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SYNOPSIS: The paper discusses briefly the state of art on the subject of pile dynamics including consideration of soil-pile interaction. An analytical model which gives the response of a single pile buried in a layered soil medium considering variation in soil properties in the radial direction in each layer is illustrated. The paper also presents an experimental study on a full size test pile 40 cm dia and 7 m long driven into a five layered soil stratum. The results of the analytical and experimental studies are compared and suggestions for further work are given.

INTRODUCTION

Pile foundations are being used to support machinery and structures which experience dynamic forces. Examples are machinery foundations, off-shore installations, nuclear power plant structures, chimneys, etc. The dynamic forces involved may be of steady state type, as in the case of rotating machinery or of a random nature, as in the case of wind, seismic or wave actions. The subject of pile dynamics has been engaging the attention of researchers for more than a decade now. Yet, the modelling of piles for dynamic analysis incorporating the composite action of the pile and the surrounding soil in a realistic manner is far from satisfactory. Analytical solutions suggested in literature are not often corroborated by experimental results to justify the theoretical assumptions, some of which are too idealistic in nature. Keeping this in view, the authors undertook a study of the dynamic behaviour of a full sized test pile, 40 cm dia and 7 m long, driven through a five layered soil stratum at the CSIR Campus site at Madras, India (Srinivasulu et.al., 1986). The object of this paper is to present salient details of this study and discuss the results obtained therefrom.

REVIEW OF THE STATE OF ART

A review of the current practices for the design of pile foundations experiencing dynamic loads suggests that while some of the methods (Barkan, 1962, Richart et.al., 1970) ignore pile-soil interaction altogether, some others deal with it in an empirical way. Among the latter, the method suggested by Singh et.al.(1977) is worth mentioning. This method suggests equivalent effective length of the pile for vertical vibrations and an equivalent bending length for horizontal vibrations under different site conditions. However, the method fails to represent a layered soil medium often encountered in practice.

Among the continuum-methods, Novak's plane strain model using complex soil reactions acting on discrete pile elements is popular (Novak, et.al., 1974, 1978). Plane strain approximation has been assumed in deriving modes of vibration. The original version of "PILAY" (SACDA, 1977) developed on this basis did not take into account the contact effects due to imperfect bond, slippage etc., near the periphery of the pile. Some improvements were effected by Novak himself in a subsequent model (Novak & Sheta, 1980) which considers an inner massless ring of visco-elastic medium having lower shear modulus and higher damping than the far field soil medium. The size of the inner ring has, however, not been suggested. Lakshmanan and Minai (1981) suggested what they called a non-homogeneous soil model to represent the reduced shear modulus in the soil region close to the pile boundary. This method will be further discussed in the next section.

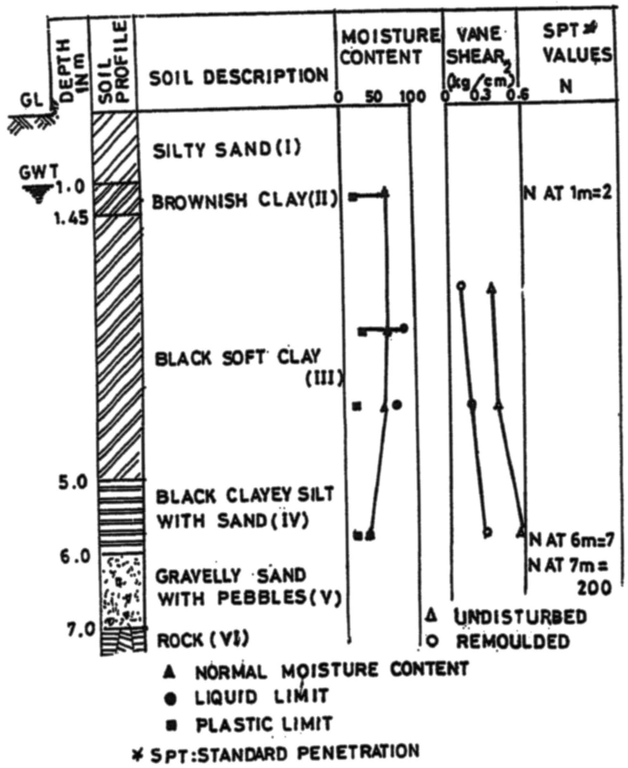
Among the classical finite element models, those involving axisymmetric elements to represent the pile and viscous energy absorbing boundaries to model the infinite boundaries in the soil medium (Kuhlemeyer, 1979) may be mentioned. These methods however, involve high computational costs and are not usually preferred, except for very special applications.

