength criterion which defines local yielding.

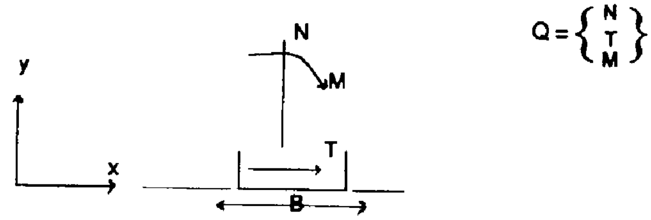

FIGURE 1. Geometry of the system

For the present study, the material is assumed to obey Tresca strength criterion with or without resistance to traction. The domains $G(\underline{x})$ of the corresponding allowable stress states are represented in the principal stress space on fig.2.

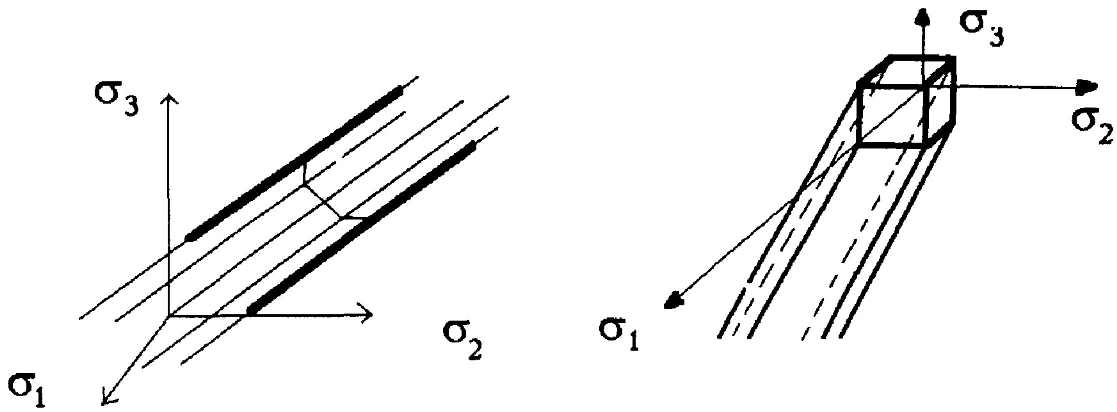

FIGURE 2. Strength criteria ([5])

Their equations are given by

- Material with resistance to traction:

$$f(\underline{\underline{\sigma}}(\underline{x})) = \sup_{i,j=1,2,3} \{\sigma - \sigma - 2C\}$$

- Material without resistance to traction:

$$f(\underline{\underline{\sigma}}(\ )) = \sup_{i,j=1,2,3} \{\sigma - \sigma - 2C, \sigma_i\}$$

where C is the undrained shear strength (cohesion) of the soil.

The domain G(x) of allowable stress tensors (x) is defined by:

$$f(\underline{\underline{\sigma}}(x)) \leq 0$$

where $\underline{x}$ stands for the coordinate vector; tensile stresses are counted positive.

The interface between the foundation and the soil is perfectly rough without resistance to traction.

Although the soil is a heavy material, it is proved that the effect of the vertical body forces in the soil (gravity) can be disregarded in the analysis. If $\underline{\underline{\sigma}}$ is the stress tensor solution of the problem for the weightless soil, the stress tensor solution of the problem for the heavy soil is given by:

$$\underline{\underline{\sigma}} = \underline{\underline{\sigma}}' + f_y \cdot y \cdot \underline{\underline{1}}$$

where f_y stands for the gravity force per unit volume counted positive upwards ^{y}

The indefinite equilibrium equations and boundary conditions are satisfied by $\underline{\underline{\sigma}}'$ and the strength criterion is not affected by the change of variables defined by eq.4. The same result is valid for any constant vertical body force; it therefore applies also to vertical earthquake forces which need not be considered in the analysis.

Method of analysis

The presentation, based on the more general one given in [5] and [6], will be restricted to the main results relevant to the present study.

Let us define the geometry of the soil-foundation system as Ω with boundary S. Let $\underline{\underline{\sigma}}$ be a stress field at point x, U a velocity field with the associated strain rate field $\underline{\underline{d}}$; let $[\![\underline{U}]\!]$ denote the jump of $\underline{U}$ when crossing a velocity discontinuity surface at point x following the normal $\underline{n}(\underline{x})$. The load vector is given as $\underline{Q}$ and q is the kinematic vector associated with $\underline{Q}$ when expressing the work of external forces in a kinematically admissible velocity field $\underline{U}$.

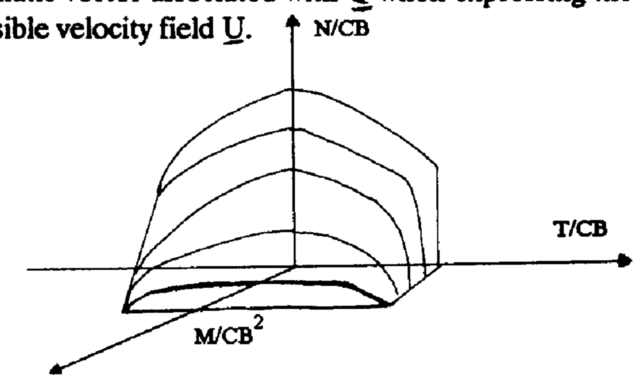

FIGURE 3. 3D representation of K

The domain K of the potentially stable loads (fig. 3) will be approximated from the outside using the kinematic approach of the yield design theory, derived from the principle of virtual works which states:

. for any statically admissible stress field $\underline{\underline{\sigma}}$ in equilibrium with $\underline{Q}$,

. for any kinematically admissible velocity field $\underline{U}$,

$$\int_{\Omega} \underline{\underline{\sigma}}(\underline{x}) : \underline{\underline{d}}(\underline{x})\, d\Omega + \int_{\Sigma} [\underline{U}(\underline{x})] \cdot \underline{\underline{\sigma}}(\underline{x}) \cdot d\Sigma = \underline{Q}(\underline{\underline{\sigma}}) \cdot q(\underline{U})$$

where . and : represent simple or double contraction on tensors.

