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# Prediction and Performance of Piles under Static and Dynamic Loads

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# Predictions and Performance of Piles Under Static and Dynamic Loads

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SYNOPSIS:

Pile driving changes the soil properties for use in analysis of piles under lateral static and dynamic loads. In this study, the analytical predictions of pile group use soil and pile data obtained from tests on a single pile. The group predictions are compared with field and laboratory test results for both static and dynamic lateral response. A reasonably good agreement is found between the computed and measured dynamic response.

## INTRODUCTION:

Pile foundations are used extensively to transfer both static and dynamic loads to soil strata of adequate capacity. The soil-pile characteristies in the actual field conditions vary considerably from those determined from analytical models using soil properties from undisturbed samples. The influence of pile driving on the soil properties and pile response cannot be quantified using the available analytical models. Also, piles are commonly used in groups. The behavior of pile group subjected to static and dynamic loads is quite different than the behavior of a single pile.

The two factors mentioned above make the prediction of the performance of single pile and pile groups under lateral loads exceedingly difficult. Hence, soil-pile properties determined from tests on a single pile have been used to model the actual field conditions in this paper. These, coupled with a realistic evaluation of group effects should form a basis for better predicting the pile group response.

## PILES UNDER STATIC AND CYCLIC LATERAL LOADS

A comprehensive test program on single piles and pile groups in overconsolidated clay has been conducted at the University of Texas (Brown, 1985). Static and cyclic lateral loading tests have been reported by Brown et al (1987).

In these tests the pile group consisted of nine steel pipes, 10.75 in. (273 mm) outside diameter, with wall thicknesses of 0.365 in. (9.27 mm). The piles were installed in October 1979, in a 3 by 3 arrangement with a nominal spacing of 3-pile diameters. The piles were driven closed-ended into a layered system of overconsolidated clays to a depth of 43 ft. Prior to pile driving, a pilot hole 8 in. (203 mm) in diameter by 10 ft. (3.05 m) deep was excavated to facilitate vertical alignment of each pile. Both single pile and the 9-pile group were tested with no fixity at the pile head (in both cases).

Both static and cyclic lateral load test data has been reported. The load had been applied 1 foot above the ground level.

### Prediction for First Cycle of Loading

The single pile test data (curves A and B, Figure 6 of Brown et al, 1987) has been analyzed to determine the soil property as:

1. The deflection (y) at the load point in a fully embedded pile is given by

$$y = A_y \, Q \, R^3/EI + B_y MR^2/EI \tag{1}$$

where

$A_y, B_y$ = deflection coefficients

$Q$ = Applied load at pile top

$M$ = Applied moment at pile top

$R$ = Relative stiffness factor = $4 \frac{EI}{K}$

$k$ = soil modulus assumed constant with depth

$EI$ = flexural stiffness of the pile

2. On the basis of pile tests on groups in sand, Prakash (1962, 1981) and Davisson (1970) had recommended as follows:

"If the spacing of piles in the direction of load is 3d the effective value of k ($k_{eff}$) is 0.25 k where 'd' is diameter of the pile".

The pile spacing in this test series is 3d.

3. For s spacing of 3-d in the pile group soil modulus

$$K_{eff} = 0.25 \text{ k} \tag{2}$$

Substitution of value of $K_{eff}$ for K from Eq 2 in Eq 1 gives the following relationship:

$$y' = (2.87) A_y \, Q \, \frac{R^3}{EI} + (2) \, B_y \, \frac{MR^2}{EI} \tag{3}$$

where y' = deflection of a pile in group at the same load per pile as on single pile.

$$R^3_{eff} = 2.827\ R^3$$

and $R^2_{eff} = 2\ R^2$

Davisson and Gill (1963) calculated the "A' and 'B' coefficients for clays as:

$$A_y = 1.4 \qquad (4a)$$

$$B_y = 1.0 \qquad (4b)$$

Equation (1) has been solved for 'R' for several values of y in Table 1.

The values of y' computed from Eq 3 have been listed in Table 1. The predicted and measured load deflection curves are plotted in Fig. 1.

