# Class A Predictions of Pile Behaviour

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**SUMMARY** Class A (before the event) predictions were made of the load-settlement behaviour of four different test piles at a site in Evanston, Illinois. The paper describes the prediction procedures adopted and the correlations employed to assess the required geotechnical parameters. Predictions are presented for the load-settlement behaviour, the distribution of load with depth and the distribution of residual load after installation of the driven piles. These are then compared with the measured behaviour of the test piles.

## INTRODUCTION

Class A (before the event) predictions of the axial behaviour of four different pile types wei made in 1988. Together with a number of other predictors, the author presented these predictions at the ASCE Foundation Engineering Congress held in the United States in June 1989, after which the results of the load tests were revealed.

This paper outlines the procedures adopted by the author to predict the pile behaviour, and then compares the predicted behaviour with that observed. The main predictions were of the axial load capacities of the four pile types, at various times (2 weeks, 1 month, 1 year) after installation. However, predictions were also made of the load-settlement behaviour, the axial load distribution along each pile, and the residual load distribution in two of the piles after installation by driving.

## GEOTECHNICAL DATA

The test site was located on the campus of Northwestern University in Evanston, Illinois. A considerable amount of geotechnical data was available from both insitu and laboratory tests.

A complete description of the data is presented by Finno (1989), but in summary it consisted of the following:

(1) a soil boring to a depth of 21.8m;
(2) four CPT soundings;
(3) SPT data;
(4) Menard pressuremeter tests at five depths;
(5) two dilatometer soundings;
(6) piezocone data, including three dissipation tests;
(7) data from four piezometers;
(8) pile driving data.

The laboratory data included:

(1) grain size distributions;
(2) Atterberg limits and natural water content;
(3) one dimensional consolidation data;
(4) undrained shear strength data from unconsolidated undrained tests and direct shear tests;
(5) limited stress-strain data from consolidation undrained triaxial compression and extension tests.

A summary of some of this data is given in Figure 1, which shows that there are two predominant layers within the depth of the test piles - an upper sand layer approximately 6.1 to 7.3m thick, underlain by a relatively soft clay layer which extends to about 18.3m depth. Below that depth, much stiffer layers of silty and sandy clay exist.

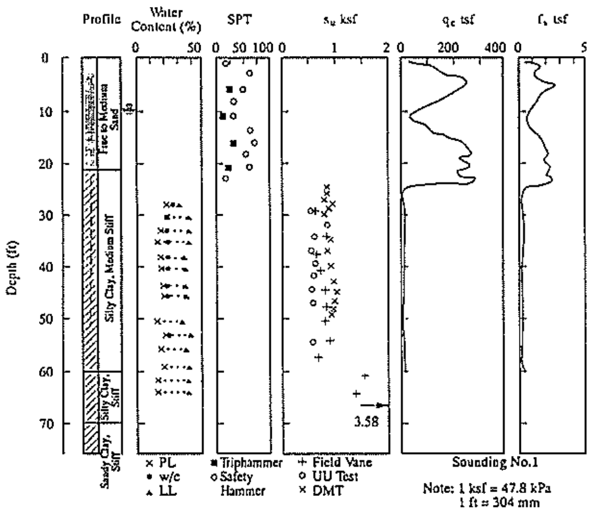


FIG.1 SUMMARY OF GEOTECHNICAL PROFILE AND DATA PROVIDED

## TEST PILE DETAILS

Four piles were tested:

(1) Pile 1, a driven steel tube pile 0.457m in diameter with 9.5mm wall and 0.483m boot plate;

(2) Pile 2, a driven steel H-pile (14x73) section;

(3) Pile 3, a drilled pier constructed under slurry, having a nominal diameter of 0.61m from the surface to about 3.05m depth and a nominal diameter of 0.457m below that depth;

(4) a drilled cased pier with a nominal diameter of 0.61m to a depth of about 9.5m and 0.457m thereafter.

## PREDICTION PROCEDURES

A detailed description of the author's prediction process is given elsewhere (Poulos 1989) and hence only a summary of the approach is presented here. It had been intended to employ an effective stress approach to predict the shaft resistance in the sand and clay and a total stress method to estimate base resistance in the clay. However, reliable data was lacking for a key parameter, the coefficient of earth pressure at rest, and in addition there was some doubt about the ability of refined analyses to predict accurately the stress state around both driven and bored piles after installation.