Introducing the π (.) functions with the following definitions:

$$\pi(\underline{x}; \underline{\underline{d}}(\underline{x})) = \mathrm{Sup}\{ \underline{\underline{\sigma}}(\underline{x}) : d(\underline{x}) \mid \underline{\underline{\sigma}}(\underline{x}) \in G(x) \}$$

$$\pi(\underline{x}; \underline{n}(\underline{x}), [U(\underline{x})] = \mathrm{Sup}\{ [U(\underline{x})] \cdot \underline{\underline{\sigma}}(\underline{x}) \cdot \underline{n}(x) \mid \underline{\underline{\sigma}}(\underline{x}) G(\underline{x})\}$$

for any virtual velocity field U the maximum resisting work is expressed by:

$$P(U) = \int_{\Omega} (x;\ d\ (x))\, d\Omega + \int_{\Sigma} \pi(x; n(x), [U(x)])\ d\Sigma$$

which yields to the statement that for any potentially safe load Q - K and for any kinematically velocity field U and associated q (U)

$$Q \cdot q(U) \leq P\ (U)$$

The method therefore consists in constructing kinematically admissible velocity fields and in minimizing P(U) to obtain the better approximation to K (fig.4).

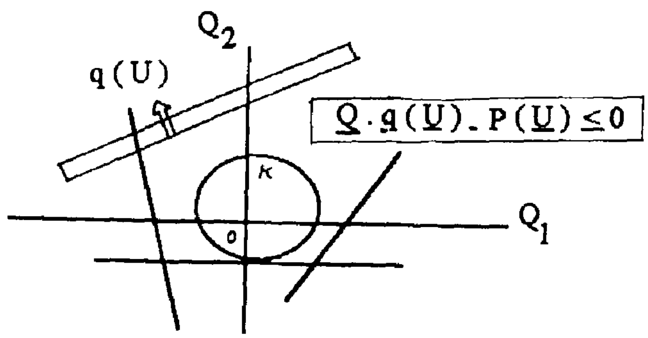

FIGURE 4. Kinematic approach "from outside" [6]

Application of the kinematic approach requires the knowledge of the π (.) functions and the construction of kinematic mechanisms. The latter are presented in the following paragraph. The π (.) functions are calculated in [5] and recalled hereafter:

- Material with resistance to traction:

in the volume Ω :

$$\pi(\underline{\underline{d}}) = +\infty \quad \text{if } tr(\underline{\underline{d}}) \neq 0$$

$$\pi(\underline{\underline{d}}) = C \left[|d_1| + |d_2| + |d_3| \right] \quad \text{if } tr(\underline{\underline{d}}) = 0$$

. along the surfaces Σ of velocity discontinuities:

$$\pi\,(\underline{n}\,;\,[\![\,\underline{U}\,]\!]) = +\infty \quad \text{if} \quad [\![\,\underline{U}\,]\!]\cdot\underline{n} \neq 0$$

$$\pi\,(\underline{n}\,;\,[\![\,\underline{U}\,]\!]) = C\,|[\![\,\underline{U}\,]\!]| \quad \text{if} \quad [\![\,\underline{U}\,]\!]\cdot\underline{n} = 0$$

- Material without resistance to traction:

. in the volume Ω

$$\pi(\underline{\underline{d}}) = +\infty \quad \text{if} \quad tr(\underline{\underline{d}}) < 0$$

$$\pi\,(\underline{\underline{d}}) = C\left[\,|d_1| + |d_2| + |d_3| - tr(\underline{\underline{d}})\right] \quad \text{if} \quad tr(\underline{\underline{d}}) \geq 0$$

. along the surfaces Σ of velocity discontinuities:

$$\pi\,(\underline{n}\,;\,[\![\,\underline{U}\,]\!]) = +\infty \quad \text{if} \quad [\![\,\underline{U}\,]\!]\cdot\underline{n} < 0$$

$$\pi\,(\underline{n}\,;\,[\![\,\underline{U}\,]\!]) = C\left[\,|[\![\,\underline{U}\,]\!]| - [\![\,\underline{U}\,]\!]\cdot\underline{n}\right] \quad \text{if} \quad [\![\,\underline{U}\,]\!]\cdot\underline{n} \geq 0$$

- for the soil-foundation interface in both cases:

$$\pi\,([\![\,\underline{U}\,]\!]) = +\infty \quad \text{if} \quad [\![\,\underline{U}\,]\!]\cdot\underline{n} < 0$$

$$\pi\,([\![\,\underline{U}\,]\!]) = C\,\left|[\![\,\underline{U}\,]\!] - ([\![\,\underline{U}\,]\!]\cdot\underline{n})\,\underline{n}\right| \quad \text{if} \quad [\![\,\underline{U}\,]\!]\cdot\underline{n} \geq 0$$

SUMMARIES OF SIGNIFICANT NEW RESULTS FOR THE STATIC CASE

Aside from the seismic aspect of the problem, it appeared necessary in the course of the study to develop new solutions for the static, excentrically, inclined load. As mentioned previously, attention was focused on the kinematic approach; however, the static approach was also used to assess the validity of the solutions. Since the static problem is out of the scope of the present paper, only the final results will be recalled. All the details of the derivation can be found in [4].

Among the tested kinematic mechanisms, the most significant ones for the dynamic analyses are presented in more details hereafter. These two mechanisms yield the better approximation of K for a positive (downward) normal force and positive horizontal force and moment (fig.1); this combination of forces corresponds to the case of a foundation initially loaded by a vertical centered force and subjected to an increasing horizontal force applied at an elevation H above its base.

Both mechanisms are basically identical; they only differ by the position of the center of instantaneous rotation Ω (fig.5).