TABLE 1
RESULTS OF GROUP PILE CALCULATIONS
(FIRST CYCLE OF LOADING)

| S.NO | Deflection of Single Pile (y inches) | Load (Q-lbs) | Predicted Displacement of Pile Group (y'-inches) |
|---|---|---|---|
| 1 | .10 | 4200.00 | 0.27 |
| 2 | .20 | 7200.00 | 0.54 |
| 3 | .24 | 8300.00 | 0.65 |
| 4 | .30 | 9400.00 | 0.81 |
| 5 | .40 | 11600.00 | 1.08 |
| 6 | .50 | 13200.00 | 1.36 |
| 7 | .60 | 14600.00 | 1.63 |
| 8 | .70 | 15800.00 | 1.90 |
| 9 | .75 | 16300.00 | 2.04 |
| 10 | .80 | 16800.00 | 2.18 |
| 11 | .90 | 17800.00 | 2.45 |
| 12 | 1.00 | 18600.00 | 2.72 |
| 13 | 1.10 | 19600.00 | 3.00 |
| 14 | 1.20 | 20400.00 | 3.27 |
| 15 | 1.25 | 20800.00 | 3.41 |

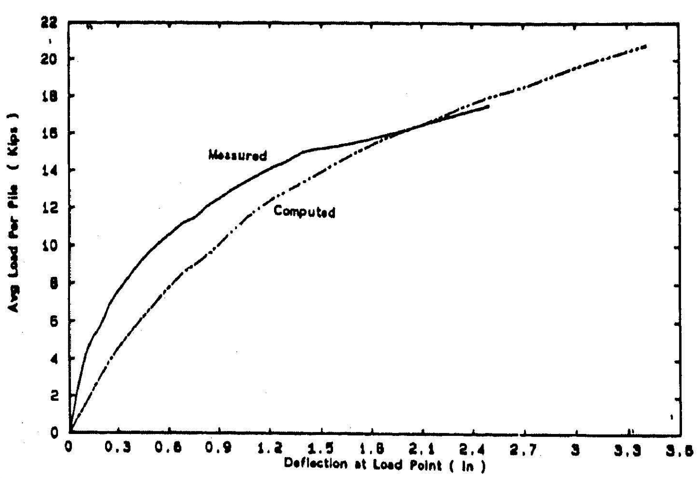


Figure 1 Comparision of Computed Deflections with measured deflections in 1 Cycle of Loading - 9 pile group (data from Brown et al., 1987)

Prediction for 100th Cycle of Loading

The cyclic load test data of single pile had been analyzed in the same manner and the corresponding results are shown in Table 2.

The predicted and observed load deflection curves are plotted in Figure 2.

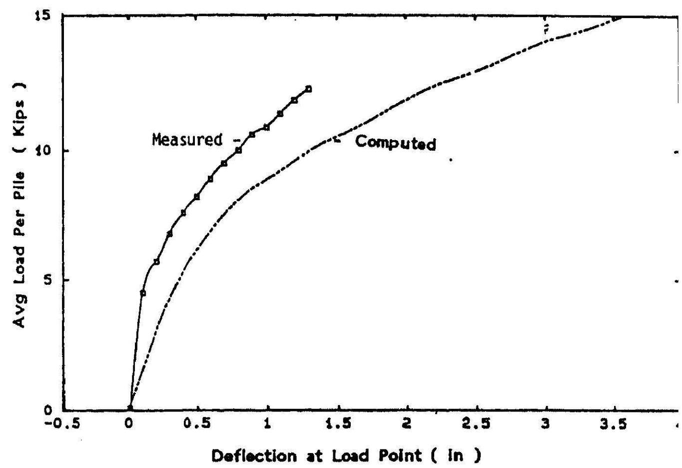


Figure 2 Comparison of Computed Deflections with Measured Deflections after 100 Cycles of Loading - 9 pile group (data from Brown et al., 1987)

Discussion

The full scale pile tests in this paper and the model pile tests of Prakash (1962) differ in the following respects (Prakash et al 1988):

1. The full scale pile tests are in over consolidated clay, while the model pile tests were in sand.

2. The full scale pile tests were performed with 2-directional loading, while the model pile tests were performed with one directional loading.

3. The full scale pile tests are performed with complete control of moment at the point of load application (M=0) while the model tests were performed with indeterminate moment (or rotation condition). The rotation of the pile cap had however been monitored.

Despite the above variations in the test conditions, it is extremely interesting to note that:

1. The results of load-deflection predicted for full scale pile group agree very well with the measured load-deflection.

2. The most significant conclusion that can be drawn on the basis of this comparison is that the analysis of single pile and pile groups according to theory of modulus of subgrade reaction predicts the behavior well provided a reasonable value of soil modulus is estimated.