Consequently, it was decided to use one of the simplest, and least sophisticated approaches, that of correlating skin friction $f_s$ and end bearing resistance $f_b$ to CPT data. In the approach adopted, $f_s$ and $f_b$ were estimated as follows:

$$f_s = \frac{\alpha_s q_c}{N_0} R_t \qquad \ldots (1)$$

$$f_b = \alpha_b \cdot q_c \qquad \ldots (2)$$

where $q_c$ = measured cone resistance
$\alpha_s$ = soil pile factor for skin friction, depending on soil and pile type
$N_0$ = factor for skin friction, depending on soil type
$R_t$ = time factor, depending on time after installation
$\alpha_b$ = soil-pile factor for end bearing, depending on soil time
$q_c$ = average cone resistance above and below tip, for distance 4d above, and 4d below the pile tip.

Equations (1) and (2) were developed from a number of suggested approaches, e.g. Bustamante and Gianeselli (1982), Van Impe (1986) and Belcotec (1985). The factors $\alpha_s$, $\alpha_b$ and $N_0$ are empirical and are based on correlations with load test data. Significant differences exist between recommendations from different sources, but after consideration of the available correlations, those adopted are shown in Table 1. No upper limits were placed on $f_s$ and $f_b$, although in a design situation it would be appropriate to do so.

The "time factor" $R_t$ expresses the effect on pile capacity of the dissipation of excess pore pressures developed during installation. It depends on both the overall increase in shaft resistance and the rate at which this increase occurs. For driven piles, some indication of the overall increase was obtained by comparing the undrained shear strength of the undisturbed clay and of the remoulded clay. The average ratio of these strengths was 2. Furthermore, it was felt that the correlations developed between cone resistance and shaft resistance were relevant to long term conditions, i.e. after complete dissipation of excess pore pressures. Thus, the shaft resistance immediately after driving was taken to be one half of the long term values.

The rate of development of this increase was estimated from theoretical solutions presented by Poulos and Davis (1980) and Randolph et al (1979). Taking an average value of horizontal coefficient of consolidation (from the piezocone data) of 17.5 $m^2$/year, the values of $R_t$ for the driven piles were found to be about 0.75, 0.85 and 1.0 for the three times under consideration.

**Table 1**
**Factors for Determination of Shaft Friction and End Bearing from CPT Data**

| Pile Type | Sand | Clay | |
|---|---|---|---|
| | $\alpha_s$ | $\alpha_s$ | $\alpha_b$ |
| Driven tube (with enlarged shoe) | 0.67 | 0.42 | 1.00 |
| H-pile | 0.90 | 0.59 | 0.77 |
| Slurry pier | 0.86 | 0.65 | 0.30 |
| Cased pier | 0.45 | 0.65 | 0.30 |

NOTES:
1. The above factors relate to long term pile load capacity.
2. For the H-pile the surface area is that of the rectangular prism enclosing the H-section.
3. For driven tube without enlarged shoe, values of $\alpha_s$ are 1.5 times larger than with enlarged shoe.

For the bored piles it was assumed that installation would cause little or no overall change in pore pressure, and hence it was assumed that $R_t$ = 1, i.e. there was no significant time effect on the load capacity of the bored piles.

Predictions of the load-settlement behaviour of each pile were made using an incremental boundary element analysis of pile-soil interaction (Poulos 1979). Elastic continuum theory was used to model the soil behaviour, but at the pile-soil interface it was assumed that the response was hyperbolic. The pile was divided into 11 cylindrical shaft elements and a single pile tip element; for the bored piles an annular element at the diameter discontinuity was also allowed for.

The initial tangent Young's modulus of the soil, $E_{si}$, was correlated to the cone penetration resistance $q_c$ as:

$$E_{si} = \alpha_E \cdot q_c \qquad \ldots (3)$$

where $\alpha_E$ = factor depending on soil and pile type

Based on the author's limited experience, $\alpha_E$ was chosen to be 10 for sand and 30 for clay for the driven piles, and 7 and 21 respectively for the bored piles.

