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APPLICATION OF PRESSUREMETER TESTING TO ASSESS LATERAL PILE RESPONSE IN CLAYS

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ABSTRACT: Typical lateral load magnitudes encountered on land and offshore are compared. The effect of pile deflection and stiffness response on jacket performance is discussed. The limitations of current design methods are noted. Alternative design procedures are given, using site specific pressuremeter load-deformation data and a quasi three-dimensional non-linear finite element method. Comparisons are made between the observed and computed behaviour of lateral load tests on small diameter piles in clay for which pressuremeter data are available. The problem of extrapolation to larger diameter single piles, and the behaviour of pile groups, is discussed.

KEYWORDS: soils, pressuremeter, piles, lateral load, finite elements

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INTRODUCTION

For single piles, the lateral pile response to static loads is normally determined using 'p-y' curves. These are non-linear force displacement relationships for the soil mass supporting a unit length of pile. These 'p-y' curves in design codes have been derived from back-analysis of laterally loaded piles and laboratory test data. The number of such case histories is small; consequently the number of soil types is also small - three in fact.

Problems therefore occur when different soil types are encountered. A secondary problem occurs in the extrapolation to the behaviour of piles which are considerably larger than used in the original lateral load tests. Such pile sizes are commonly encountered offshore.

Pile groups are generally required at each leg when the platform is situated in more than about 50 m water depth. The group response is normally obtained from the above p-y method plus elastic solutions to allow for interaction effects. Significant assumptions are generally made which lead to conservative deflection estimates.

An alternative method is discussed. This uses site specific data, derived from in-situ pressuremeter testing, plus a finite element method. Recommendations are given for the pressuremeter testing and analysis procedures.

The alternative method has been evaluated for 4 case studies in clay soils. These include the historic laterally loaded test pile sites at Lake Austin, Sabine River and Manor. The E, ν and Su data from pressuremeter tests of Smith [1] have been used.

CONVENTIONAL DESIGN PRACTICE

Beam-column Method

In this widely used model, the beam (i.e. pile) is supported by a set of springs which model the soil behaviour. The spring characteristics are given by non-linear 'p-y' curves. They represent the lateral soil resistance per unit length of the pile (p) against pile deflection (y).

If the 'p-y' curves can be accurately specified, then the results of deflected shape and bending moment profiles are acceptable.

Formulae and tables to construct the 'p-y' curves are given in various design codes. For offshore constructions, the most commonly used are the API code [2]. Therein, the curves are limited to three soil types: soft clay, stiff clay and (non carbonate) sand.

The 'p-y' criteria for soft clay were derived from lateral load tests performed on 12.75" (325 mm) dia. piles at Lake Austin, and at the mouth of the Sabine River [3]. The stiff clay criteria were obtained from lateral load tests on 6" (152 mm) and 24" (610 mm) diameter piles at Manor [4]. In addition a unified approach for both clay types has been proposed [5]. This was based on the above sites, plus additional testing made in San Francisco Bay. Similarly, the sand 'p-y' curves are based on tests at Mustang Island [6].

For clay, the above criteria are expressed in terms of the pile diameter, undrained shear strength, the axial strain half way to failure in a compression test, and empirical coefficients. For sands, the soil friction angle, empirical coefficients and the modulus of subgrade reaction, are required. Generally, the selection of the latter value is usually of concern for offshore structures.

The site specific data is thus obtained from laboratory tests plus empiricism.

Further problems arise when soils are encountered (such as soft rock, silts and carbonate sands) whose behaviour is different to those tested above.

Pressuremeter Method

More recently, methods have been proposed which develop the 'p-y' curves from the entire pressuremeter expansion curve. Briaud et al [7] show that the 'p-y' curves may be obtained from the addition of the frontal and side resistance components. Each of these is subject to a pile shape factor and a reduction factor to allow for the critical depth.

