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Predicted and Measured Performance of Piles in Calcareous Soils

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SUMMARY A numerical modelling study has been conducted to develop a procedure for predicting the behaviour of piles embedded in calcareous soils. A non-linear soil model has been implemented in a boundary element analysis to determine the pile performance under static loading. The parameters required have been determined from laboratory tests on model piles, and the theory is shown to reproduce the model pile behaviour quite accurately over a wide range of displacement.

The data from these tests has also been used to predict the behaviour of two different published full-scale grouted pile load tests in calcareous soils. It is found that the predicted behaviour is in good agreement with the field measurements.

INTRODUCTION

Reliable predictions of the static ultimate axial load capacity of piled foundations in calcareous soil are difficult to achieve due to the unusual nature of these soils. Their behaviour is characterised by a tendency to crush under moderate stresses and to exhibit volume reduction during shear. Consequently, both the ultimate skin friction and end-bearing capacity of piles in these soils is less than in conventional silica-based sands, and the stiffness and compressibility characteristics of the two types may differ significantly. To better model the behaviour of piles in calcareous soils, a non-linear model of pile-soil interface behaviour has been developed and implemented in a boundary element (BEM) analysis.

This paper briefly describes this model and its application to the prediction of actual field tests and compares predicted and observed load-settlement behaviour. The required soil parameters can be derived from laboratory model pile test data. Some of these predictions are also compared with those from an alternative published analytical model.

Outline of Analysis

The pile can be divided into cylindrical shaft elements, annular base elements or annular elements at discontinuities in shaft diameter, as shown in Figure 1.

The following equation can be obtained from consideration of compatibility of incremental pile and soil vertical displacements (Poulos, 1987);

$$\left[\frac{I}{E_s} - AD.FE\right]\{\Delta p\} = \Delta\rho_b\ \{1\} \qquad (1)$$

in which

$[I/E_s]$ = matrix of soil influence factors determined from elastic theory (Poulos and Davis, 1980), divided by E_s, the non-linear 'soil modulus'.

[AD] = summation matrix

[FE] = pile compression matrix

$\{\Delta p\}$ = vector of incremental pile-soil interaction stresses

$\Delta\rho_b$ = incremental displacement of pile base

{1} = vector whose elements are unity.

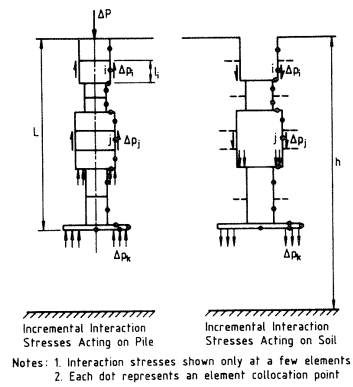


Figure 1 Division of pile into elements

The modified Ramberg-Osgood formulation (Ramberg and Osgood 1943; Hara 1980) is adopted to simulate the non-linear pile-soil interface behaviour. The incremental tangent 'soil modulus', E_s, can be obtained by differentiating the following equation:

$$p = \frac{E_m \epsilon}{1 + \alpha\left|\frac{p}{p_f}\right|^{R-1}} \qquad (2)$$

where

E_m = initial maximum tangent 'soil modulus'

p, ϵ = current stress and normalised strain (displacement/pile diameter) respectively

p_f = limiting resistance

α, R = experimentally - determined parameters.

The physical significance of α and R is demonstrated in Figure 2.