The hyperbolic factor $R_f$ for the pile-soil interface response was taken to be 0.5 for pile shaft elements and 0.9 for the pile tip element, since it has been commonly observed that the pile tip behaviour is markedly more non-linear (and hence corresponds to a higher $R_f$ value) than pile shaft behaviour.

To determine the residual stresses after driving of the steel tube pile and the H-pile, a static load-settlement analysis was carried out to failure (using the estimated short term shaft resistance values), followed by unloading to zero load. Poulos (1987) had indicated that the loads remaining in the pile could give a reasonable estimate of the residual loads in the pile after driving.

No attempt was made to predict pore pressure generation and dissipation around the pile as it was felt that, apart from the initial installation of the driven piles, changes in pore pressure around the piles during test loading would be small.

**PREDICTED TEST PILE BEHAVIOUR**

Table 2 summarises the predicted axial load capacities of the piles at the three specified times after installation.

**Table 2**
**Predicted Pile Load Capacities**

| Pile | Predicted Capacity kN | | |
|---|---|---|---|
| | 2 weeks | 1 month | 1 year |
| Driven steel tube | 846 | 856 | 872 |
| Driven H-pile | 1066 | 1081 | 1104 |
| Slurry pier | 1585 | 1585 | 1585 |
| Cased pier | 860 | 860 | 860 |

The following points are worthy of note:

(1) Most of the load capacity was predicted to derive from shaft resistance in the sand; the shaft and tip resistances in the clay contribute a relatively small proportion of the capacity.

(2) Because of the small contribution of the clay to the load capacity, the effects of time on pile capacity were predicted to be also relatively small.

(3) The slurry pier was predicted to have the highest capacity; a significant component of this came from the end bearing resistance of the "step" at the shaft diameter discontinuity in the sand.

For the driven piles, the residual loads were predicted from an incremental boundary element load-settlement analysis in which installation was simulated by loading the pile to failure and then unloading to zero load. The computed maximum residual load was of the order of 7 to 12% of the long term load capacity.

For the bored piers it was considered unlikely that residual loads would exceed 10% of the pile load capacity, but no attempt was made to predict detailed distributions of residual load.

For all four piles, the predicted load-settlement behaviour was substantially linear to loads well beyond 50% of the predicted ultimate load. At 50% of the ultimate load, the predicted settlements were between 1.5 and 2mm for all piles except the slurry pier, for which a settlement of about 3.5mm was predicted for loading one year after installation.

**COMPARISONS BETWEEN PREDICTED AND MEASURED PERFORMANCE**

A detailed account of the measured performance of the piles is given by Finno et al (1989a) while Finno et al (1989b) summarise the results of all the predictions made. The following outlines the comparisons between the measured performance and that predicted by the author.

**Load Capacity**

Table 3 summarises the predicted and measured axial load capacities for three times after installation. The following observations may be made:

1) For the H-pile, the pipe pile and the slurry pier, the long term capacity predictions are in fair agreement (within $\pm15\%$) with the measurements.

2) For these three piles the shorter-term capacities were overestimated.

3) For the cased pier, the measured capacity was substantially in excess of that predicted by the author.

4) There were very substantial beneficial time effects with both bored piles, contrary to the author's prediction that time effects would not be significant.

5) Conversely, the observed time effects for both the driven piles were less than predicted.

Thus, while three of the predictions of long term load capacity were reasonable, the influence of time effects was quite different to that expected.

A summary of the predictions of all 24 predictors for long term capacity is shown in Table 4. It will be noted that there is a very substantial range of predicted capacities and that while the average prediction for the driven piles was quite good, the capacities of the two bored piles were generally under-predicted by a substantial margin. The author also took a small amount of comfort from the fact that his predictions were generally similar to (and in one case considerably better than) the average of the predictions.

**Load-Settlement**

Figures 2 and 3 compare the predicted long term load-settlement behaviour of the four piles with that predicted by the author. The piles exhibited a relatively linear load-settlement response over a considerable range of load, and the predicted behaviour mirrored this response and also gave an axial stiffness which was close to that measured. It would therefore appear that the combination of the method of analysis and the soil modulus assessed from Equation 3 gave satisfactory settlement predictions at this site. Of course, at loads approaching failure, the predictions become less satisfactory because of the shortcomings of the ultimate capacity predictions.