Robertson et al. [8] present a simpler method based on both full displacement and self boring pressuremeter curves. For the latter type, pressures are magnified by a factor 9/5, and the strains by half the pile diameter to obtain displacement. Again the critical depth is allowed for by reduction of the resultant pressure values.

Such pressuremeter based methods constitute an improvement, since the soils data is now obtained in-situ and is site specific. However some degree of empiricism remains in the analytical procedures.

OFFSHORE CONSIDERATIONS

Offshore Pile Loads

Offshore, Hult, the ultimate lateral capacity of the piles is never achieved due to their long length and fixed head condition. In order to provide a rough indication, Hult has been evaluated from the formula

$$\text{Hult} = N_c * S_u * D_o * L$$

where N_c = bearing capacity factor = 9
S_u = undrained strength = 200 kN/m^2 for N.Sea clays
= reported values for test sites
D_o = outer pile diameter
L = pile length, assumed = 10 D_o

In addition, Randolph [9] has shown that the relevant measure of pile/soil stiffness is the value of Ep/G*
where Ep = pile effective Young's modulus
$= (EI)p/(\pi D_o{}^4/64)$
$G^* = G(1 + 3\nu/4)$
G = soil shear modulus
ν = soil Poisson's ratio

Table 1 compares typical load and stiffness ratios for offshore piles and the original laterally loaded piles on land.

TABLE 1--Typical Pile Data.

Location	Clay Type	Pile Diameter D_0 (m)	Max. Lateral Load H (MN)	Load Ratio H/Hult (%)	Pile/Soil Stiffness Ep/G*
Offshore piled platforms					
Southern N. Sea	stiff	1.00-1.25	0.5-1.0	2.8-4.4	1100-1400
Northern N. Sea	stiff	1.50-2.10	1.0-3.0	5.6-7.9	1400-1700
Land 'p-y' test sites					
Lake Austin	soft	0.323	0.10	57.0	32700
Sabine River	soft	0.323	0.08	56.8	41400
Manor	stiff	0.641	0.60	10.8	1250

The above table illustrates the following points:

(1) The significant increase in scale from onshore to offshore foundations. Design of onshore piles can be done with a relatively large confidence in their performance, since it is based on pile load tests on similar pile sizes installed in comparable soil conditions. Virtually no such data are available for offshore piles.

(2) The offshore piles mobilise a smaller percentage of their ultimate capacity despite their larger loads.

(3) The pile/soil stiffness ratio offshore differs little from the land test made in stiff clay.

Structural Performance

For shallow water depths, the effect of reasonable variations in horizontal soil stiffness on the jacket design is not great. Under horizontal loading, the smallest eigen frequency of the jacket/soil system is lower than the environmental (i.e. wave) frequency.

However, as structures move into deeper water, the platform and wave frequencies may coincide. Thus, in this case, better estimates are required of the foundation stiffness components both for a single pile and the interaction effects for the pile group.

PRESSUREMETER METHOD

The Finite Element Method requires as input real soil properties. These are obtained using the pressuremeter. This may be either of the push in (self boring) [10, 11] or the full displacement [12] type. In the pressuremeter test, at least 2 unload/ reload cycles should be performed prior to reaching the maximum volumetric strain. This is in order to check the variation of shear modulus G with stress level. Current Fugro practice is to cycle at the third points of the maximum strain range; however this procedure is soil formation dependent.

Table 2 shows how the soil parameters are determined for the push-in type of pressuremeter.

TABLE 2--Soil Parameters

Parameter	Obtained from PMT	Reference
Shear modulus G	unload/reload loops	Henderson et al. [10]
Undrained Shear Strength S_u	total pressure vs log volumetric strain	Gibson + Anderson [13]
Friction angle Ø'	log effective pressure vs log volumetric strain	Hughes et al. [14]
At rest pressure coefficient K_o	lift off pressure	Lacasse et al. [22]

By performing a series of tests at various depths in one or more boreholes, profiles of the above parameters versus depth are obtained. These are then used in the Finite Element analyses.

