Finite element analysis of plates and foundations

L'analyse par éléments finis des plaques et des fondations

A.P.RUFFIER, CEPEL, Electrical Energy Research Center, Rio de Janeiro, Brazil
C.F.MAHLER, COPPE/School of Engineering and Graduate School of Engineering, Federal University of Rio de Janeiro, Brazil

SYNOPSIS: This work deals with a finite element simulation of the uplift of plates and foundations. The program uses joint elements for the soil-structure interface and two-dimensional and axisymmetric elements with four nodes for the soil and foundation. Several tests performed in tropical residual soils near Rio de Janeiro and near Goiânia are simulated. Both soil non-linearity and plastification are taken into account. The results obtained suggest that the proposed procedure can be used for the prediction of pull-out resistance of similar foundations.

1 INTRODUCTION

In Brasil, in recent years, great enphasis has been given to the study of foundations subject to pull-out forces.

Growing demand of electrical energy in large urban centres, together with hydroelectric generation result in large blocks of energy being transmitted through long distances.

Therefore, there is a growing tendendy towards larger self supporting and guyed transmission towers, with tighter requirements on safety and cost effectiveness.

The usual procedure for estimating pull-out resistance of foundations is based on the behaviour of sedimentary soils. In Brazil, however, the great majority of transmission towers rest on residual soils. Several tests performed on residual soils near Rio de Janeiro (Barata et alii - 1979) and near Goiânia (Oliveira - 1986) showed poor agreement between the experimental uplift failure loads and those predicated by conventional methods.

The method presented in this paper predicts not only the uplift failure load but takes also into account the whole soil-structure system, through a finite element formulation. The finite element program deals with the non-linear and plastic behaviour of the soil in pull-out load conditions through an incremental iterative formulation. Axisymmetric elements are used for the soil and for the foundations and joint elements are used for the soil-structure interface.

In this work it is presented the conclusion of a research developed since 1985 which partial results have been shown in Ruffier and Mahler (1986 and 1988).

2 METHODS OF ANALYSIS

Consider a solid body surrounded by a non linear soil mass. The model idealized to represent this system consists of the soil, the structure and the interface between soil and structure.

Four - constant - strain - triangles elements (4CST) are used to simulate soil and structure. This element presents a good performance in axisymmetric problems.

Joint elements are used to accomodate the relative displacement between soil and strucutre which occur when an axial load increment is applied. The joint element, originally developed for analysis at discontinous rocks (Goodman - 1976), is undeformable under compression and permits two kinds of movements, sliding and gapping, under tangential and tensile stress respectively.

The non-linear stress dependent stress-strain characteristics of the soil was considered. A hyperbolic stress-strain relationship (Duncan and Chang - 1970) was adopted to obtain the tangent value of Young's modulus. The tangent value of Poisson's ratio was obtained through an exponencial formulation (Lade - 1972), and its value presented very small variations.

For the interface, the value of the normal stiffness of the joint, k_n was maintained constant, and the value of the tangential stiffness of the joint, k_s changed with stress and strain in the following manner:

$$k_s = \frac{a + \sigma n \ . \ \tan\delta}{\varepsilon_\xi}$$

where:

a - adhesion or tangential stress for n = 0
δ - angle of friction,
σn - normal stress,
ε_ξ - tangential strain.

For both elements two failure plastification criteria were adopted:

a) Mohr-Coulomb,
b) Traction limits.

When the 4CST element fails the program adopts an extremely low value for Young's modulus.

When the joint element fails, k_s and k_n are considered equal to zero.

The non-linear material effect is incorporated by adopting the incremental iterative Newton-Raphson procedure. The convergence criterion is assured using the Euclidean Norm, in each direction, for the elements and the global system. In the examples presented here an average of five iterations was required for a tolerance criterion of 10^{-4}.

3 PARAMETER DETERMINATION

The plate tests were conducted on the campus of the Federal University of Goias, near Goiânia (Oliveira - 1986). The site consists of a tropical residual laterictical soil with the water table 6.7 m deep.

These tests consisted of a series of pull-out essays in eleven circular plates whose dimensions are shown in Table 1. The least distance between the plates is about 2.0 m. The backfill material is obtained by compactation of original soil.

Table 1. Plate dimensions

Plate	Diameter (cm)	Depth (cm)
P-1	20	30
P-2	20	45
P-3	20	60
P-4	30	30
P-5	30	45
P-6	30	60
P-7	40	30
P-8	40	45
P-9	40	60
P-10	50	30
P-11	50	45

The other test were conducted on the plateau of a hill near Rio de Janeiro. The site consists of a layer of mature tropical residual soil originated from a gneissic rock with approximately 2.5 m thickness, superimposed on a less weathered very thick layer. The water table was found to be bellow 15 m depth.

These tests consisted of a series of pull-out essays in four footings and five pier foundations, two with enlarged base (T-1 and T-2) and there without base (T-3, T-4 and T-5). The least distance between the foundations is about 8.5 m. Their dimensions and the depth of the first layer in each case are shown in Figure 1.