**Pile Load Distributions**

Figures 4 and 5 show the predicted and measured distributions of load at failure along the pile for the long term test. In terms of the normalised load (load divided by load at the pile head), there is fair agreement between the predicted and measured load distributions for the driven piles, except near the top of the pile. There, the predicted load is less than the measured, indicating that the predicted skin friction is larger than that actually developed.

Table 3
Comparison Between Predicted and Observed Axial Load Capacities

| Pile | Load Capacity kN | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | 2 Weeks | | | 4 Weeks | | | 43 Weeks | | |
| | Pred | Obs[1] | Pred/Obs | Pred | Obs[1] | Pred/Obs | Pred[2] | Obs[1] | Pred/Obs |
| H-Pile | 1066 | 797 | 1.34 | 1081 | 850 | 1.27 | 1104 | 1017 | 1.09 |
| Pipe | 846 | 623 | 1.36 | 856 | 714 | 1.20 | 872 | 1026 | 0.85 |
| Slurry Pier | 1585 | 1155 | 1.37 | 1585 | 1517 | 1.04 | 1585 | 1840 | 0.86 |
| Cased Pier | 860 | 1135 | 0.76 | 860 | 1571 | 0.55 | 860 | 1853 | 0.46 |

NOTES: 1 Observed values are average of values from load cell and jack
2 Predicted values are for 52 weeks

Table 4
Comparison Between Predicted and Observed Long-Term Capacities

| Pile | Load Capacity kN | | | | |
|---|---|---|---|---|---|
| | Range of 24 Predictions | Average | Standard Deviation | Author | Measured |
| H-Pile | 610 to 1670 | 988 | 296 | 1104 | 1017 |
| Pipe | 560 to 1650 | 930 | 280 | 872 | 1026 |
| Slurry Pier | 575 to 2260 | 1007 | 409 | 1585 | 1840 |
| Cased Pier | 580 to 1330 | 911 | 213 | 860 | 1853 |

For the bored piles, the agreement is not particularly good, with the predicted load being greater than the measured, indicating that the predicted skin friction was less than that actually developed. The difference is particularly marked for the cased pier, for which the prediction of ultimate load was very conservative (see Table 3). The difficulty of accurately predicting the detailed performance of a pile is clearly demonstrated by these comparisons.

**Residual Loads**

Figures 6 and 7 compare the distributions of predicted and measured residual loads after driving in the driven steel piles. The predictions suggest relatively small compressive loads along each pile, whereas the measurements indicate tension along some or all of the pile length. However, the accuracy of these residual load measurements may be questionable (Finno et al, 1989a). Nevertheless, both the measurements and predictions indicate that the residual loads after driving are relatively small compared to the ultimate pile load capacity.

Figures 6 and 7 also show the residual loads in the piles after the completion of loading. These distributions agree more closely with the predicted residual loads, and it should be noted that the method of prediction used is in fact more relevant to the after-loading case, as it simulates loading to failure, followed by unloading to zero load.