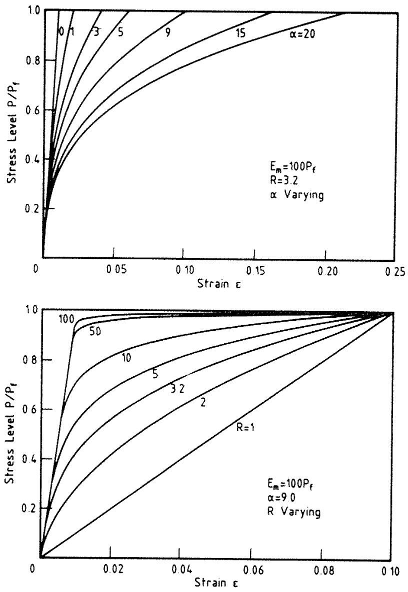


Figure 2 The significance of non-linear model parameters

In addition, vertical equilibrium requires that:

$$\sum_{i=1}^{n} A_i \, \Delta p_i = \Delta P \qquad (3)$$

where

A_i = surface area of element i

Δp_i = interaction stress increment on element i

ΔP = increment of applied load on pile head

n = total number of elements.

Equations (1) and (3) may be solved for the unknown interaction stress increments Δp and base incremental displacement $\Delta \rho_b$. To allow for pile-soil slip or yield of an element, when p_i reaches the limiting resistance p_f, the displacement compatibility equation for element i in Equation (1) is replaced by the condition:

$$\Delta p_i = p_f - p_{ti} \qquad (4)$$

where

p_{ti} = total pile-soil stress at previous load increment.

The solution is then recycled until, for all elements, the total pile-soil stress is less than or equal to the limiting value.

For non-homogeneous soil, the incremental soil displacement for an element i, $\Delta \rho_{sij}$, due to element j can be approximated as (Poulos, 1979);

$$\Delta \rho_{sij} = \frac{I_{ij}}{0.5\,(E_{si} + E_{sj})} \cdot \Delta p_j \qquad (5)$$

where

I_{ij} = soil influence factor, evaluated from integration of Mindlin's equation;

E_{si}, E_{sj} = values of 'soil modulus' at elements i and j;

Δp_j = incremental interaction stress on element j.

The vector of incremental soil displacements can then be assembled accordingly, and the displacement compatibility equation (equation 1) solved as before.

Evaluation of Nonlinear Model

Model pile tests have been carried out in a specially-designed apparatus to investigate the skin friction of grouted piles in calcareous soil by the authors (1987). The procedure described by Cervantes et al (1973) was adopted to backfigure the non-linear model parameters E_m, p_f, α and R from the model tests. The values of E_m obtained from these model tests, and from conventional drained triaxial tests, are similar, as shown in Table 1. It was also found that the parameter R is independent of effective overburden pressure, σ'_{vo}, at least in the range 100 kPa $\leqslant \sigma'_{vo} \leqslant$ 400 kPa and has an average value of 3.2, while α has an average value of 9.0.

Table 1 Values of Soil Modulus, E_m

σ'_{vo}	E_m (MPa)	
kPa	Model Test	Triaxial Test
100	50	40
200	60	72
400	120	136

To assess the capability of the non-linear BEM model, these backfigured parameters were used to predict the measured model test results at different values of overburden pressure σ'_{vo}, as shown in Figure 3. This figure indicates that the stiffness and resistances of the model tests can be predicted reasonably accurately by the non-linear model over the whole range of load to failure. However, if a purely linear model is used, it is only able to predict accurately the displacement up to a load of about 20% of the ultimate value.

