Soil structure interaction and stability analysis of gravity platforms with discontinuous/irregular foundations

L'interaction sol-structure, la stabilité et l'analyse des plateformes-poids avec des fondations discontinues/irrégulières

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SYNOPSIS: A discrete element method is presented which can be used for soil-structure interaction and stability analysis of gravity platforms with discontinuous foundations (e.g. tripod platforms,circular protective barriers,platforms on "skirt piles",jackets in unpiled condition,etc).The basic assumptions and the procedure incorporated into a computer program,**DEAP**,are presented.The use of the program is illustrated by several examples.

INTRODUCTION.

The difficulties encountered in the design of platformes with discontinuous/irregular foundations arise from the following particularities:

1) The foundation geometry requires a three-dimensional analysis in order to determine the contact stresses,and displacements.Ideally this stress distribution should be determined by 3-D non-linear finite element analysis using non-linear,elasto-plastic constitutive laws for soil and iterative solutions.

2) Additional considerations,apart from equilibrium and failure criterion,are required to determine the load distribution between the different parts of the foundation.

3) The analysis should provide an unique material coefficient,valid for all parts of the foundation.

The proposed method is aiming to provide a simplified solution which can take into account the above mentioned particularities.

1.PRINCIPLES OF UNDRAINED,EFFECTIVE STRESS ANALYSIS.

The method is developed for analysis of contact stresses and stability of gravity platforms.

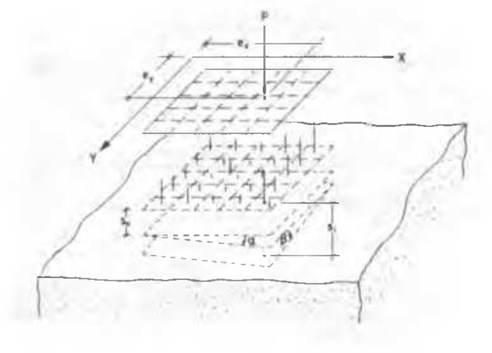
The on bottom weight of the platform is first applied on the foundation.Depending on the type of soil and drainage conditions, the resulting normal stresses in the soil may either be effective stresses or partly effective stresses partly excess pore pressures.The environmental loads are then applied in undrained conditions.

1.1.Elastic solution.

The platform base is divided into a number of discrete,rigid elements, supported by an elastic halfspace (**Fig.1**).

An initial elastic solution is first obtained by assuming that the soil is an elastic isotropic half-space, and that all discrete elements are rigidly connected to the centre of the structure.

A computer program (**SPRINT**) based on these assumptions was already developed,(Clausen,



Soil settlements : $s_i = \Sigma\, m_{ij} \cdot F_j$ $\quad \{S\} = [M] \cdot \{F\}$

Structure displacements : $d_i = s_0 + \alpha \cdot x_i + \beta \cdot y_i$ $\quad \{D\} = -[C] \cdot \{S_0\}$

Soil-structure contact: $s_i = d_i$ $\quad [M] \cdot \{F\} + [C] \cdot \{S_0\} = \{O\}$

Equilibrium :

$$\Sigma F_j = P, \quad \Sigma F_j \cdot x_j = P \cdot e_x, \quad \Sigma F_j \cdot y_j = P \cdot e_y \qquad [C]^T \cdot \{F\} + [O] \cdot \{S_0\} = \{P\}$$

Fig.1.Elastic solution.

1982) , and its theoretical bases are not repeated here.

The contact forces and the element displacements are found as a solution of elastic analysis.

1.2.Redistribution of contact stresses due to local yielding.

The following conditions may violate the assumptions on which the elastic solution is based:

1) The normal contact stress,σ_v,from elastic solution may exceed the normal stress, p_l, which will cause a local failure in the soil beneath the structural element.

2) The resultant shear stress,τ,from elastic solution can exceed the limit shear stress, τ_l,

(undrained shear strength) at the soil-structure interface.

3) The normal contact stress on a horizontal element, σ_v, from the elastic solution may be lower than a limit value, σ_t, which will produce failure in the soil element adjacent to structural element.