FINITE ELEMENT METHOD

The Finite Element Method was used for this case study. It offers the following advantages over the traditional beam column method:

(1) real soil properties are used.

(2) a soil model identical with that used for back-analysis of the pressuremeter data [10, 13, 14] is used, namely a linear elastic-perfectly plastic (Mohr-Coulomb) model [23]. In this model, parameters E and ν define the elastic soil behaviour, and c', Ø' and θ' (effective stress-sands) or S_u (total stress-clays) define the plastic work components.

(3) no additional value judgements are required; for example the reduction in ultimate lateral pressure at shallow depth, or the base shear stiffness for short piles, is automatically included. The analysis of laterally loaded retaining walls (both flexible [24] and stiff [25]) has demonstrated the capability of the Finite Element Method to obtain classical ultimate Rankine earth pressure distributions for the equivalent plane strain problem.

(4) the method may be also used for pile groups.

In this application, the soil is considered as a continuum initially attached to the pile shaft. A Fourier harmonic analysis technique is used to reduce the three dimensional problem to a two dimensional one. Each node has three degrees of freedom (r, z, θ). In the r-z plane the normal shape functions are used. In the θ plane a Fourier series is used since the material and geometry remain constant. Elastic analyses were first reported by Wilson [15]. Elasto-plastic formulations have been subsequently incorporated [16, 17, 18].

To handle the non-linearity, a constant stiffness matrix formulation was used, together with the viscoplastic method for redistribution of the excess stresses in the plastic zones. A no tension criterion [19] was incorporated. Eight noded rectangular (isoparametric) elements with two point gauss integration were used. The stresses were checked at 45 degree intervals. Five harmonics were used. Further program details are given by Griffiths [20]. Typical run-times of 200 seconds were required on a CDC 7600 computer system to obtain the presented results.

SITES SELECTED

Four sites were selected where both pile and pressuremeter tests had been performed. The latter were all conducted by Smith [1]. The pressuremeter testing was done either in an augered or a drilled borehole using a prototype TEXAM unit. The values of E and S_u presented by Smith were reused in the finite element analyses. Table 3 shows that the time lapse between the two types of tests was up to 25 years.

TABLE 3--Site History

Sites	Date of Lateral Pile Test	Date of Pressuremeter Test	Soil Type
Lake Austin	1956	1981	soft clay
Sabine River	1960	1982	soft clay
Manor	1967	1982	stiff clay
Texas A + M	1977	1981	stiff clay

Common to all sites were the following features:
(1) clay soils were tested
(2) the piles were free headed
(3) the initial static loading curve was analysed
(4) the reload modulus (E_R) was used in preference to the initial modulus (E_i)
(5) a fixed Poisson's ratio $\nu = 0.3$ was used by Smith to derive the Young's modulus value E from the shear modulus G.

Lake Austin (Figure 1)

The soft clays and silts had been deposited this century behind the Lake Austin Dam, Texas. Due to exposure during drawdown, the upper soils are dessicated, jointed and fissured. During the pile test in 1956, vane testing gave shear strength values averaging 38 kN/m^2 with little variation with depth [3]. Unconfined triaxial tests were also done, giving shear strengths of about 24 kN/m^2 [1].

The soil profiles determined by Smith [1] from the pressuremeter testing in 1981 are shown on Fig. 1a. The shear strength values so derived (17 to 20 kN/m^2) are slightly lower than those previously reported.

Sabine River (Figure 2)

The site is located at the mouth of the Sabine River, which forms the Texas/Louisiana State boundary. The pile test was performed in 1960. This, plus the Austin test, formed the basis for the "soft clay" design method [3]. The soils are mainly soft slightly overconsolidated marine clays with a sand layer occuring from 5.0 to 6.1 m below ground level. Sand seams and partings are present throughout the soft clay. Vane shear strengths averaged 14.5 kN/m^2 in the significant upper zone. Unconfined tests showed a general strength increase with depth, ranging from 5 kN/m^2 near the mudline to 24 kN/m^2 at 9.1 m depth. The pile test was made in a 1.2 m deep pit flooded to 0.15 m depth.