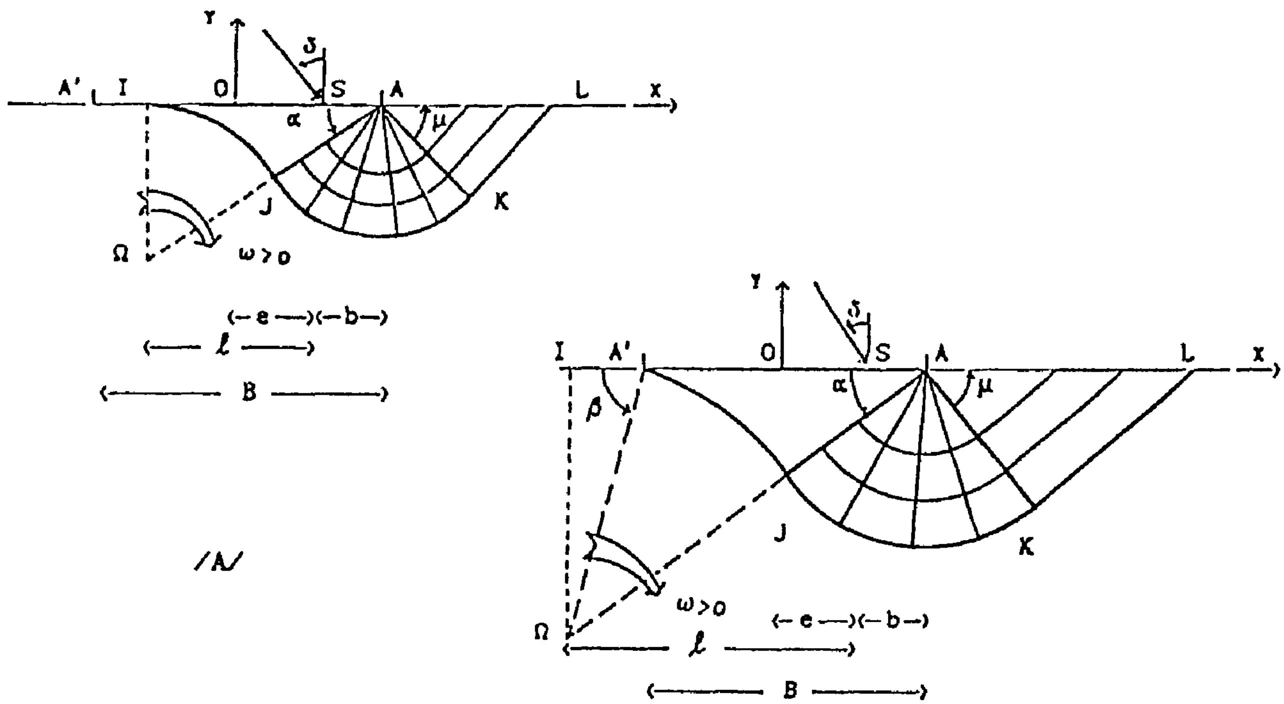

FIGURE 5. Kinematic mechanisms

Mechanism A in fig.5 is associated to: /B/

- a rotation, with angular velocity ω, of the system composed of the foundation A'A and of the soil volume IJA; the center of rotation is Ω,

- an uplift of the foundation along A'I,

- a purely tangential velocity field in the circular sector AJK and in the triangle ALK.

Mechanism B presents a center of rotation whose projection on the surface of the halfspace lies outside the foundation A'A. It differs from mechanism A since there is no longer a contribution from the uplifted zone to the value of the maximum resisting work (eq.14).

These mechanisms depend on three independent parameters which are defined in fig. 5: α, μ and λ = (+ b)/B for mechanism A; α, β and μ for mechanism B. The maximum resisting work is a function of these three parameters; for instance, for mechanism A, its analytical expression for a material with resistance to traction is given by:

$$P(\underline{U}) = \omega\ C\ B^2\ \lambda^2 \left[\frac{\pi}{2} - \mu + \frac{1}{2} \tan\mu + (\frac{\pi}{2} - \alpha)\ \frac{1}{\cos^2\alpha} \right]$$

$$+ \omega\ C\ B^2\ \lambda\ (1 - \lambda)\ \tan\alpha$$

Because of space limitations, the expressions for mechanism B or for a material without resistance to traction cannot be reproduced. They are found in [4].

The external work in the rotational movement is given by:

$$\underline{Q} \cdot \dot{\underline{q}} (\underline{U}) = \omega \; [\ell \; N + T \; (\ell + b) \; \tan \alpha]$$

Inserting eqs. 15 and 16 in eq.9 yields the following upperbound for T,

$$\frac{T}{CB} \leq \phi \; (r_1, \; r_2, \; r_3, \; \frac{H}{B}, \; \frac{N}{CB})$$

where the r_i stand for independent expressions in the parameters of the considered mechanisms.

The function ϕ (.) has then to be minimized on the parameters r_i $(i=1,2,3)$.

The choice of the form of eq. 17 results from the fact that for the seismic analyses, the vertical Force N and the elevation H of the horizontal force can be considered as approximately constant.

The results of the minimizations given by eq.17 for both mechanisms, and for others not reproduced herein, are summarized on fig.6 for a material with resistance to traction and on fig.7 for a material without resistance to traction. These figures depict the projection onto the (N, T) plane of the domain K (fig.3); the curves are graduated as functions of the load excentricity e/B.

The following conclusions were drawn from these results in [4]. For a material with resistance to traction and a zero excentricity, the static and kinematic approaches yield coincident solutions for a zero load inclination and for an inclination greater than 7°; between zero and 7°, they differ by only a few percent. When the load excentricity increases, the difference between both approaches increases; however, from a practical standpoint, where the normal load is equal to 5.14 C.B/FS with FS being the safety factor (FS 3) and the excentricty remains moderate (e/B 0.25), the difference is small and justifies the use of the kinematic mechanisms in the following. It has also been shown that the usual rule for excentric loads which consists in substituting the foundation width B by a reduced width B':

$$B' = \left(1 - 2 \; \frac{|e|}{B}\right) B$$

is erroneous and overconservative: the point (N/CB=0, T/CB= 1) always belongs to the domain K whichever the excentricity.