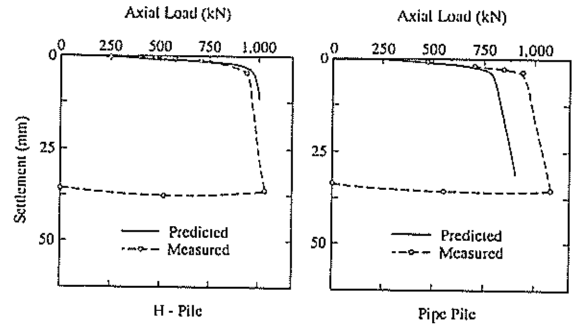


FIG.2 PREDICTED AND OBSERVED LOAD-SETTLEMENT OF DRIVEN PILES

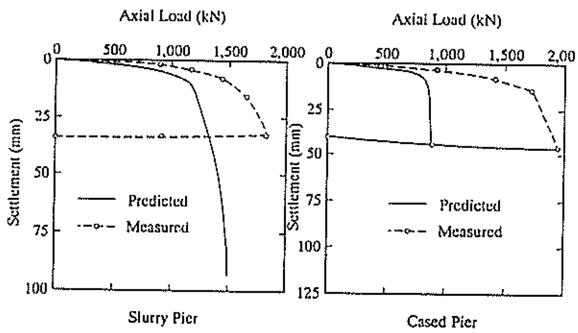


FIG.3 PREDICTED AND OBSERVED LOAD-SETTLEMENT OF BORED PILES

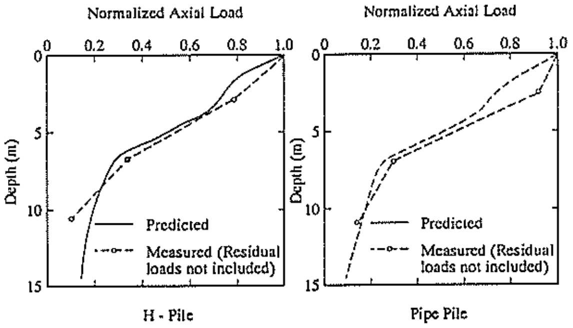


FIG.4 PREDICTED AND OBSERVED LOAD DISTRIBUTIONS AT FAILURE (Long Term)

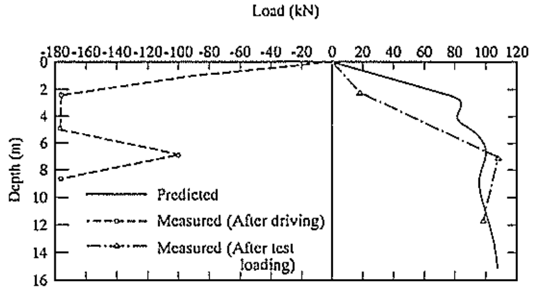


FIG.6 PREDICTED RESIDUAL LOAD DISTRIBUTION FOR DRIVEN STEEL TUBE PILE

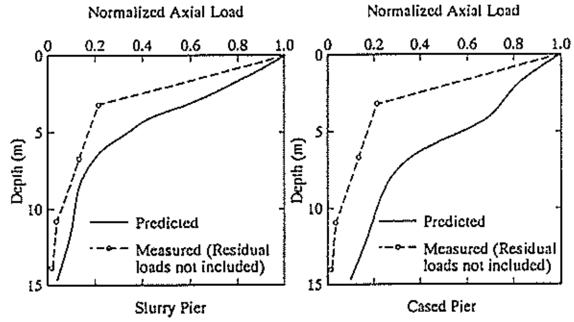


FIG.5 PREDICTED AND MEASURED LOAD DISTRIBUTIONS AT FAILURE (Long-Term)

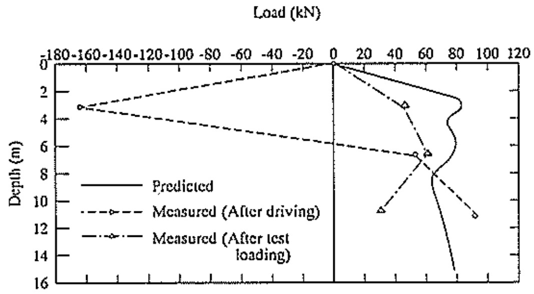


FIG.7 PREDICTED RESIDUAL LOAD DISTRIBUTION FOR DRIVEN H-PILE

## CONCLUSIONS

The circumstances of this prediction exercise were more favourable than could be expected in most practical problems. There was a good definition of the subsoil profile, an abundance of geotechnical data, and loading that was well defined. Despite these favourable circumstances and the relatively straightforward nature of the problem, the Class A predictions of pile behaviour were fair only. For the driven piles, the ultimate load capacity, load-settlement behaviour and the load distribution along the piles were relatively well predicted. However, for the bored piles, the predictions of load capacity were conservative, particularly for the cased pier. For these piles, the time effects were significant, contrary to the predictions, with the load capacity increasing substantially with time. It is interesting to note that for the cased pile, all 24 predictors under-predicted the long term load capacity. A variety of prediction methods was used by the predictors, and it seems clear that the method used for prediction is likely to be of less significance than the assessment of the parameters to be input into the analysis. Certainly, the author does not believe that his predictions would have been improved by the use of more sophisticated techniques of analysis.

The difficulty of predicting both time effects and residual loads in these tests suggests that there is scope for more research aimed at a better understanding of installation and pore pressure effects on pile capacity and response.

## ACKNOWLEDGMENTS

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