Among the experimental studies reported, those of Novak and Grigg (1976), Hall (1984) et.al., may be mentioned. Reported observations from experimental studies on full sized piles under actual field conditions are, however, still meagre. This paper aims to fulfil this need.

AUTHOR'S STUDY

Analytical Work

For the rigorous analysis of a single pile buried in a layered soil stratum, the non-homogeneous model (Lakshmanan et.al., 1981) which assumes a linear variation of the shear modulii in a certain region of soil surrounding the pile will be considered. The analytical work done to define the non-homogeneous model in all respects will be illustrated with particular reference to a case study example of a full size test pile shown in Fig.1. The pile has a circular cross-section with a diameter of 40 cm and a total height of 7.0 m. The surrounding soil has five distinct layers, the properties of which are separately evaluated. This data is given in Table 1.



Soil-Profile at Test Site

TABLE-1 SOIL PROPERTIES

Layer No.	Thick-ness (m)	Shear wave velo-city (V_s) m/sec	Density (γ) t/m^3	Young's modulus (E_s) t/m^2
I	1.00	175	1.68	14400
II	0.45	150	1.68	14400
III	3.55	85	1.68	3400
IV	1.00	150	1.94	22000
V	1.00	200	1.94	22000

Fig.2 shows the two analytical models involved in the dynamic analysis based on the continuum approach:- (a) a plane strain model assuming a homogeneous variation of shear modulus in the radial direction, and (b) the non-homogeneous model earlier referred to. In the latter, the shear modulus of soil is assumed to vary linearly from a value of C.Go (where $C < 1$) at the pile-soil interface to a value Go at a distance of d/2C where d is the diameter of the pile and Go is maximum shear modulus in the far field. The reduced shear modulus of soil is attributed to the high shear stresses in this region, the lowest value occuring at the pile-soil boundary. This model, therefore, divides the soil stratum in the radial direction into an inner ring having variable shear modulus of the above description and the outer medium which extends to infinity and possesses uniform visco-elastic soil properties.

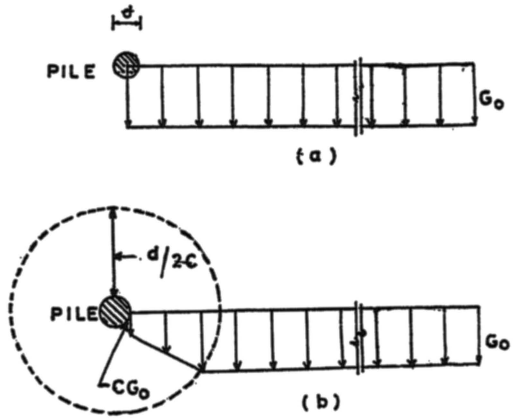


Fig.2 (a) Homogeneous Model
(b) Non-Homogeneous Model

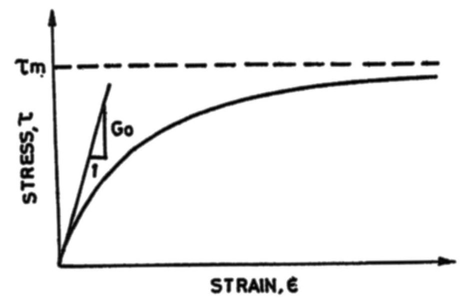


Fig.3 Typical Stress-Strain Variation in Soils

Response Under Static Vertical Loads

The general variation of shear stress with strain in soils is shown in Fig.3. According to the Ramberg-Osgood model, the shear modulus at any given shear stress τ is given by:

$$G_\tau = G_o \left[\frac{1}{1 + 3\left(\frac{\tau}{C_1 \tau_m}\right)^2} \right] \qquad (1)$$

where C_1 is a constant equal to 0.8 for sands and 0.4 for clays. Go is the initial tangent modulus which represents the highest value of G uniformly present in the far field of the non-homogeneous model of the pile-soil system.