For a material without resistance to traction, the same conclusions on the validity of the solutions still apply. However, the use of he concept of reduced width (eq.18) overestimates the actual bearing capacity.

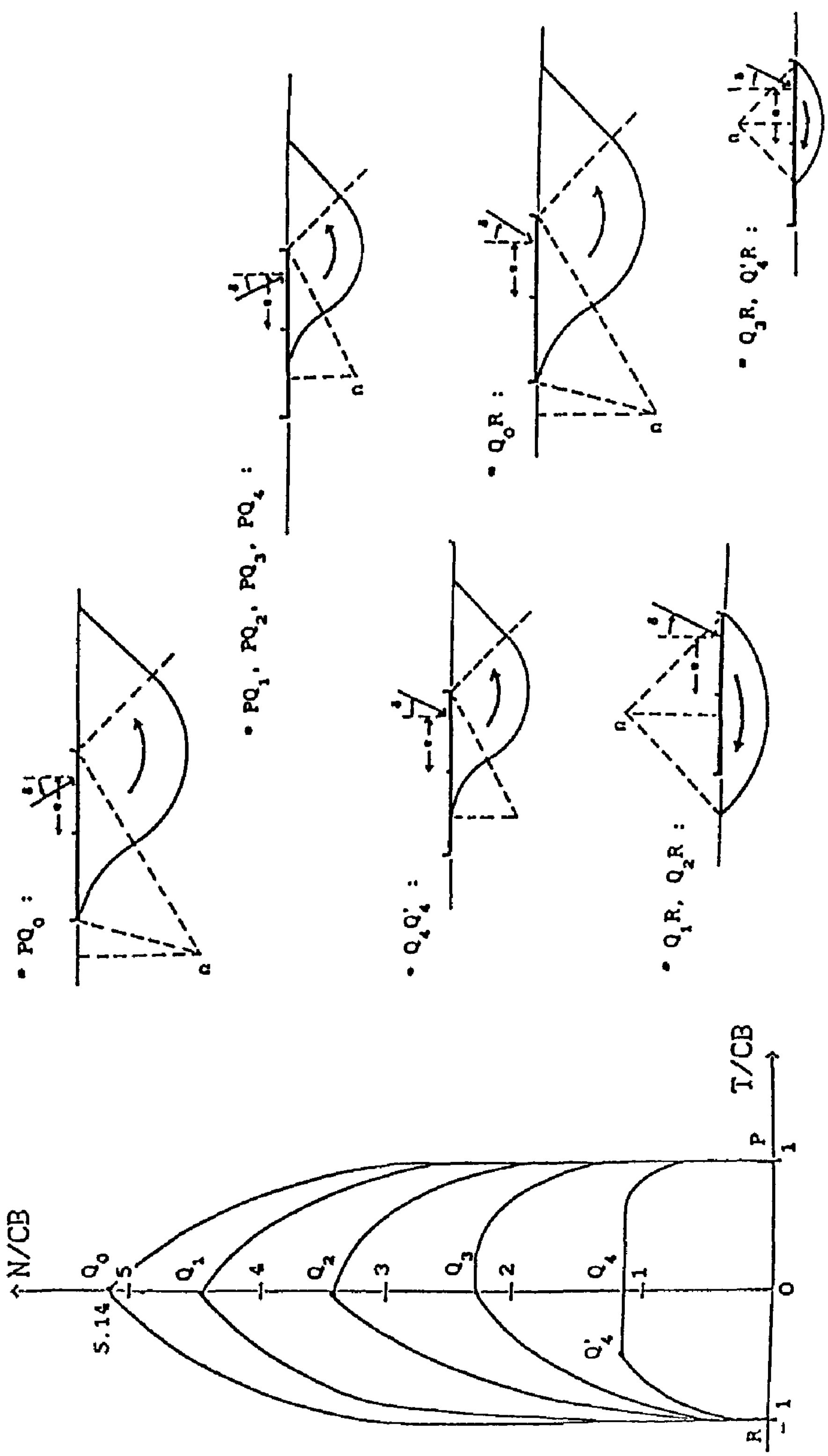

FIGURE 6. Approximation of the domain K from outside
Material with resistance to traction