The procedure of stress redistribution consists of the following steps(**Fig.2**):

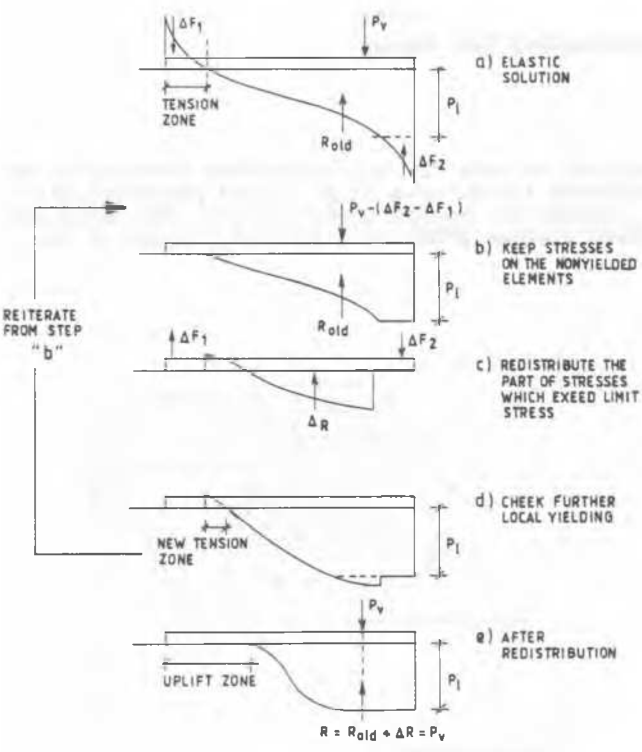


Fig.2.Redistribution procedure.

1.▸ Determine the elements for which normal or/and shear stress exceeds the corresponding limit stresses.

2.▸ Set the contact stresses for these elements equal to the limit stresses.

3.▸ Calculate an additional load vector by summing up the "excess" stresses.

4.▸ Perform an elastic analysis were only the "non-plastified" contacts are active to carry the additional loads and add the obtained increments of stresses to the existing ones.

5.▸ Repeat the steps 1.▸ to 4.▸ until additional forces to be redistributed become negligible.

1.3. Total stresses and pore pressures at the soil-structure interface.

After the contact stresses for each element have been redistributed as described above, each basic element (consisting of several subelements) is considered as a footing.

For each basic element (footing) the **contact area** is determined as the sum of areas of all elements which are not in the uplift zone and which belong to this basic element,and the **effective area** is calculated to account for local eccentricity.The **shear area** is also defined as the area which can carry shear stresses.

The total normal stress for each basic element is then determined as the average of stresses acting on contact area.The average shear stress is also determined over shear area.

The determination of pore pressures at the structure base is illustrated in **Fig.4**.

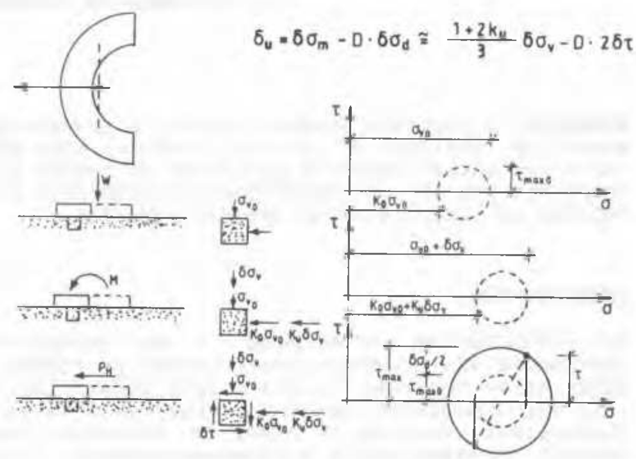


Fig.3.Determination of pore pressures at soil-structure interface.

1.4.Stability analysis.

The mode of failure for each discrete element of the structure is essentially the same as for a footing having the same width as the element and a given direction of failure,depending on relative position of the element towards neighbouring ones (for example the tangential direction to the circle for a ring foundation (**Fig.4**).

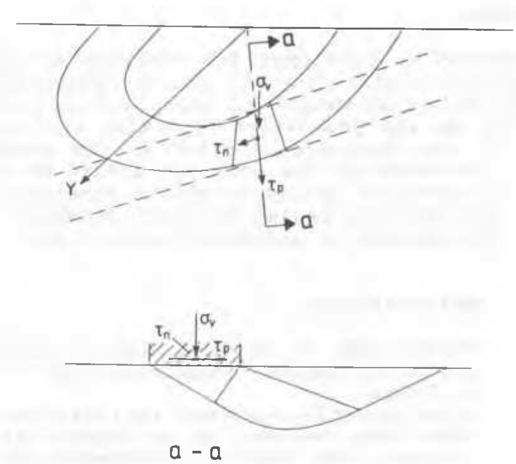


Fig.4.Failure mode of a ring foundation.