"G" can be expressed in the form "CGo" where

$$C = \left[\frac{1}{1 + 3\left(\frac{\tau}{C_1 \tau_m}\right)^2} \right] \qquad (2)$$

It is possible to obtain C for each soil layer from the above equation, once the induced shear stresses (τ) in each layer under a given axial load on the pile are evaluated. To evaluate "C" by computation, the pile is discretised into finite elements, such that the boundaries between two adjacent soil layers coincide with the node points on the pile. To obtain the stiffness matrix of a pile element axially loaded at top and embedded in a homogeneous soil stratum, it is proposed to utilise the data provided by Poulos (1974) for an end bearing pile in the form of curves (Fig. 4 & 5) to yield the deflections at top and reactions at the tip of the pile respectively. If P is the load at the top of the element, then the deflection δt at top is given by:

$$\delta_t = I\left(\frac{PL}{E_p A_p}\right) \qquad (3)$$

where I is a reduction factor less than Unity. Ap and Ep are respectively the area of cross section and modulus of elasticity of the pile. Likewise, the tip reaction can be expressed in the form n.P where n is less than 1.0. The stiffness matrix of a pile element corresponding to the vertical deformations at its top and bottom can be written as:

$$[k] = \frac{E_p A_p}{I L}\begin{bmatrix} 1.0 & -n \\ -n & 1.0 \end{bmatrix} \qquad (4)$$

The variation of I for the case of l/d equal to 10.0 is chosen as the representative curve. A second order equation is fitted to the above curve as given below:

$$I = -0.0613x^2 + 0.5413x - 0.118 \qquad (5)$$

where $x = \log K$ and $K = E_p/E_S$.

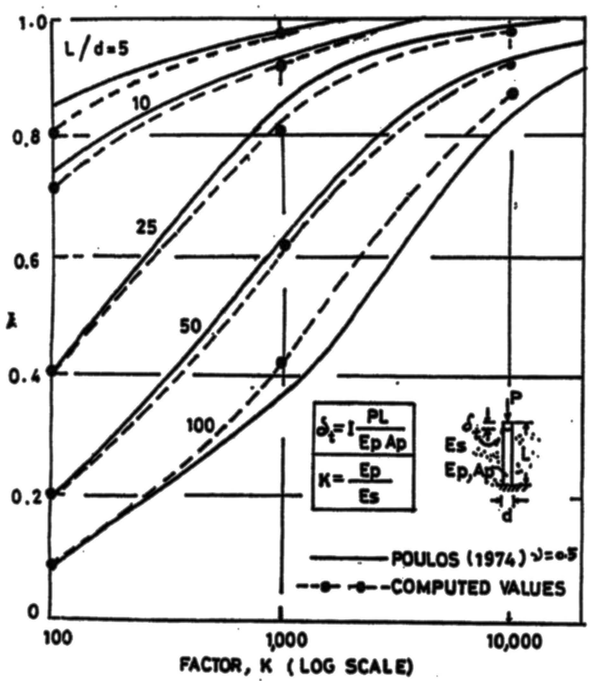


Fig. 4 Top Deformations for a Bearing Pile

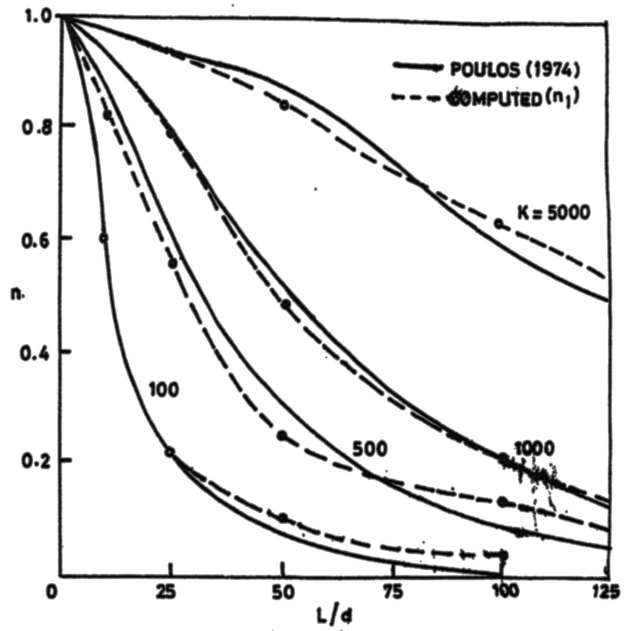


Fig.5 Tip Reactions for a Bearing Pile

Assuming linear variation of the load acting along the pile element as shown in Fig.6, the deflection δt can be written as:

$$\delta_t = \left(\frac{P + nP}{2}\right)\frac{L}{E_p A_p} \qquad (6)$$

From equations (3 & 6), the following relation is obtained:

$$n = (2I - 1.0) \qquad (7)$$

A pile element having a longer length of embedment in a uniform soil medium greater than 10d can be discretised into smaller segments each having a length of 10d and a last embedment of length (l_1) less than 10d (Fig.7). Equations (5) & (7) together may be used to define the load variation on the pile element as shown in this figure. The ordinate of

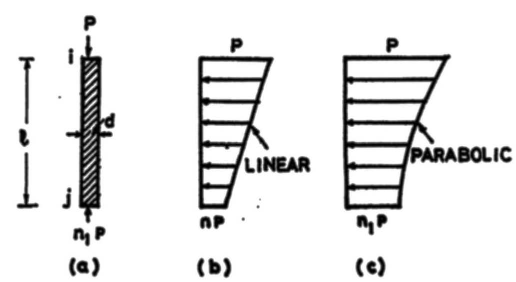


Fig.6 Load Variation on a Typical Pile Element in Uniform Soil

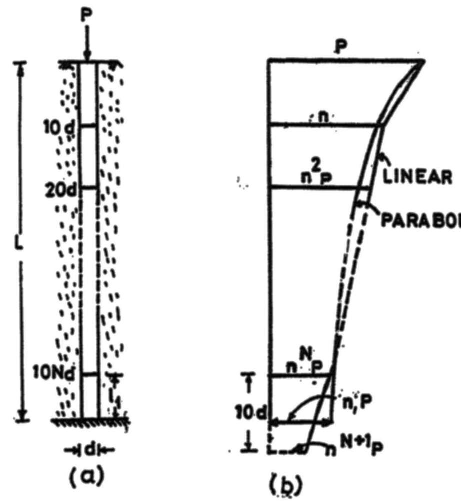


Fig. 7 Load Variation on a Long Pile Element in Uniform Soil

the load distribution diagram at the bottom of the pile element can be evaluated assuming a linear variation of the load in the last fictitious segment of length 10d of which, the last component of length 1 is a part. The deflection at the top of the pile element can now be computed using the load distribution diagram thus obtained and the given geometry of the pile. Fig.4 shows that the computed values of I agree reasonably well with the Poulos curves. It was however seen that the computed values of tip reaction (coefficient n) did not agree well with the Poulos' curves given in Fig.5 for various values of l/d and K. In order to get a closer agreement with the curves shown in Fig.5, a parabolic load variation is tried along the pile element (Fig.6c) in such a way that the resulting deflection at the top of the pile remain unchanged. This gave the modified relation for tip reaction coefficient n_1 in the form

$$n_1 = 3/2(I - 1/3) \qquad (8)$$

With the known value of I earlier evaluated, n_1 now represents the modified tip reaction coefficient. The minimum value of n_1 is taken as 0.5I. Fig.5 shows good agreement between the values computed on this basis and the Poulos' curves for all the values of l/d and K. Replacing n by n_1 in eq.(4), the stiffness matrices of successive pile elements can be formulated. The assembled stiffness matrix of the pile as a whole in a layered soil stratum is then obtained using the normal assembling procedures. Knowing the applied load at the top of the pile, it is now possible to evaluate the deformations at all the node points as well as stress resultants in the pile elements. If P_t and P_b are the force resultants acting at top and bottom of a pile element, the shear stress in the soil layer surrounding that element is given by $(P_t - P_b)/\pi d.1)$ The value of C which represents the reduction factor in shear modulus for any particular load on the pile is then deduced using eq.(2). When the computed shear stress in any soil layer exceeds the specified ultimate shear stress for that layer, the value of n_1 in the stiffness matrix (eq.4) of the particular element which is in contact with that layer of soil shall be replaced by unity.