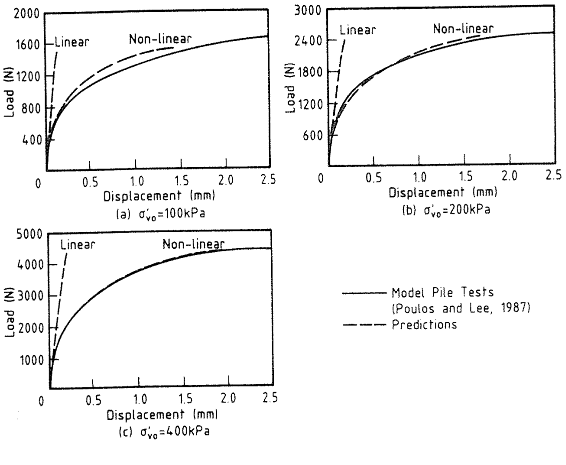
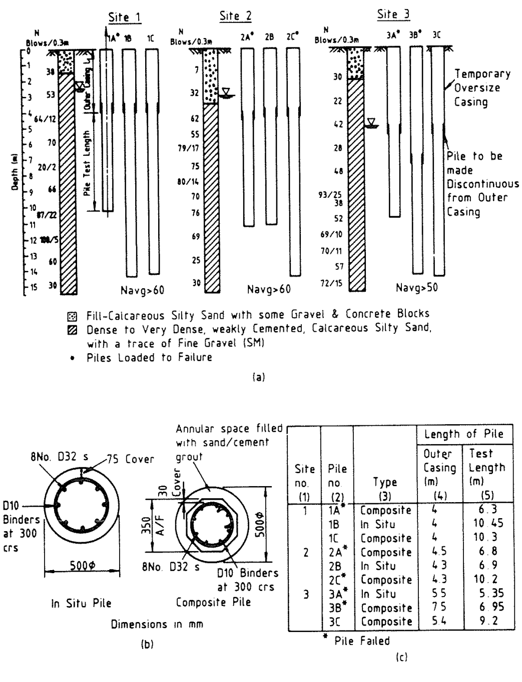


Figure 3 Load-displacement curves

Field Load Tests

Tests of Ismael et al (1986)

Field test results on grouted or concrete piles in calcareous soils are very limited at present. Ismael et al (1986) performed a series of such tests at three different sites in Kuwait. At each site one reinforced in-situ bored pile and two composite piles were installed and tested. The soil profiles and pile details in these tests are shown in Figure 4. Only five of the piles were tested to failure.



Site no. (1)	Pile no. (2)	Type (3)	Length of Pile: Outer Casing (m) (4)	Length of Pile: Test Length (m) (5)
1	1A*	Composite	4	6.3
	1B	In Situ	4	10.45
	1C	Composite	4	10.3
2	2A*	Composite	4.5	6.8
	2B	In Situ	4.3	6.9
	2C*	Composite	4.3	10.2
3	3A*	In Situ	5.5	5.35
	3B*	Composite	7.5	6.95
	3C	Composite	5.4	9.2

* Pile Failed

(c)

Figure 4 Soil conditions and pile details at the test sites (Ismael and Al-Sanad, 1986)

Since there were no model test results available for the soils at these test sites, the soil parameters required for the non-linear BEM model (except α) were derived from authors' model tests (Poulos and Lee, 1987). The α values were backfigured from the load-displacement curves for the piles not tested to failure (see Figure 5). The parameters used in the non-linear model are shown in Table 2.

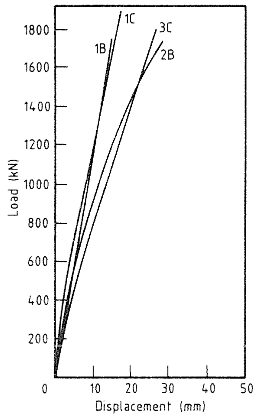


Figure 5 Load deflection curves for piles not tested to failure (Ismael and Al-Sanad, 1986)

Table 2 Parameters used in Non-linear Model

Test	E_m MPa	α	R	P_f MPa
Ismael et al (1986):				
1A	50	5	3.2	0.045 + 0.0105Z
2A	50	2	3.2	0.045 + 0.0105Z
2C	50	2	3.2	0.045 + 0.0105Z
3A	50	12	3.2	0.058 + 0.0105Z
3B	50	14	3.2	0.079 + 0.0105Z
Nauroy et al (1985b):	200	1	3.2	0.2

Z = soil depth below ground surface.

Except for the results at the initial stages, where some of the field measurements were doubtful, the non-linear BEM model was found to simulate the response of the field tests adequately, as demonstrated in Figure 6. It can also predict the ultimate load of the field tests very well.