Pressuremeter tests were made in 1982. The shear strength derived was again less than obtained previously by other tests. Smith [1] concluded that the soil profile was representative of that twenty years ago.

Manor (Figure 3)

The Manor site is located some 8 km North East of Austin, Texas. The surface soils are stiff overconsolidated clays of marine origin. They have a secondary slickensided and fissured structure. The shear strength increases rapidly with depth. The "stiff clay" design method [4] was based on results of testing 6.25" and 25" diameter pipe piles.

To simulate an offshore environment, a flooded pit 1.8 m deep was used. After excavation was completed, the pit was flooded with water some 5 months prior to pile installation. Another month passed before the piles were loaded. Borings made at various stages of the pit excavation indicated that the water content had increased and the shear strength had decreased in the upper zones.

The reported final shear strength profile from the test mudline gave an increase from 96 kN/m^2 at 0.3 m depth to 385 kN/m^2 at 3.6 m depth.

In 1982, pressuremeter testing was done. Due to the placement of about 1.6 m of fill and the limited depth to which testing could be done, only two tests could be made below the original pile mudline. Thus, at depth, the profiles used for the finite element analysis have been estimated on the originally reported shear strength profile shown on Fig. 3a, plus the ratio $E/S_u = 300$.

Texas A & M (Figure 4)

Lateral pile tests have been conducted on the campus of Texas A & M University in 1977, 1978 and 1979. All were on cast in-situ concrete piles. In this study, only the first was analysed. The clay is firm to stiff in the upper 1.5 m, and thereafter stiff to very stiff. Above the water table (4.5 - 5.5 m depth), the soil has been preconsolidated by dessication. The concrete pile failed structurally during the test [7].

In 1981 a pavement pressuremeter was used, giving the profiles shown on Fig. 4a. No fill had been placed and no water content alterations had occurred; thus, like the Sabine test site, a reasonable correlation was anticipated.

FINITE ELEMENT ANALYSES

Results

The profiles adopted for the analysis are shown on Figs. 1 to 4. The ratio E/s_u was around 300 (using ν = 0.30).

The computed and observed load-deflection results are compared on the same Figures. As is obvious from the results, the stiffness has been overestimated, though not grossly, in all cases. However, the initial stiffnesses are reasonable, particularly for the Sabine and Texas A+M cases.

The cases studied illustrate the need to determine soil conditions at the time of the actual tests. The Lake Austin and Manor Sites have probably been influenced by dessication and flooding respectively. In addition, the latter portion of the Texas A+M result may be less stiff due to plasticity of the reinforced concrete pile occurring prior to failure.

Softened Zone Influence

As part of a sensitivity study, a softened zone with $E/2$ and $s_u/2$ immediately adjacent to the pile shaft for the Sabine and Texas A + M cases was adopted. The match was improved slightly, but the initial stiffness decreased marginally (Figs. 2b and 4b). The results were for an annulus 50 mm thick. Little alteration was obtained for 100 and 150 mm thicknesses. It was concluded from this that the initial pile stiffness is primarily influenced by the stiffness of the soil mass in the far field as opposed to the soil zone near to the pile.

Elastic Modulus Influence

The probable cause of the over-stiff response lies in the fact that the non-linear load-displacement behaviour of the clay has been modelled in the analyses by two straight lines. A better approximation may have been by a hyperbola (Fig. 5). Time constraints precluded the insertion of this feature. The following procedure was therefore adopted. The Sabine River case was re-analysed with $E/s_u = 100$ to give a lower bound deflection response. The results (Fig. 2b) are encouragingly close to the measured pile response at the high load levels. It is concluded that an E/s_u ratio of 300 gives reasonably good estimates of initial stiffness, and that an E/s_u ratio of 100 should be used when soil strains in excess of, say, $\varepsilon_{max}/2$ are encountered.