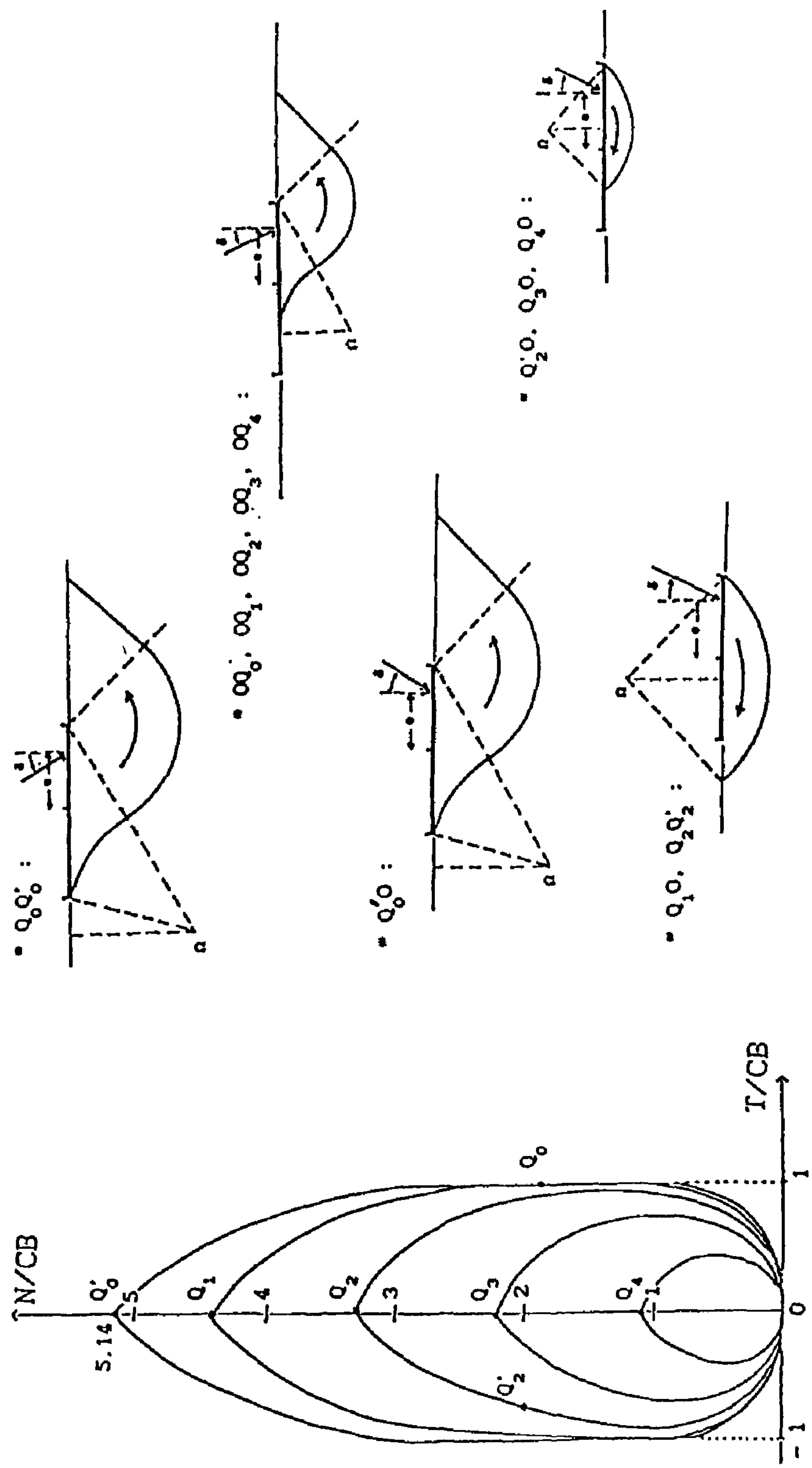

FIGURE 7. Approximation of the domain 𝕂 from outside
Material without resistance to traction

SEISMIC BEARING CAPACITY

In this paragraph, the effect of the inertia forces developed in the soil by the passage of the seismic waves are incorporated in the analysis to define a new domain K for the potentially stable loads.

Definition of the seismic excitation

As mentioned previously, only the horizontal seismic force will be considered, since the vertical one does not enter in the definition of K. We will use the common assumption in earthquake engineering which considers the horizontal motion caused by the vertical propagation of horizontally polarized shear waves. Under these conditions and provided the soil deposit presents only vertical heterogeneities, the seismic force depends solely on the vertical coordinate. We will further consider this force constant, which is approximately valid over some depth; if h is the thickness of the clay deposit overlying the bedrock, it can be shown ([4]) that this hypothesis is valid for depths of the order of h/10.

We will note the amplitude of the constant horizontal seismic force as f_x in the following; $f_x\ e_x$ represents the associated vector where e_x is the unit vector of the x axis.

Evaluatian of the seismic bearing capacity

The two kinematic mechanisms presented on fig.5 are used for this evaluation. The maximum resisting work P(U) developed in the velocity field U is the same as for the static case.

However, the work of the inertia forces in the soil must be included in the derivation of the work of the external loads. Equation (16) is modified accordingly:

$$\underline{Q} \cdot \underline{\dot{q}}\ (\underline{U}) = \omega\ [\ell\ N + T\ (\ell + b)\ \tan\alpha] + \iint_{\mathcal{D}} f_x\ \underline{e}_x \cdot \underline{U}\ dx\ dy$$

where represents the domain IJKLAI (mechanism A) or A'JKLAA' (mechanismbB).

Noting that

$$f_x\ \underline{e}_x = f_x\ \underline{grad}\ x$$

and that

$$\text{div}\ (x\ .\ \underline{U}) = x\ \text{div}\ \underline{U} + \underline{U}\ .\ \underline{grad}\ x = \underline{U}\ .\ \underline{grad}\ x$$

because div U = 0 for the kinematic mechanism, it comes from the divergence theorem in the case of mechanism A:

$$\iint_{\mathcal{D}} f_x\ \underline{e}_x \cdot \underline{U}\ dx\ dy = \oint_{IJKLAI} x\ f_x\ \underline{U} \cdot \underline{n}\ ds = \int_{IL} x\ f_x\ U_y\ dx$$

since, aside from the segment IL, the velocity field U verifies U.n=0. (Similar expressions are evidently obtained for mechanism B).

The incorporation of the work of the inertia forces in the work of the external forces therefore amounts to the evaluation of the simple integral IL in eq 22.