In the theoretical formulation above explained, it is possible to include consideration of the elasticity of the bedrock at the pile tip which can be expressed in the form

$$k_b = \left[\pi r E_b / 2(1 - \nu_b^2) \right] \qquad (9)$$

where d is the diameter of pile; E_b and ν_b are modulus of elasticity and Poisson's ratio respectively of the bedrock below the pile tip.

The above analytical procedure has been programmed in Fortran IV on Prime 750 computer and is available in the name of "PILSTAT" at the Structural Engineering Research Centre, Madras (India).

Application to a Dynamic Environment

The discussion in the preceding section is applicable only to statically applied vertical loads on the pile. It is, however, believed that the results of the analysis used for deducing values of shear modulii under static loads can also be used in dynamic computations. This is better justified for applications in machine foundations, where the dynamic strains are relatively smaller than the static values.

Dynamic Soil Reactions

The complex soil reactions acting on unit length of the pile and for unit deformation under various modes of vibration, using the non-homogeneous soil model earlier defined can now be evaluated (Lakshmanan, et.al., 1981). These can be expressed as follows:

$$\text{Vertical : } K_v = G_o \left(Sw_1 + i Sw_2 \right) \qquad (10)$$

$$\text{Horizontal : } K_h = G_o \left(Sh_1 + i Sh_2 \right) \qquad (11)$$

The real part of the above equations represents the soil stiffness while the imaginary part denotes the damping offered by the soil. Figs.8 & 9 show the variation of the non-dimensional stiffness parameters Sw_1, Sw_2 and Sh_1, Sh_2 respectively for the vertical and horizontal vibrations of the pile-soil system shown in Fig.1. The above stiffness and damping parameters of soil are also functions of the reduced shear modulus coefficients (C) of each soil layer, its material damping value (DR), and the excitation frequency (f). A value of DR = 0.1 is assumed uniformly in this example. Figs.8 & 9 also show the variation of the stiffness and damping coefficients of soil for the case of homogeneous soil model assumed in the PILAY program. Considerable deviations are seen in the two sets of values which show the importance of the non-homogeneous soil model in dynamic computations.

Stiffness and Damping of the Pile-Soil System

The modified values of Sw_1, Sw_2 etc., are now used in place of the corresponding values derived from the PILAY program (SACDA, 1981) which is applicable for the plane strain case in a homogeneous soil medium. The PILAY program then yields the net stiffness and damping values of the pile-soil system for the non-homogeneous soil model earlier referred to.

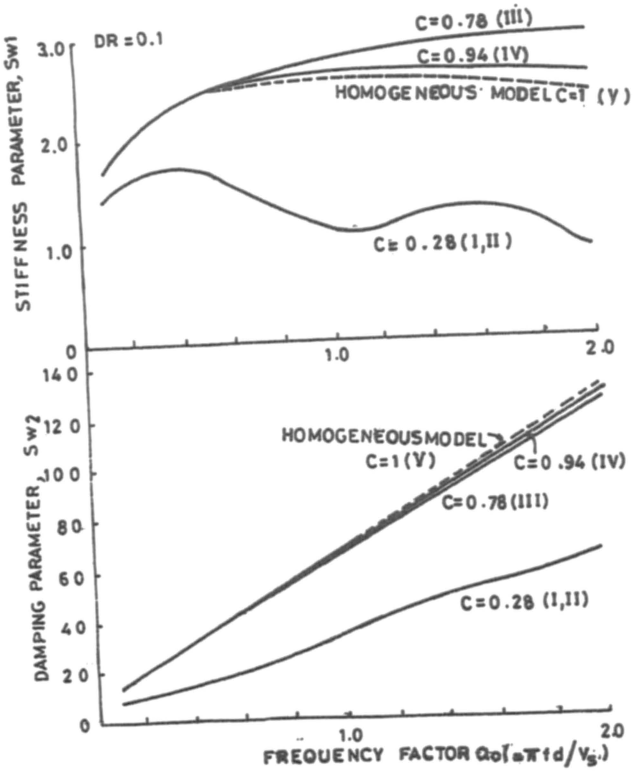


Fig. Variation of Sw_1 & Sw_2 with a_0
(Non-Homogeneous Model)