The adopted parameters were obtained from laboratory test (Barata et alii - 1979, Oliveira - 1986) and from analysis of plate loading tests (Werneck et alii - 1979) and pull-out load test data. Their determination is well described in Ruffier (1985). The parameters used here are presented in Tables 2 and 3, for interfaces and soils, respectively. The method of determination of the hyperbolic parameters can be seen in Duncan and Chang (1970). The large value adopted for k_n has the obvious objective of avoiding an interpenetration of two neighbouring elements. The plates and the concrete foundations where considered undeformable (with very high stiffness). This agree with the field observations.

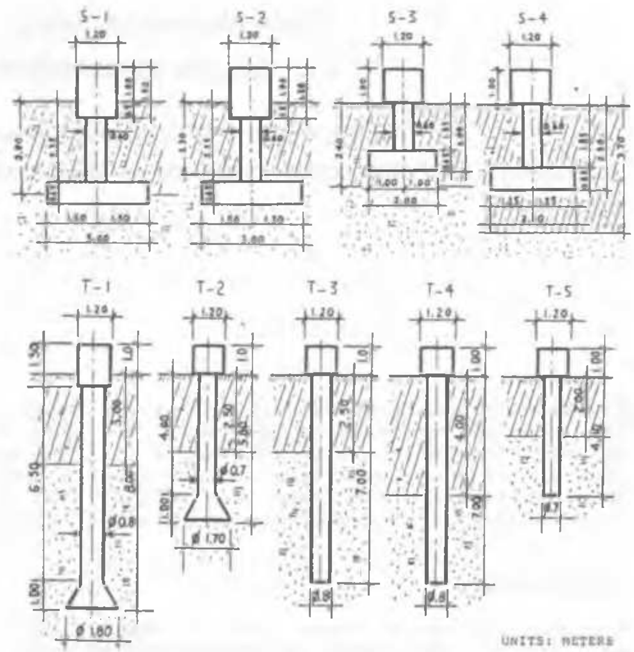

Figure 1. Footings and Pier Foundations Geometry and Soil Layers

4 NUMERICAL EXAMPLES

The cases of plates, footings and pier foundations subjected to uplift loading were simulated by the above described method

One difficulty in analysing the infinite boundary surface problem by the Finite Element Method (F.E.M.) is an adequate choice of the limit boundaries. Another question is the choice of the degree of mesh complexity. The four meshes presented in Figure 2 were developed taking these facts into account. They are examples of meshes used in the finite element analysis.

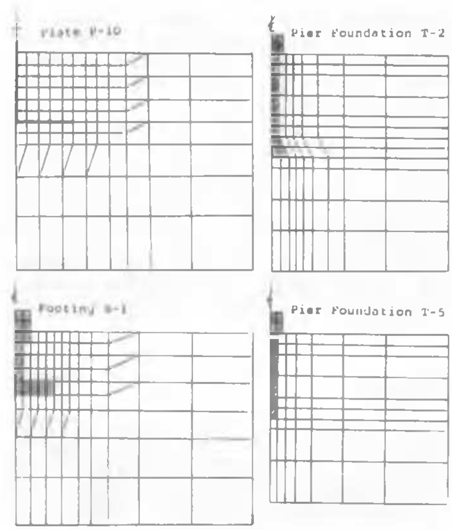

Figure 2. Examples of Finite Element Meshes

Table 2. Interfaces parameters

	Soil of Goiânia		Soil of Rio de Janeiro	
	Natural	Compacted	Upper Layer	Lower Layer
k_n (MPa/m) (normal stiffness)	1 000 000	1 000 000	1 000 000	1 000 000
k_s (MPa/m) (tangential stiffness	500	500	500	500
a (MPa) (adhesion)	0.045	0.080	0.044	0.035
δ (degree) (angle of friction)	14	15	19	20
σ'adm (MPa) (admissible tensile stress)	0.001	0.001	0.001	0.001

Table 3. Soils parameters

		Soil of Goiânia		Soil of Rio de Janeiro	
		Natural	Compacted	Upper Layer	Lower Layer
γ (kN/m³) (unit weight)		13.8	15.9	16.5	18.0
c (MPa) (cohesion)		0.030	0.053	0.029	0.023
ϕ (degree) (angle of internal friction)		20	22	27	29
Hyperbolic parameters	K	550	650	600	350
	n	0.52	0.51	0.52	0.50
	R_f	0.75	0.80	0.75	0.85
	K_{ur}	700	900	900	590
ν (Poisson's ratio)		0.4	0.4	0.4	0.4
E_R (MPa) (residual tangent Young's modulus)		0.1	0.1	0.1	0.1
σ adm (MPa) (admissible tensile stress)		0.001	0.001	0.001	0.001

It was verified, for plates and footings, that the plastification of the soil initiates near the extreme of the base and propagates upwards toward the surface.

For piers it was verified that the failure process initiates at the base of the pier foundation and propagates, element by element, along the shaft towards the surface. For the pier foundation with enlarged base plastification of the soil was also observed. It initiates at the extreme lateral joint of the base and propagates upwards through a certain portion of the soil mass.