Assuming that the vertical force is approximately constant during the seismic excitation, the upperbound for the horizontal force, i.e. the inertia force developed in the super-structure and applied at the elevation H of the center of gravity is given by:

$$\frac{T}{CB} \leq \phi \left(r_1, r_2, r_3, \frac{N}{CB}, \frac{H}{B}, \frac{f_x B}{C}\right)$$

The functions ϕ have the following expressions for the soil with resistance to traction:

- mechanism A

$$\phi = \frac{\lambda^2\left[\left(\frac{\pi}{2} - \mu + \frac{1}{2}\,\mathrm{tg}\,\mu\right) + \left(\frac{\pi}{2} - \alpha\right)\frac{1}{\cos^2\alpha}\right] + \lambda\,(1-\lambda)\,\mathrm{tg}\alpha - \left(\lambda - \frac{1}{2}\right)\frac{N}{CB}}{\frac{H}{B} + \lambda\,\mathrm{tg}\,\alpha}$$

$$- \frac{f_x B}{C} \; \frac{\frac{\lambda^3}{6}\left[\frac{(1 + 2\sin\alpha)(1 - \sin\alpha)^2}{\cos\mu\,\cos^3\alpha} + 1\right]}{\frac{H}{B} + \lambda\,\mathrm{tg}\,\alpha}$$

- mechanism B

$$\phi = \frac{\frac{\sin^2\beta}{\sin^2(\beta-\alpha)}\left[\left(\pi - \mu + \frac{1}{2}\,\mathrm{tg}\,\mu\right)\left(1 - \frac{\sin^2\alpha}{\sin^2\beta}\right) + \beta\,\frac{\sin^2\alpha}{\sin^2\beta} - \alpha\right]}{\frac{H}{B} + \frac{\mathrm{tg}\,\alpha\,\mathrm{tg}\,\beta}{\mathrm{tg}\,\beta - \mathrm{tg}\,\alpha}}$$

$$- \frac{f_x B}{C} \cdot \frac{\dfrac{1 + \frac{2\,\mathrm{tg}\,\alpha}{\mathrm{tg}\,\beta}}{1 - \frac{\mathrm{tg}\,\alpha}{\mathrm{tg}\,\beta}} + \dfrac{\left(1 - \frac{\sin\alpha}{\sin\beta}\right)^2\left(1 + \frac{2\sin\alpha}{\sin\beta}\right)}{\left(1 - \frac{\mathrm{tg}\,\alpha}{\mathrm{tg}\,\beta}\right)^3\cos^3\alpha\,\cos\mu}}{\frac{H}{B} + \frac{\mathrm{tg}\,\alpha\,\mathrm{tg}\,\beta}{\mathrm{tg}\,\beta - \mathrm{tg}\,\alpha}}$$

Equations 24 and 25 have to be minimized under the following constraints:

$$0 \leq \alpha \leq \frac{\pi}{2} \ , \ 0 \leq \mu < \frac{\pi}{2} \ , \ 0 \leq \lambda \leq 1 \text{ or } \alpha \leq \beta \leq \frac{\pi}{2}$$

For the soil without resistance to traction, the expressions for P(U) are given in [4] and the work of the external forces, including the inertia forces in the soil, are identical to that derived above. The analytical expressions for ϕ can be straightforwardly obtained.

Results of the minimizations of ϕ are presented in figs.8 to 13 which describe for a given normal force, the variation of the upperbound of the horizontal force as a function of its elevation H and of the seismic horizontal force f_x in the soil.

Comments on the results

The figures 8 to 13 show that there exists a maximum value for the seismic force beyond which no equilibrium can be reached. This limiting force is obtained upon minimizing eq. 9 with N=T=0 and is equal to:

$$\frac{f_x B}{C} \leq 2$$

This limit results from the fact that the seismic force has been taken constant with depth. It corresponds to a maximum value of f_x; however, as soon as the normal force becomes high, the combination of the permanent load (N) and of the seismic force (f_x) precludes a state of equilibrium, even under small f_x values. This has a physical sense since a high normal force means that the foundation soil system is close to a limit equilibrium; any further perturbation brings it on the surface of the domain K of the potentially stable loads. It must be noted that for practical problems the limit given by eq.26 is not constraining.

Figures 8 to 13 clearly show that the influence of the seismic force is almost negligible for small values of the normal force, as those encountered in practice for well-designed foundations (safety factor greater or equal to 3). The seismic force has a paramount importance for high values of the normal force and decreases dramatically the bearing capacity.

These results may be an explanation to the few observed bearing capacity failures during earthquakes. Referring to the state of practice in the evaluation of the bearing capacity, which does not include the effect of the soil inertia forces, the error on the ultimate load is negligible for well-designed foundations and very important for others. It is therefore most likely that these failures affected the foundations presenting a low safety factor with respect to dead loads.

DYNAMIC ASPECTS

The soil foundation system is assumed to behave as an elastic perfectly plastic system with respect to the load parameters N, T, M, f_x. The boundary of the domain K, previously determined, is adopted as the boundary for the apparition of plastic deformations. The plastic behavior of the system is defined by the kinematic mechanisms associated with the extreme loads and therefore obeys the normality rule with respect to the kinematic variables associated to the load parameters.

Noting K(U) the kinetic energy of the system, P_e (U) and P_i (U), the work of the external and internal forces

$$P_e(\underline{U}) + P_i(\underline{U}) = \frac{d}{dt} K(\underline{U})$$

with

$$K(\underline{U}) = \frac{1}{2} \iint_{\mathcal{D}} \rho \, U^2 \, dx \, dy$$

which can be shown to take the form [4]

$$K = \frac{1}{2} \rho \, \omega^2 \, B^4 \, k$$

where k depends only on the parameters r defining the kinematic mechanism.

The work of the external forces during the plastic movement of the foundation is, in view of eq.9, given by eq. 19 where T is replaced by T^+ the extreme load.

From eqs. 19, 27 and 29, the angular velocity of the foundation around point Ω (fig.5) is computed as:

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

with $\lambda = (\ell + b)/B$ and $T(t_0) = T^+$.

Integrating eq.30 between t0 and t1 (such as $\omega(t_1) = 0$), yields the maximum permanent rotation of the foundation. The maximum displacement of the foundation is equal to the product of two terms:

- one related to the geometry of the kinematic mechanism,

- one related to the time history of the applied load $T(\tau)$, i.e. the inertia forces developed in the super-structure.

Under the assumptions spelled at the beginning of the paragraph, the method permits a rigorous definition of "failure" in terms of unacceptable permanent displacements. It is asically similar to the one developed by Newmark for the seismic stability of dams [2].

A program working on a microcomputer has been developed in the course of the study to perform all the calculations presented throughout paragraphs 3 to 4. A very efficient algorithm for the minimization [1] yields the results in a few seconds on an AT286.