3. The interaction effects under lateral loads both in sands and clay are of the same order.

TABLE 2

RESULTS OF GROUP PILE CALCULATIONS

(100th CYCLE OF LOADING)

| S.NO | Deflection of Single Pile (y inches) | Load (Q-lbs) | Predicted Displacement of Pile Group (y'-inches) |
|---|---|---|---|
| 1 | 0.1 | 3800. | 0.270 |
| 2 | 0.2 | 6300. | 0.541 |
| 3 | 0.3 | 8000. | 0.814 |
| 4 | 0.4 | 9000. | 1.087 |
| 5 | 0.5 | 10000 | 1.361 |
| 6 | 0.6 | 10700 | 1.635 |
| 7 | 0.7 | 11500 | 1.910 |
| 8 | 0.8 | 12200 | 2.184 |
| 9 | 0.9 | 12700 | 2.459 |
| 10 | 1.0 | 13300 | 2.734 |
| 11 | 1.1 | 13900 | 3.009 |
| 12 | 1.2 | 14300 | 3.285 |
| 13 | 1.3 | 14800 | 3.560 |

PILES UNDER LATERAL VIBRATION

Single Piles

Woods (1984), presented pile load tests in lateral vibrations in soft clay at Belle, Michigan. The natural frequency decreases with the level of excitation indicating a non-linear behavior of soil-pile system Figure 3. The pipe pile was 14" outside diameter with 0.375" wall thickness and 157 feet long.

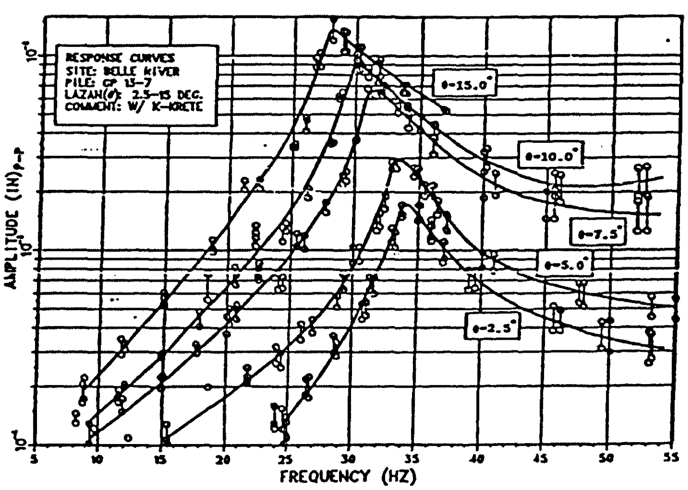


Figure 3 Response Curves of a Single Pile in Soft Clay in Lateral Vibrations (Woods et al., 1984)

The tests have been interpreted based on the following

1. The non-linear response of clays is characterized by relationship of $G/G_{max}$ versus shear strain ($\gamma_\theta$) is as in Figure 4 (Drnevich, 1985)

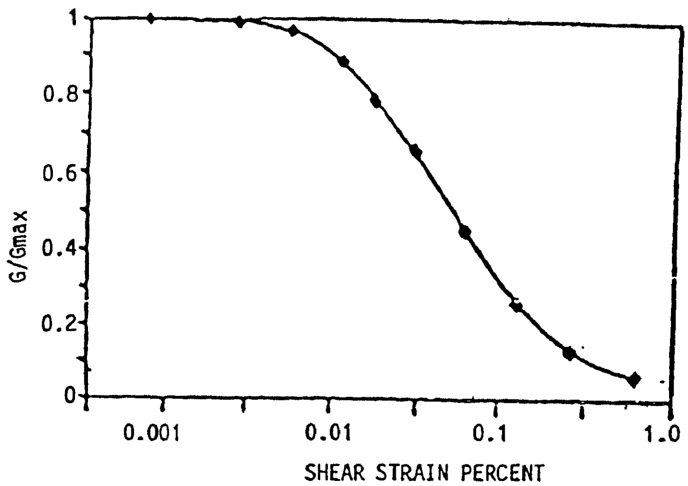


Figure 4 Normalized Shear Modulus Versus Shear Strain at Low Confining Stress (Drnevich, 1985)

2. The lateral vibration of pile has been considered as a one degree of freedom problem for this analysis.

The following procedure has been developed for predictions:

1. Read the natural frequency ($f_{nl}$) from the lowest graph in Figure 3.
2. Work back shear modulus G from the known value of $f_{nl}$.
3. The value of $G_{max}$ at low strain ($\gamma_\theta = 10^{-6}$) has been reported by Gle (1984) as 6564 psi. Now compute $G/G_{max}$ and determine from Figure 4.