LARGE DIAMETER PILES

Offshore, extremely large piles, up to 7.0 m diameter, are occasionally required to serve as monopile constructions or as seabed anchor points. It is generally believed that, for such piles, the displacements computed in accordance with the API code are an overestimate. However, there is very little published quantitative data on full scale large diameter pile tests to positively confirm the "diameter effect".

However, the present study indicates the probable reason for the diameter effect: The pile head stiffness is influenced greatly by the soil secant Young's modulus E_s. For example, in the beam column/p-y model for clays,

E_s is determined through a point in the triaxial stress-strain curve at one half of the maximum (peak) strain. In practice the ratio E_s/S_u is taken to be between 50 and 200, and typically 100 [3]. The four case studies have indicated that a better indication of initial pile head stiffness can be made using $E/S_u = 300$.

For a given load, the soil strains are inversely proportional to the pile diameter. Since such large piles operate at relatively low load levels, the strains are extremely small for the majority of the soil mass. At such low strain levels, a higher soil stiffness value is in operation.

OFFSHORE PILE GROUPS

For piled platforms in high water depths, groups of piles are required. The piles are driven at relatively close spacings (2 to 3 diameter spacing). The lateral stiffness of the group is then a function of the single pile response, which has been described above, plus effects due to interaction between the piles.

The finite element method may be used to also improve the accuracy of the interaction factors. Errors, due to incompatibility between the beam column and continuum analyses, used in the Poulos/p-y method [21], are eliminated, since both the single pile and pile group behaviour are obtained from the same finite element analysis.

Briefly, the finite element output (Figs. 6 to 8) is re-analysed to determine the additional displacement occurring within the soil mass surrounding the loaded pile. By taking the displacements components in the direction of loading, the interaction factors for perfectly flexible piles can be derived. A typical result is shown in contour form (Fig. 9). More complex methods, incorporating pile stiffness, have been described elsewhere [11].