ACKNOWLEDGMENTS

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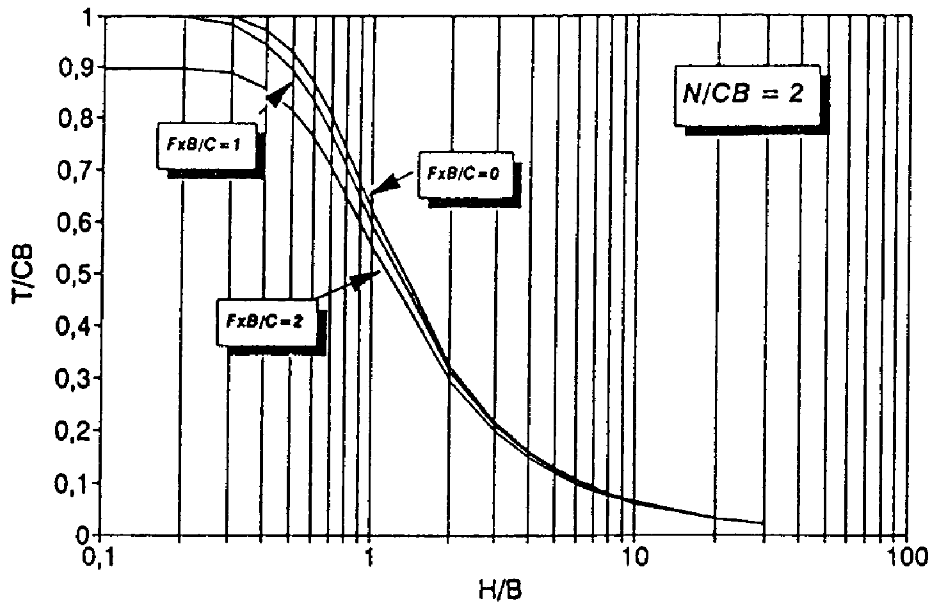