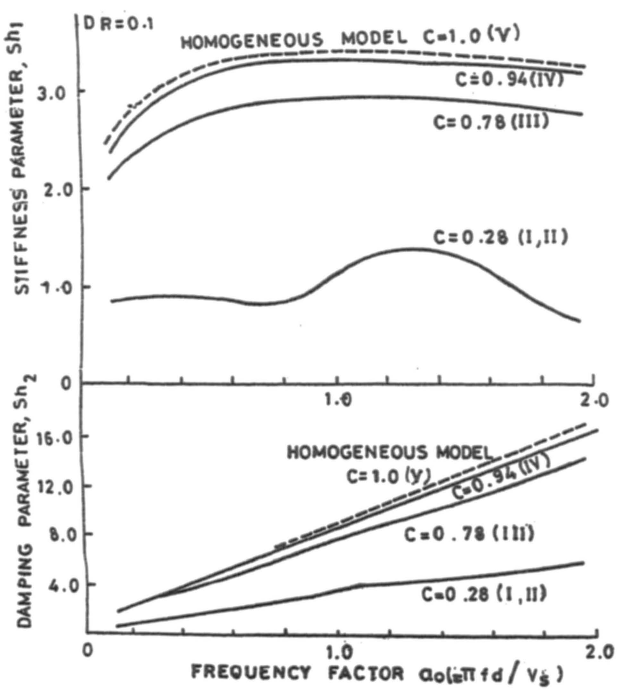


Fig. 9 Variation on Sh_1 & Sh_2 with a_0
(Non-Homogeneous Model)

EXPERIMENTAL STUDY

Static Tests

Fig.10 shows the static test set up used for loading the pile in the range of 0 to 100 tonnes. The load is incremented in stages and the average elastic deflection recorded by the dial gauges is noted after successive loading and unloading cycles. Fig.11 shows the variation of elastic deflection with the applied load as computed and as measured from the experiments. The agreement between the two sets of values is seen to be quite good, thus verifying the validity of the analytical model earlier explained.

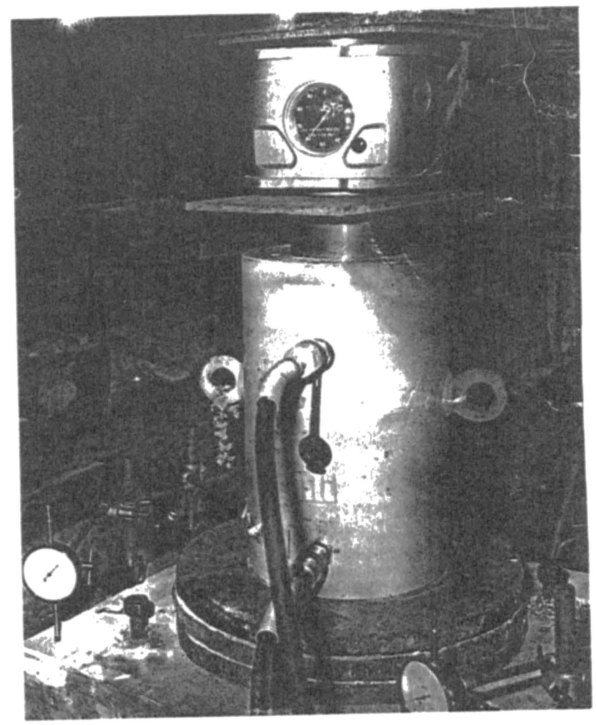


Fig. 10 Close View of the Static Test Set-up

Dynamic Tests

The purpose of this test programme was to verify under dynamic conditions, the validity of the radial non-homogeneous model earlier explained, using the values of C obtained from the static analysis. Vertical and horizontal dynamic tests have been conducted separately on the test pile by attaching at its top a mechanical oscillator coupled with a DC motor which induces steady state oscillations in the pile. Figs.12 & 13 show the test set up showing the pile under vertical and horizontal vibrations respectively.