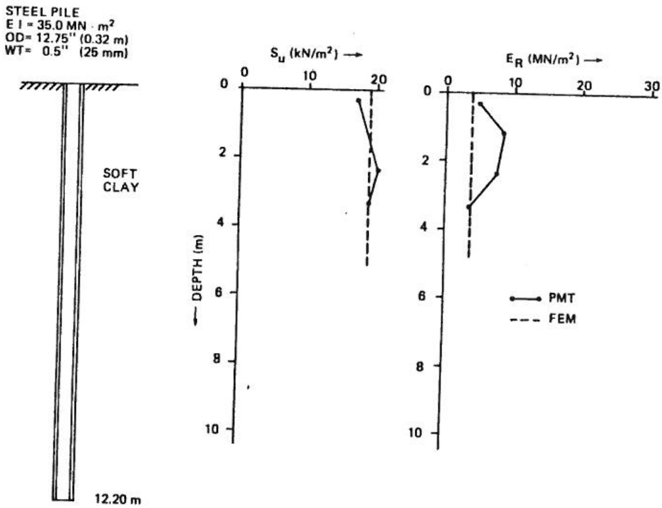

FIG. 1a Properties - Lake Austin Site

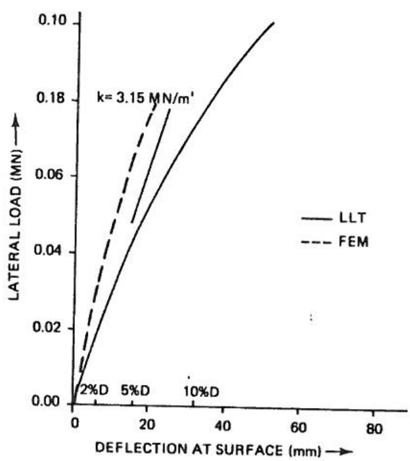

FIG. 1b Results - Lake Austin Site

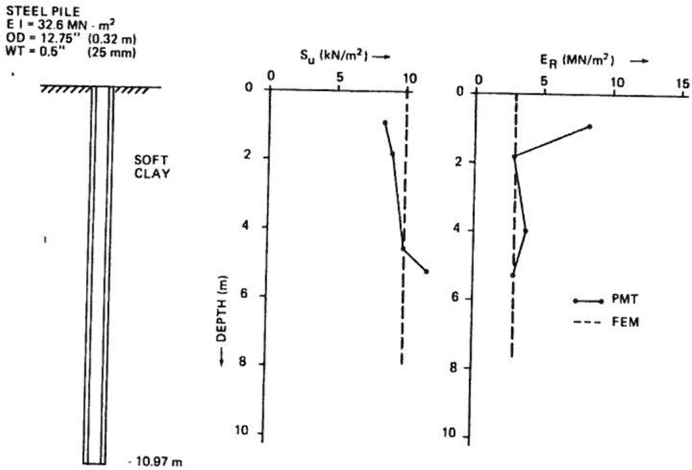

FIG. 2a Properties - Sabine Site

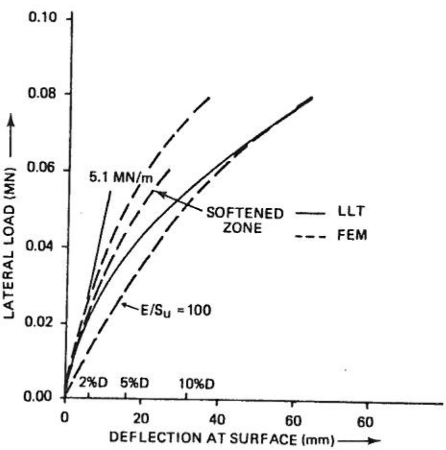

FIG. 2b Results - Sabine Site

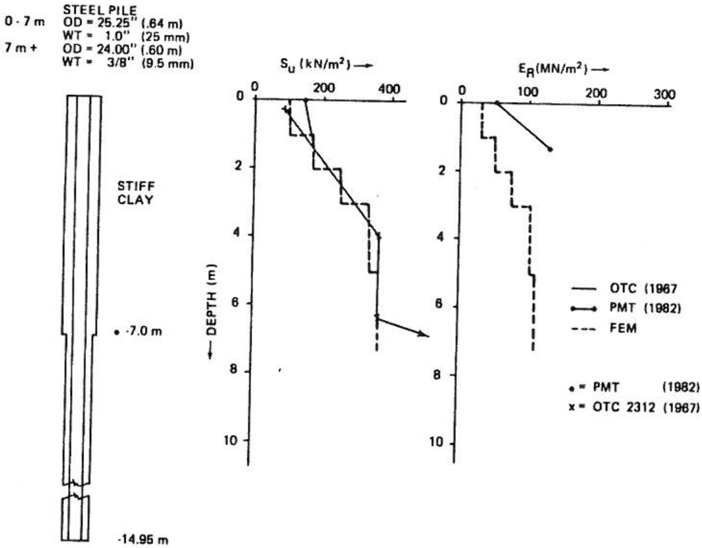