For both kinds of foundations the observed failure surfaces agree with field observations. Examples of soil-foundation interaction and soil plastification are presented in Figure 3. Examples of displacements are presented in Figure 4. These figures where obtained using a post-processing plotter system, linked to the program.

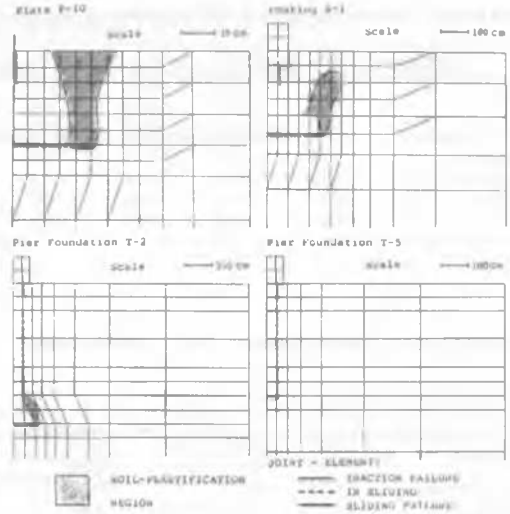

Figure 3. Examples of Soils-Foundation Interaction and Soil Plastification

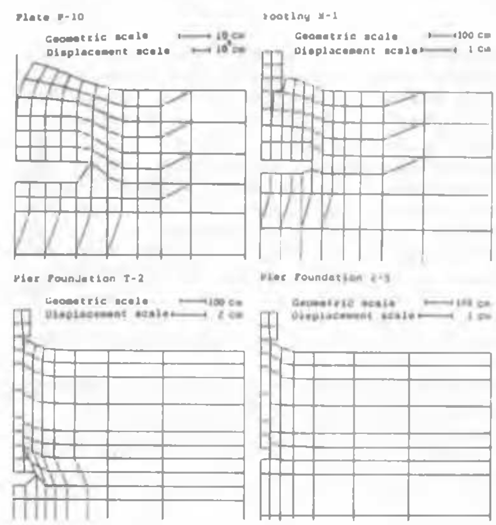

Figure 4. Examples of Foundation Displacements

Pull-out resistances obtained from field tests, traditional methods for determination of the failure load and F.E.M. simulation are presented in Tables 4, 5 and 6. These methods are well know and the calculations are explained in Oliveira (1976) and Ruffier (1985).

5 CONCLUSIONS

The Finite Element Method has been applied to analyse the behaviour of pier foundations subjected to uplift forces. Circumferential plates, footings and two kinds of pier foundations,with and without enlarged base, have been considered in the investigation of displacements, failure loads and shape of failure surfaces.

Table 4. Pull-out resistance of different plates

Plate	P-1	P-2	P-3	P-4	P-5	P-6	P-7	P-8	P-9	P-10	P-11
Field Test	581	1222	1948	1103	1908	2368	767	1181	2362	1217	1963
Balla	977	1652	2548	1197	2276	3134	1565	2718	4211	2154	3167
Meyerhoff & Adams	605	923	1253	915	1398	1874	1242	1910	2564	1565	2420
Duke University	702	1234	1781	962	1620	2418	1260	2075	2966	1534	2518
Grenoble University	573	920	1322	833	1328	1837	1122	1768	2444	1387	2202
Rowe & Davis	475	731	962	727	1103	1462	882	1509	2067	901	1610
F.E.M.	504	846	1243	756	1195	1708	976	1609	2273	1276	2114

Units: N

Table 5. Pull-out resistance of different footings

Footing	S-1	S-2	S-3	S-4
Field Test	1300	1240	585	1160
Balla	1599	1493	559	954
Meyerhoff & Adams	1636	1545	591	1015
Duke University	1423	1322	564	887
Grenoble University	1435	1346	558	881
Rowe & Davis	1376	1275	588	894
F.E.M.	1175	1125	550	875

Units: N x 10^3

Table 6. Pull-out resistance of different pier foundations

Pier	T-2	T-2	T-3	T-4	T-5
Field Test	2175	1060	1510	975	455
Meyerhoff & Adams	3239	1756	669	879	380
Grenoble University	2095	1025	857	858	323
Rowe & Davis	1712	988	617	655	268
F.E.M	1850	975	1150	1100	500

Units: N x 10^3

It was verified, for plates and footings, that the traditional methods agree reasonably with the field tests resistances, in especial the Rowe e Davis' Method and the Methods of Duke University and Grenoble University. For the two kinds of pier foundations, only the method of Grenoble University have good results. For both kinds of foundations the F.E.M. presented reasonably agreement.

The predicted shape of the failure surface clearly shows, for plates and footings, that the failure process starts at the extreme of the base. For pier foundations the process starts around the pier base and moves along the shaft towards the surface.

Displacements predicted by F.E.M. also showed a good agreement with experimental results.

Finally, the finite element program used in this report proved to be a strong computational tool for research and for engineering design.

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