The method of mounting the mechanical oscillator differs with the mode of vibration to be generated viz., vertical or horizontal. Provision exists in this oscillator to vary the eccentricity of unbalanced masses. The superimposed load on the test pile and exciting force level have been varied in the test programme. α is the mass ratio defined as the applied load normalised by the weight of the pile and R is

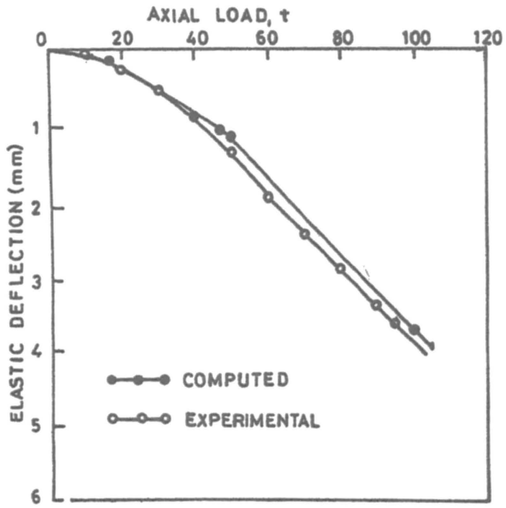


Fig. 11 Elastic Deflection Vs. Axial Load on the Test Pile

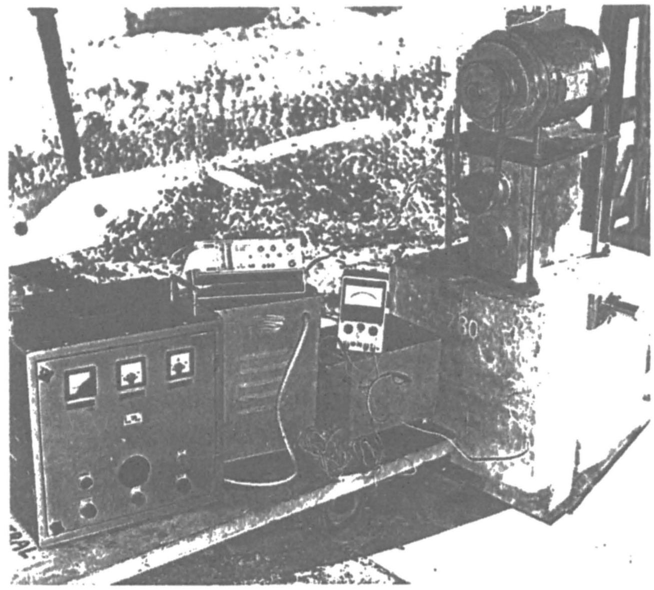


Fig. 13 Set-up for Horizontal Dynamic Test

a parameter which defines the dynamic force (F_d) given by the relation $F_d = Rf^2$, f being the frequency of excitation. Fig.14 & 15 show typical response plots obtained from the dynamic tests corresponding to one value of α = 4.53 and for varying values of R. The resonant frequencies are identified by the peaks of the response curves. Tables 2 & 3 show comparison of natural frequencies deduced from the theoretical models and experimental data. It is seen from Table 2 that there is no significant difference in the analytically deduced vertical frequencies using C = 1 (homogeneous mode I) and using variable values of C (non-homogeneous model) obtained from the static analysis. This may be attributed to the marginal contribution of the soil layers having low values of C. However, in Table 3 which shows the horizontal frequencies, the experimental values agree more closely with the non-homogeneous model than the homogeneous model assumed in the PILAY program.

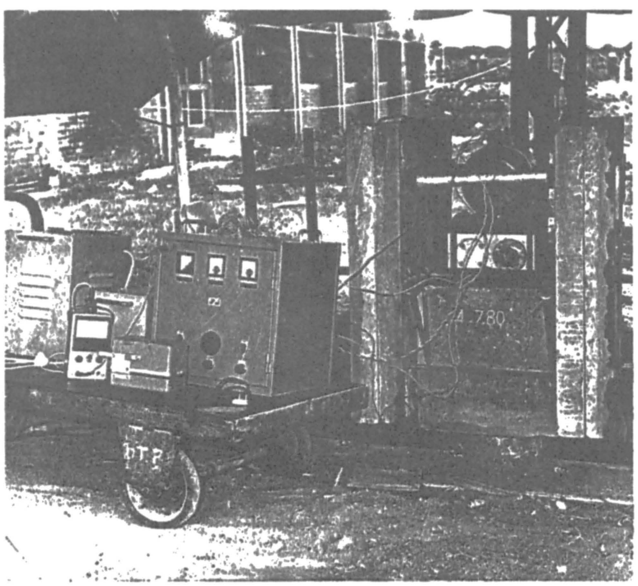


Fig. 12 Set-up for Vertical Dynamic Test

Fig. 14 Typical Pile Response (Vertical Dynamic Test)