Tests of Nauroy et al (1985)

Another pile load test was carried out in Western France by Nauroy et al (1985a and 1985b). The test pile was a steel tube of 16 m long, 220 mm OD and 10 mm wall thickness grouted into a 310 mm diameter hole. The pile was instrumented by strain gauges attached to the outside wall along three axes 120° apart. Details of the soil profile at the test site and the instrumented pile are given in Figure 7.

Another model, PSAS (Pile-Soil-Analysis-System), was used by Bea et al (1986) to predict this field test. For consistent comparisons, the soil parameters required by both PSAS and the non-linear BEM model were backfigured from model pile tests carried out by Nauroy et al (1985a and 1985b).

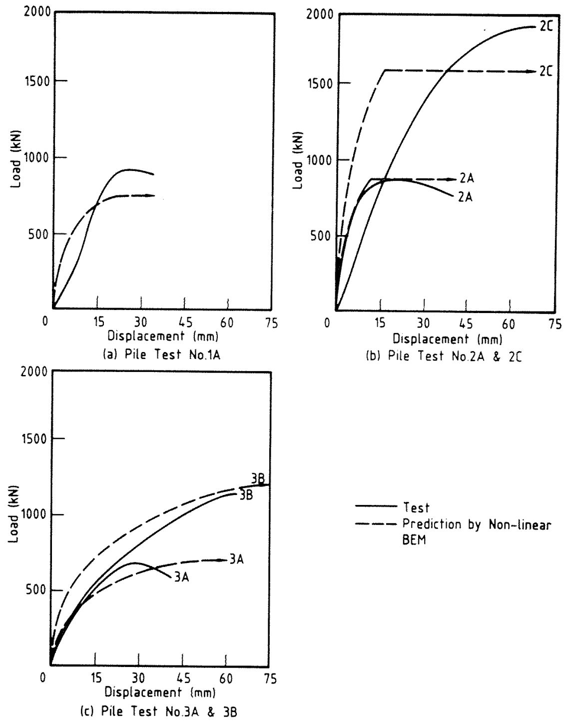


Figure 6 Load-displacement curves

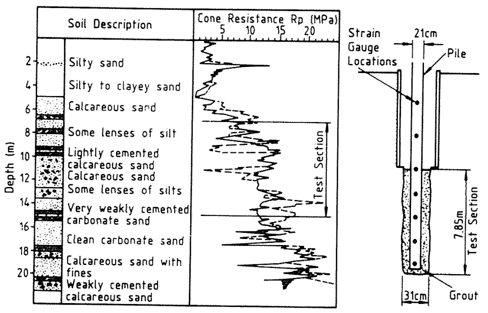


Figure 7 IFP Plouasne, Brittany pile load test pile and soils (Bea et al, 1986)

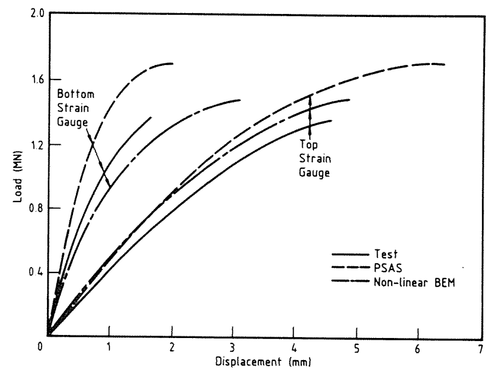


Figure 8 Comparison of predicted and measured pile resistances and displacements

The stiffness and resistance predicted by both models are in excellent agreement with those measured in the field, as shown in Figure 8.

Conclusions

This paper presents a simple non-linear interface model incorporated into a boundary element analysis to simulate the behaviour of pile shaft in calcareous soils. The soil parameters required by this model are the maximum initial tangent 'soil modulus' E_m, limiting resistance p_f, α and R. All these parameters are easily determined from a single model pile test. The values of E_m backfigured from model pile tests and conventional drained triaxial tests are similar. The predictions by this model are in good agreement with the measured load-settlement behaviour in the field.

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