   This strain corresponds to a peak amplitude of 1.8 x $10^{-3}$ inches.
4. The strain values for any other peak amplitude can now be determined by using proportionality.

   The corresponding values of "G" can then be back calculated using Figure 4.
5. With the values of "G" as determined above, the natural frequency $f_{nl}$ at other strain levels is predicted, using relationships and constants proposed by Novak and Elsharnouby (1983). For ratios of $E_p/G_s$ greater than 10,000, the 'f' factors were extrapolated using a cubic spline function of 5th order.

   The computations are shown in Table 3. The computed and measured response curves are shown in Figures 5 to 7.

TABLE 3

PREDICTED AND MEASURED NATURAL FREQUENCIES OF SINGLE PILE

| S.No | $G_*$ (psi) | $G_*/G_{max}$** | $\gamma_\theta$ | $Z_{max}$ (in)x$10^{-3}$ | Frequency Measured | Frequency Predicted |
|---|---|---|---|---|---|---|
| 1 | 5772.* | 0.88 | 0.0126 | 1.8 | 34 | Reference |
| 2 | 5705. | 0.8692 | 0.014 | 2.0 | 33 | 31.4 |
| 3 | 3583. | 0.546 | 0.0455 | 6.5 | 31.5 | 26.6 |
| 4 | 3019. | 0.46 | 0.063 | 9.0 | 30.0 | 25.0 |
| 5 | 2625. | 0.4 | 0.0735 | 10.5 | 27.5 | 23.5 |

*Calculated value

** From Fig. 4.

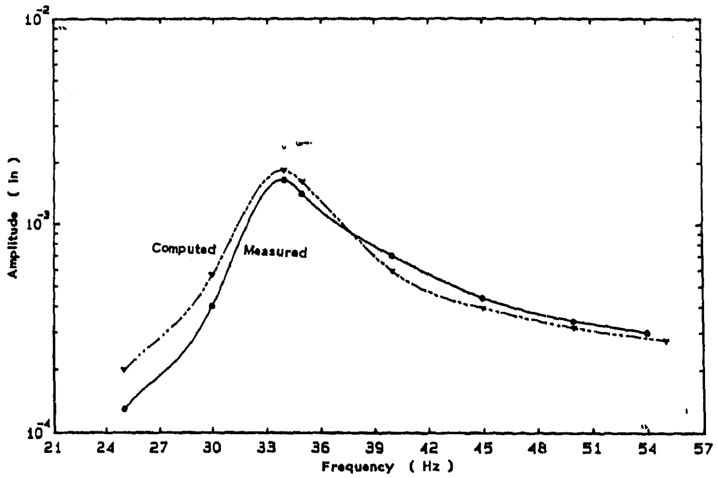


Figure 5 Comparision of Measured and Computed Displacement Response of a Single Pile ($\theta$ = 2.5) (data from Woods et al., 1984)

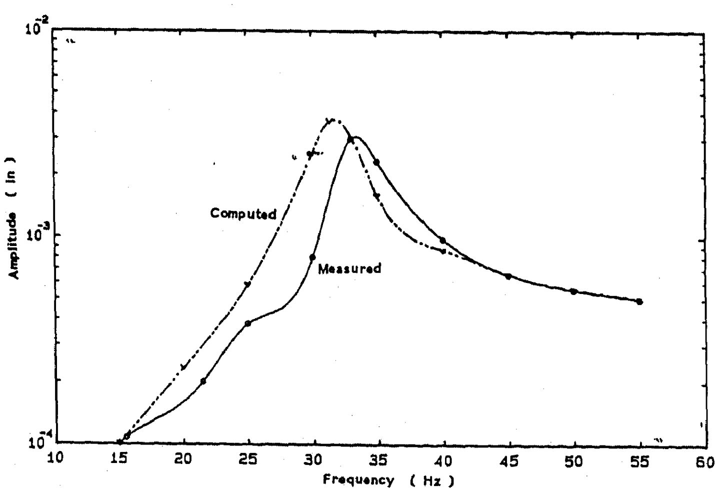


Figure 6 Comparision of Measured and Computed Displacement Response of a Single Pile ($\theta$ = 5.0) (data from Woods et al., 1984)