FIG. 3a Properties - Manor Site

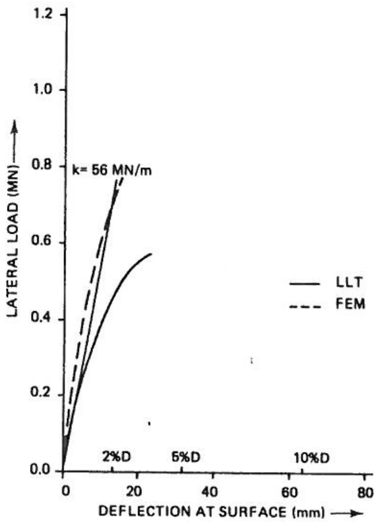

FIG. 3b Results - Manor Site 25"pile

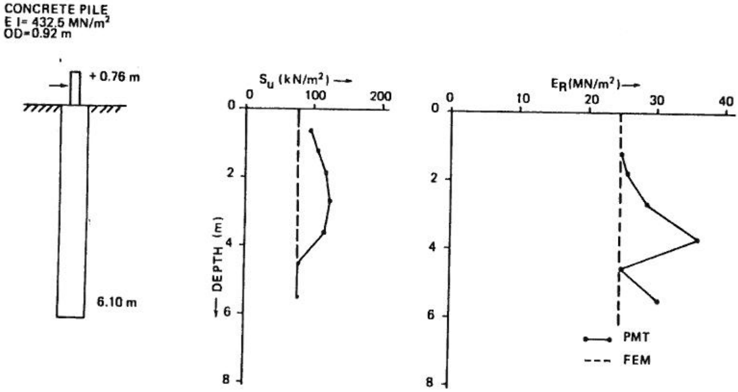

FIG. 4a Properties - Texas A&M Site

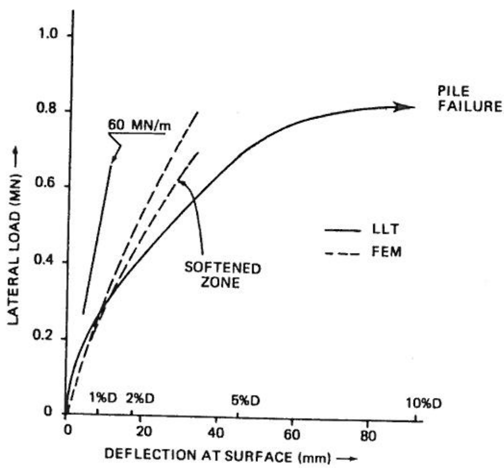

FIG. 4b Results - Texas A&M Site

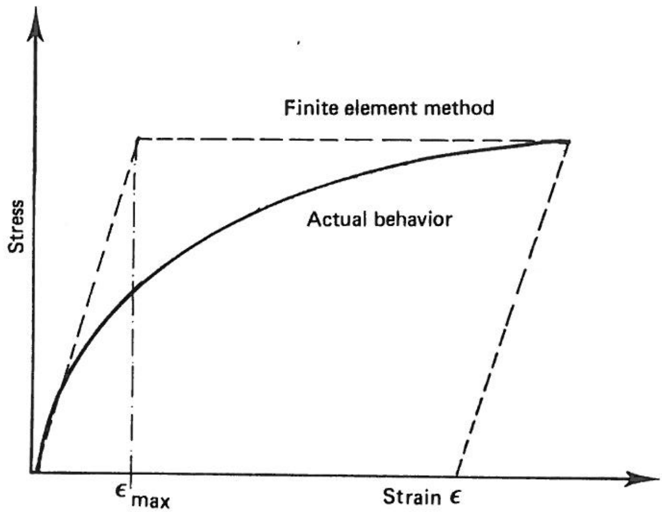

FIG. 5 Stress- strain curve

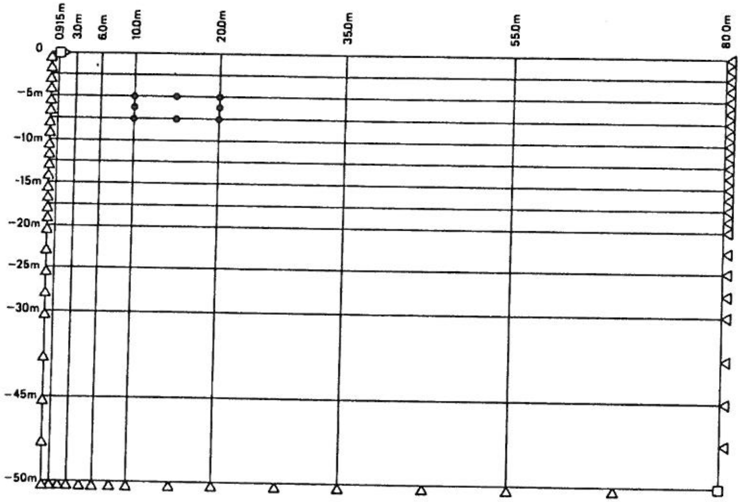