The following expressions relating the shear stress, the total normal stress and the mobilized friction, tanρ, can be written (Janbu,1976):

$$\tau_p = r_p \cdot \tan\rho \cdot (\sigma_v + a - \delta u_b) \qquad (1)$$

$$q_{ult} = N_w \cdot |N_q \cdot p' + (N_q - 1) \cdot a + 0.5 \cdot N_\gamma \cdot \gamma' \cdot B - N_u \cdot \delta u_b| \qquad (2)$$

where (Grande,1976):

$$N_w = (1 + f^2_w)/(1 + f^2_w . \tan^2(45 + \rho/2)) \qquad (3)$$

$$f_w = (1 - \sqrt{(1 - r^2_n)})/r_n \qquad (4)$$

q_{ult}= average total bearing capacity of the element
δu_b = average excess pore pressure at bottom slab/soil contact
N_q, N_γ and N_u=bearing capacity factors (Stordal,1980)
r_p, r_n= the roughness factor in failure direction and normal to failure direction, respectively.

The procedure used in the program to assess the stability of the foundation is illustrated by the flow chart in **Fig.5.**

The material coefficient will thus be:

$$\gamma_m = \tan\varphi'_u/\tan\rho \qquad (5)$$

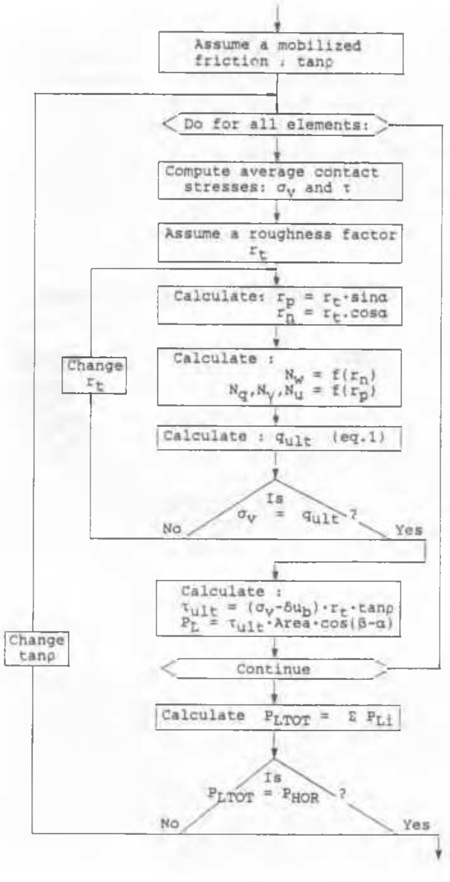


Fig.5.Flow chart of stability analysis.

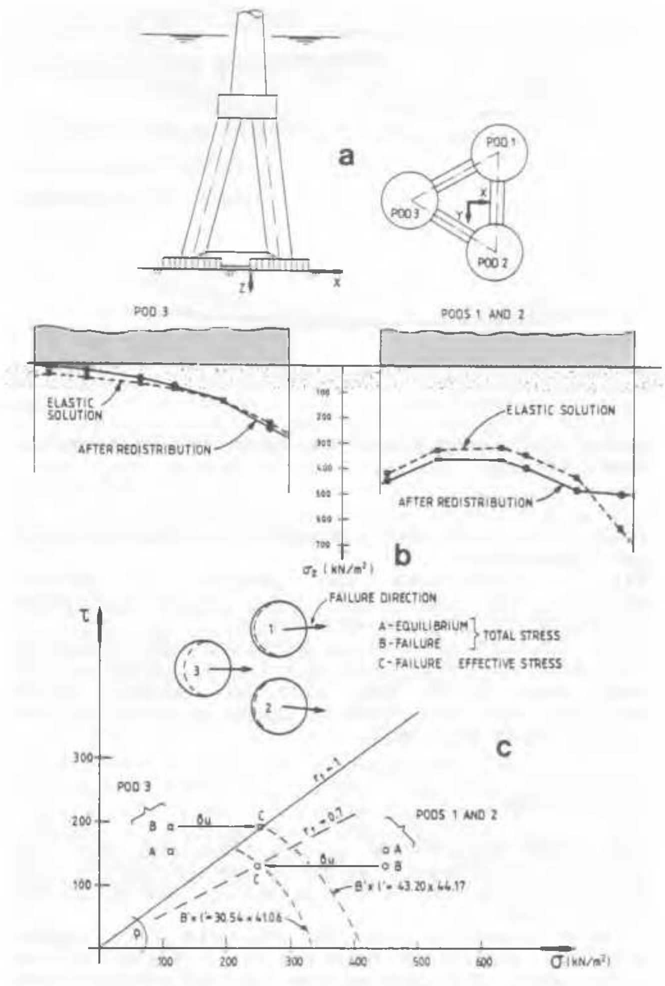


Fig.6.Tripod platform subjected to longitudinal wave force.

2.THE USE OF THE DISCRETE ELEMENT METHOD TO ANALYSE THE CONTACT STRESSES AND STABILITY OF DISCONTINUOUS PLATFORM BASES.