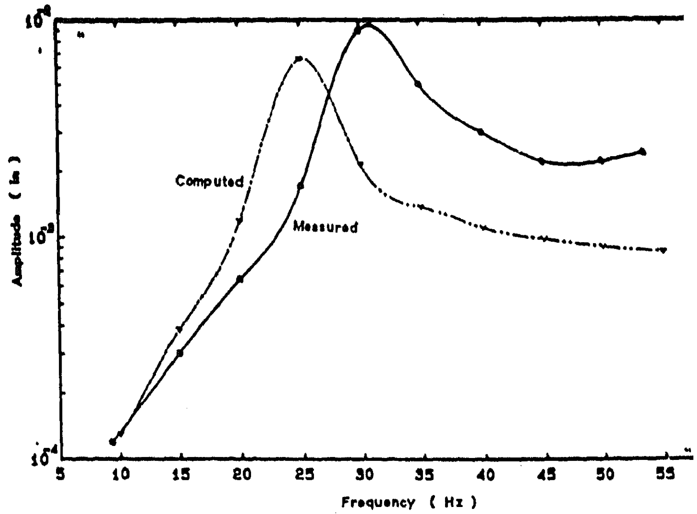


Figure 7 Comparision of Measured and Computed Displacement Response of a Single Pile ($\theta$ = 15.0) (data from Woods et al., 1984)

Pile Groups

Yao (1980) reported results of model pile and pile groups in clay under lateral vibrations. The pertinent details are:

1. At the bottom of clay layer in the soil box, various frequencies of sine waves are transmitted through the vibratory table which is equipped with the vibration generator. The soil box has light rotative walls which hold the clay layer in vibratory direction, to avoid both the unnatural confinement of soil movements and the excessive dynamic inertia reactions from rotative walls.

2. The clay layer is enclosed by the vinyl plastic sheets and glycerin to contain pore water and to cut the frictional resistance between the soil and soil box.

3. Steel mass and Televibrometer of 31 kg total weight, corresponding to one third of vertical ultimate strength of group piles, are fixed on the footing.

The details of the soil pile data are as follows:

Pile Data:

O.D. = 10 m.m.

I.D. = 8 m.m.

$E_p = 7.58 \times 10^5$ Kg/cm$^2$ $\qquad D_o = 0.9$ cm

$I_p = 2.89 \times 10^2$ mm$^4$

$A_p = 0.2827$ cm$^2$

Mass = 31 kg

Soil: $V_s = 1374.95$ m/sec

$G_s = 3242.72 \times 10^5$ kg/m$^2$.
$= 460860$ psi

Single pile test data (Test No. 6) was analyzed as explained previously. The group factors had been computed by Poulos (1979) method for static lateral interaction. The stiffness and damping of the pile group was estimated as recommended by Novak (1974).

Using the estimated values of stiffness and damping of the pile group, the response of the pile group was computed. The computed and measured pile group response are plotted in Figure 8.

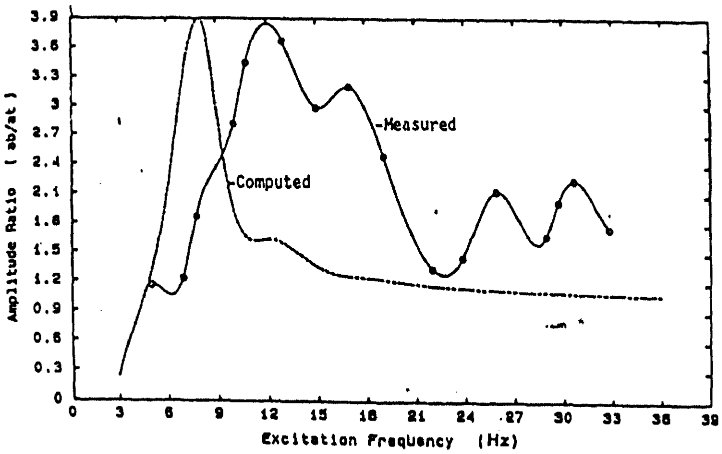


Figure 8 Comparision of Measured and Computed Pile Head Amplitude Response of a Model Pile Group (data from Yao, 1980)

## CONCLUSIONS

Prediction of pile behavior under static and dynamic lateral directions has been predicted based on the single pile test as a reference.

Realistic interaction effects were used for prediction of group behavior.

A reasonably good tally has been found between the predicted and measured response.

Further work is being carried out on this project at UMR.

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