FIG. 6 Finite Element Mesh

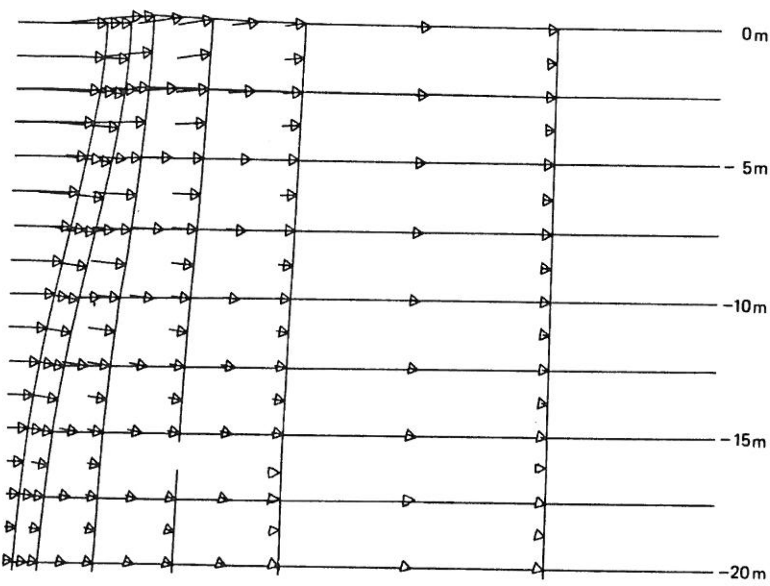

FIG. 7 Deformed Mesh and Displacement Vectors at Front of Pile

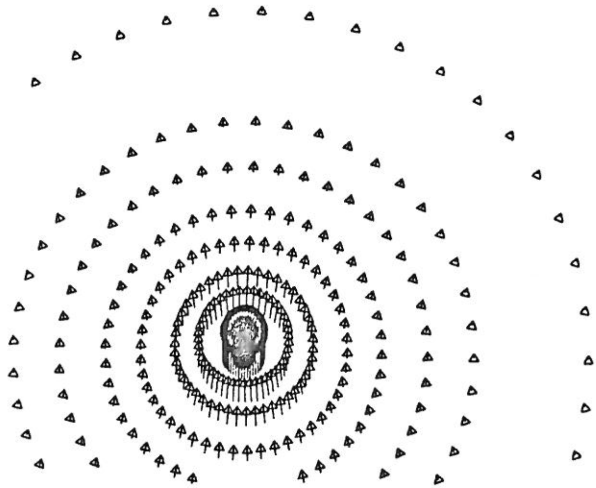

FIG. 8 Displacement Vectors at Ground Surface

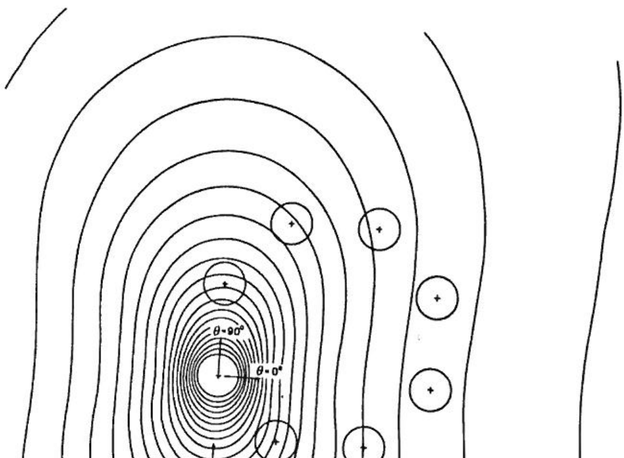

FIG. 9 Interaction Factors for Flexible Pile

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