FIGURE 8. Seismic bearing capacity - Material with resistance to traction

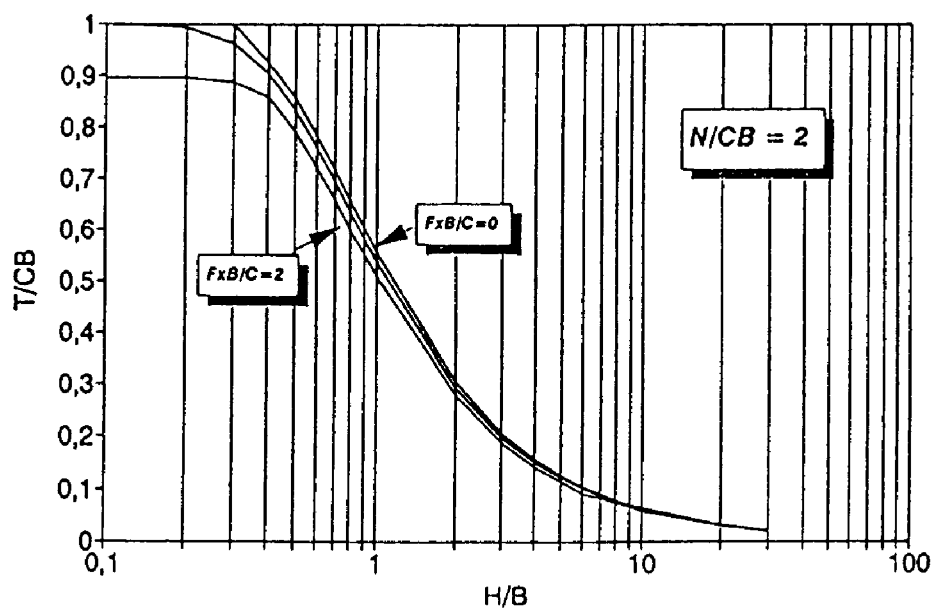

FIGURE 9. Seismic bearing capacity - Material without resistance to traction

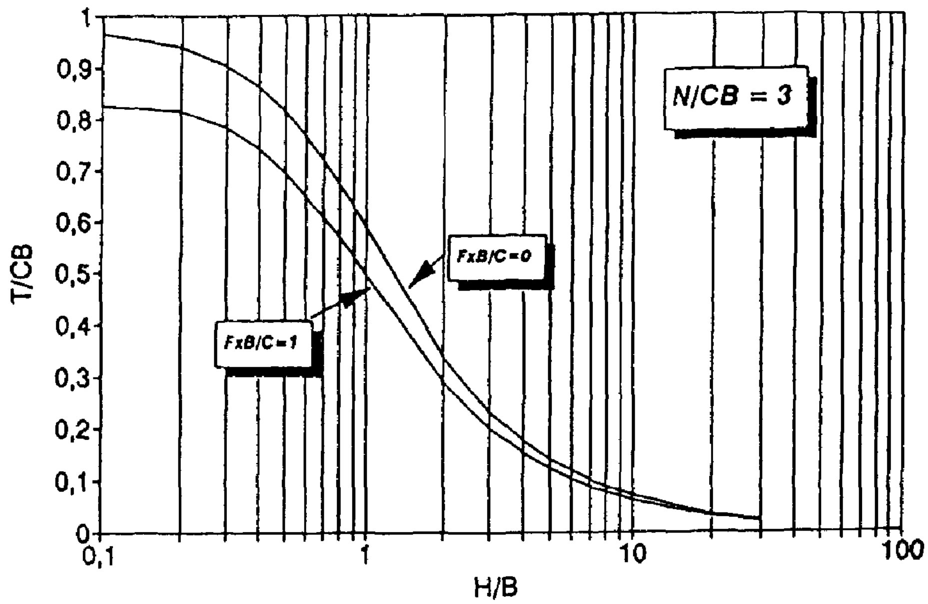

FIGURE 10. Seismic bearing capacity - Material with resistance to traction

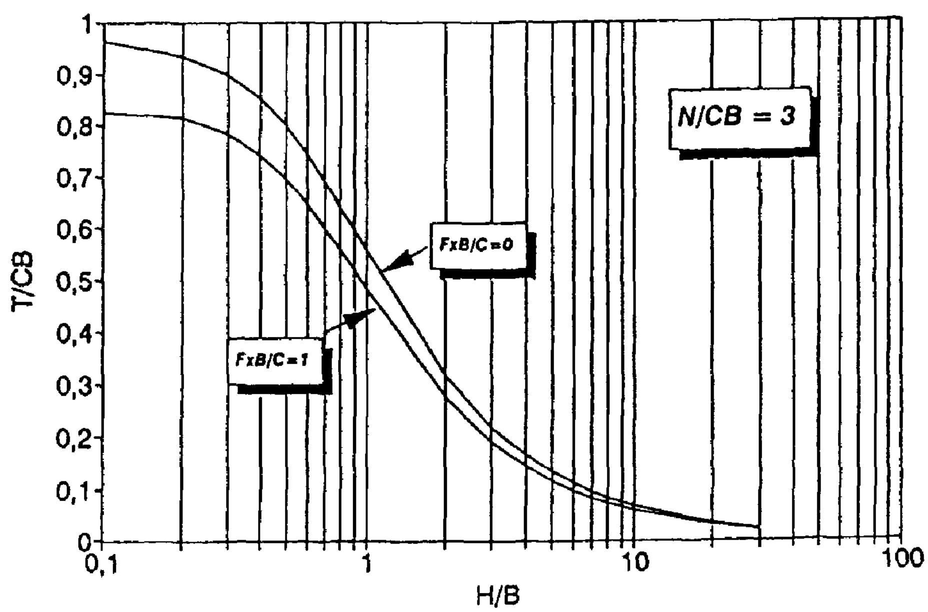

FIGURE 11. Seismic bearing capacity - Material without resistance to traction

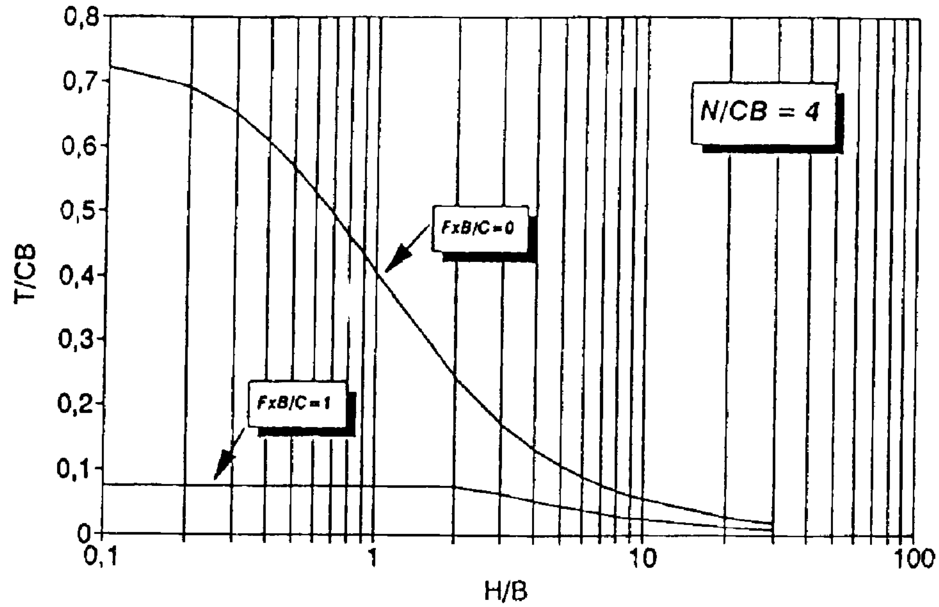

FIGURE 12. Seismic bearing capacity - Material with resistance to traction

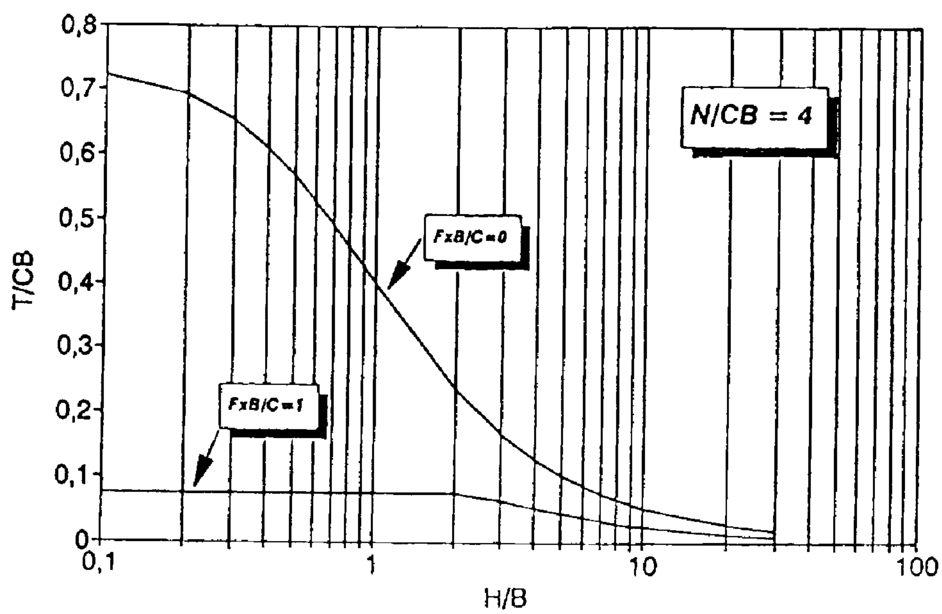

FIGURE 13. Seismic bearing capacity - Material without resistance to traction