The procedure described in the previous sections is incorporated into a computer program (**DEAP**-Discrete Element Analysis Program),which is an extension of the elastic solution program **SPRINT**.

The following examples were chosen to illustrate the applicability of the program: The tripod platform is shown schematically in **Fig.6.a.** The contact stresses on each of the pods after redistribution due to local yielding are shown in **Fig.6.b.** As can be seen,the redistribution resulted in an increase of the eccentricity for the back pod.

The results of the stability analysis are shown in **Fig.6.c.** The equilibrium shear stresses acting on front pods are larger than those corresponding to failure situation because of relatively high normal stress.These excess shear stress are transferred, at failure, to the back pod which carries lower normal stresses. After the transfer, the back pod will fail in pure sliding (total roughness ,r_t = 1.) while the

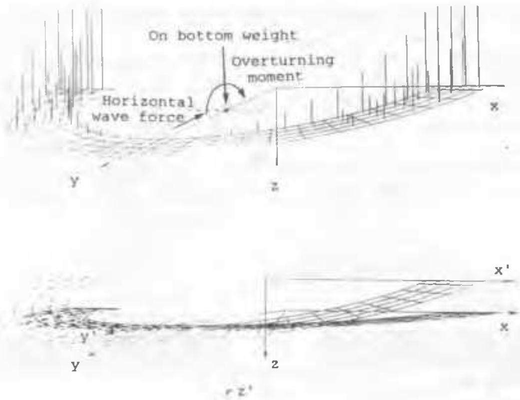


Fig.7.Half ring structure subjected to forward wave force.

front pods will fail somewhere between sliding and compression.
Fig.7. illustrates the results of contact stresses analysis for half ring structure subjected to forward wave force.

The results of contact stresses and stability analysis for the same structure subjected to skew wave force and torsional moment about vertical axis are shown as graphic output of the program **DEAP** on **Fig.8.**

CONCLUSIONS.

The presented program is an efficient tool for analysis of contact stresses and stability for rigid structures with irregular/discontinuous foundations.

In contrast to the FEM analysis this method requires only minor resurces.The program is thus very useful for preengineering and design work where parametric studies are important and necessary for determination of the most effective foundation size and geometry.

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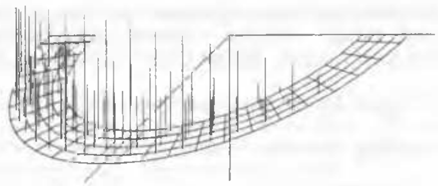
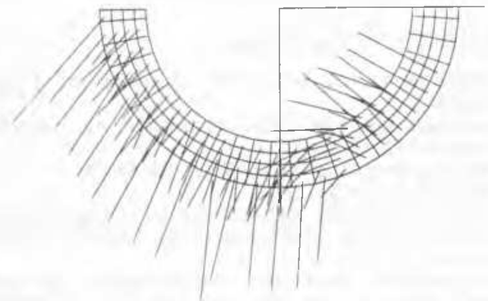
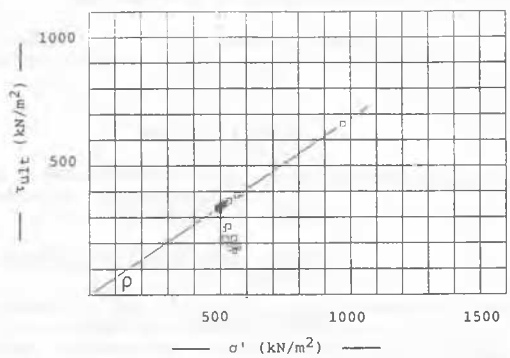


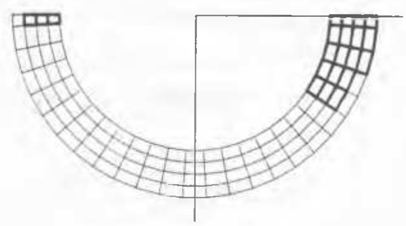
a. Normal contact stresses.



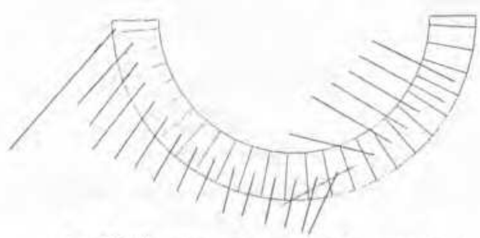
b. Shear contact stresses.



c. Ultimate contact stresses.



d. Uplift area .



e. Ultimate average shear stresses .

Fig.8.Half ring structure subjected to skew wave force and torsional moment.