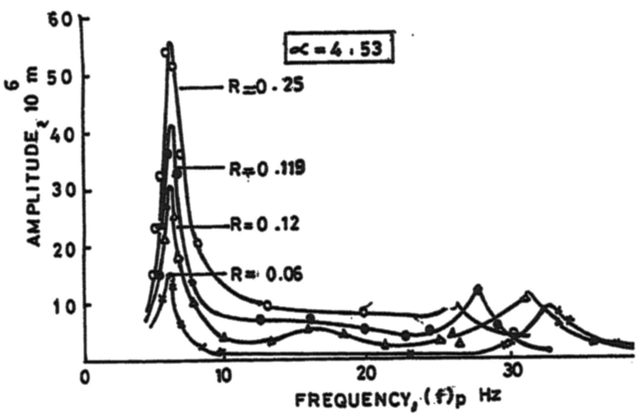


Fig. 15 Typical Pile Response (Horizontal Dynamic Test)

TABLE-2 COMPARISON OF FREQUENCIES UNDER VERTICAL EXCITATION

Mass Ratio α	Resonant Frequencies from tests (Hz)	Computed Frequencies (Hz) using	
		Homogeneous model (C=1)	Non-homogeneous model
0.35	111.3	97.24	90.45
0.67	79.2	76.80	74.48
0.99	49.0	67.87	64.09
1.30	43.3	59.20	57.88
2.56	43.0	43.60	43.05
4.53	32.0	33.11	32.64
6.89	27.0	26.84	26.78

TABLE-3 COMPARISON OF FREQUENCIES UNDER HORIZONTAL EXCITATION

Mass Ratio α	Resonant frequencies from tests (Hz)	Computed Frequencies (Hz) using	
		Homogeneous model (C=1)	Non-homogeneous model
0.35	22	27.33	25.87
0.67	15	19.73	19.15
0.99	13	16.22	15.89
1.30	11	14.15	13.95
2.56	9	10.08	9.63
4.53	6.25	7.58	6.50
6.89	4.1	6.14	4.40

TABLE 4 COMPARISON OF DAMPING VALUES UNDER VERTICAL VIBRATIONS

Mass Ratio α	Damping constant from tests	Computed damping value (t.sec/m) using	
		Homogeneous model C=1	Non-homogeous model
0.35	3.1	5.2	4.8
0.67	3.1	5.2	4.9
0.99	4.1	5.2	5.0
1.30	2.4	5.2	5.0
2.56	6.2	5.2	4.6
4.53	4.9	5.2	3.5
6.89	3.8	5.2	3.3

It may be seen that the agreement between the theoretical and experimental values is closer for higher values of α . Table 4 shows the damping values deduced from the theory of vertical vibrations using the two models aforementioned and the vertical vibration tests. The values given for the non-homogeneous model are seen to agree more closely with the experimental values.

CONCLUSIONS

The validity of the analytical model which considers the radial non-homogeneity of the soil, assuming a linearly varying shear modulus in a defined region of soil surrounding the pile has been verified. The reduction factor "C" for shear modulus at the pile-soil interface can be analytically deduced for each of the soil layers surrounding the pile, while it is carrying a certain axial load on its top. This procedure is illustrated in the paper on a full sized test pile. The values of C deduced from the static analysis can be used with confidence in dynamic computation as well. This is verified from the results of analysis and dynamic tests presented in the paper.

The agreement between the results of the analytical study using the non-homogeneous model and the experimental results is seen to be better at higher values of mass ratio. This shows the significance of the load carried by the pile in determining the influence of dynamic pile-soil interaction.

The results presented in the paper from an analytical model and the experimental study on a full sized test pile in the open field may serve as a useful reference data for the use of researchers engaged in this field. Further studies are however needed to study the effect of pile grouping and the interaction of the pile cap with the soil underneath it.

ACKNOWLEDGEMENT

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