Back analysis of test piles driven into estuarine sands

Analyse rétrospective des pieux d'essai pilotés dans des sables d'estuaire

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SYNOPSIS

Precast concrete piles were driven at Richards Bay, to support a large bridge. The piles were driven through surface clays to found in a medium dense and dense estuarine sand. Back analysis of preliminary pile tests showed that both static and dynamic computer models gave good indications of production pile behaviour. The availability of this information during construction proved invaluable for controlling installation of production piles.

INTRODUCTION

This paper describes a case history where piled foundations have been used to support part of a road bridge in Richards Bay Harbour, South Africa. The piles are precast concrete sections, driven into sands where they carry most of the working load in side friction. In particular this paper addresses the prediction of pile capacity from driving performance and in situ testing. The project was approached on a design and construct basis. The authors formed part of the client team and the analyses described here were carried out to check the design and ensure satisfactory performance. In addition as future development is expected in this area, the back analyses described not only provide valuable information regarding the behaviour of the soils in general, but will be directly applicable to future projects.

The proposed bridge is 850m long and crosses a canal and railway tracks. It comprises 20m long simply supported bridge decks and typical pier loads are 15000kN. The structure is designed to be settlement tolerant.

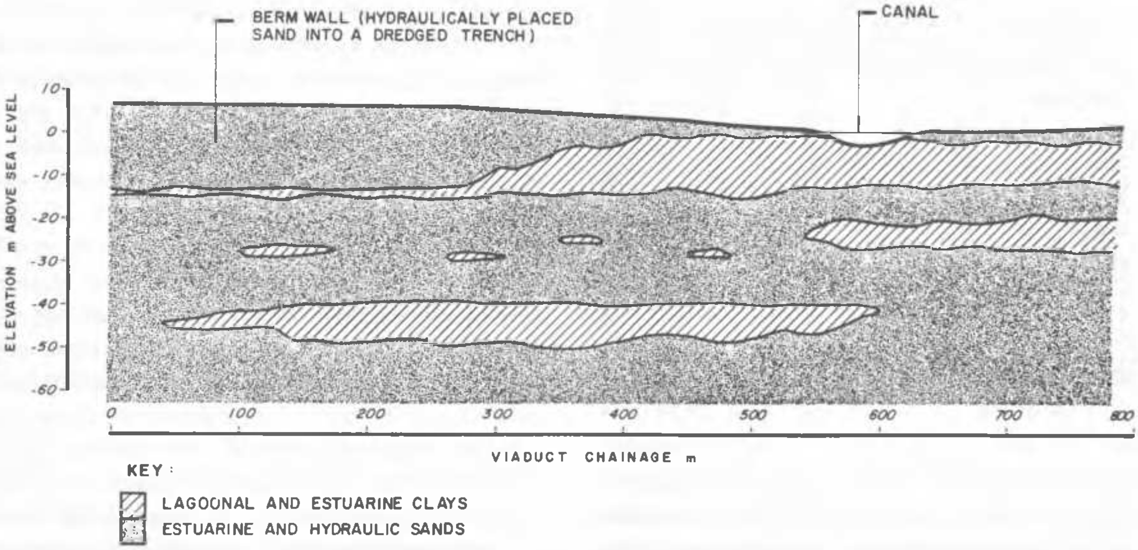

FIGURE 1 IDEALISED SOIL STRATIGRAPHY ALONG VIADUCT CENTRE LINE

INVESTIGATION, SITE GEOLOGY AND FOUNDATION CONCEPT.

The investigation was carried out using both rotary drilling techniques and piezometer cone-penetration tests. This showed very soft and soft lagoonal clays to 13,5m depth over medium dense and dense estuarine sands which became very dense with depth. The estuarine sands contain inter bedded clay horizons. Below 40m, a stiff lagoonal clay was encountered over half of the site. These soils were generally proved to 50m depth and in one borehole to 75m depth indicating that part or all of the viaduct is located over a deeply incised channel in the Cretaceous mudrocks that underlie the harbour area. The soil stratigraphy indicated by this investigation is shown in Figure 1.

Average geotechnical properties for the clays and sands at the site are presented in Table I & II respectively.

CLAY HORIZON	C_v m^2/yr	Cc	Pc/Po	Average e_o	Average γd kg/m^3
Upper Lagoonal	0,36 - 3,4	0,75	1,0	2,3	8,0
Middle Estuarine	3,5	0,4	-	-	-
Lower Lagoonal	1 - 5	0,5*	1,4	1,13	1240

TABLE 1: ENGINEERING PARAMETERS FOR CLAY HORIZONS

SAND HORIZON	Av SPT "N"	Av qc (MPa)	Ø	E (MPa)
Hydraulic Fill	10 - 25	5 - 10	30° - 36°	10 - 20
Estuarine sand				
15 - 20m	15	5	32°	15
20 - 25m	26	11	35°	22
25 - 30m	38	20+	37°	30 - 35
30 - 40m	44	20+	38°	30 - 25
40 - 60m	67	-	38°	45

TABLE 2: IN SITU TESTING RESULTS AND ENGINEERING PARAMETERS FOR SAND HORIZONS

On the basis of the stratigraphy shown in Figure 1 it was recommended that part of the viaduct be founded on friction piles driven into the estuarine sands. The problems foreseen for piling were as follows:

- downdrag forces on piles as a result of consolidation of the near surface clays due to recently placed fill.
- variable SPT "N" values in the estuarine sands indicating the presence of medium dense and very dense pockets in a generally dense soil horizon;
- the presence of a compressible lower lagoonal clay horizon.

The bridge piled foundations were put to tender on a "design and construct" basis. The design selected from tender submissions comprised the use of 350mm square precast concrete piles driven to a set in the dense estuarine sands at depths ranging from 30m to 38m. The piles were raked outwards at 1 in 6 to avoid concentrations of load into the underlying compressible clay horizon.

QUALITY ASSURANCE DURING PILE DRIVING

The selection of suitable driving control criteria for predominantly friction piles posed a problem due to the variation in density in the sand founding horizon. A conservative approach using a small set would result in many piles being overdriven into the underlying compressible clay horizon. Alternatively too large a set would result in reduced pile capacity, requiring more piles and increased piling costs.

Two preliminary test piles were driven to criteria that included a set and minimum driving resistance over the final 2 metres. The pile test results were satisfactory and these driving criteria and the results of borehole investigations at each pier were used as a basis for control of production piles. Despite these measures a small number of piles did not achieve the criteria before reaching the maximum allowable depth imposed by the underlying clay horizon. A pile test was carried out to failure on one of these piles.

It was realised that a rapid method of correlating the driving resistance of piles to their load carrying capacity was needed. Back analysis of the behaviour of the test piles under both dynamic and static conditions enabled calibration of both wave equation and static axial load transfer computer models. Correlation of the input data for these models with piezometer cone penetration resistance values and pile driving records has increased the confidence with which pile behaviour can be predicted at the bridge site and at other locations.

COMPUTER MODELLING OF STATIC AXIAL LOAD RESPONSE

The axial load transfer from the piles to the surrounding soils was modelled using a modified version of the programme described by Bowles (1974). The pile is represented using a finite element model. The deformation behaviour of the pile, pile-soil shear transfer and end bearing resistance are required as input parameters. The load response curve for the pile can then be generated by performing the analysis for a number of different loading conditions.

The selection of representative input parameters for the programme from results of in situ testing requires some discussion. Many correlations of side friction and cone resistance, SPT 'N' value and laboratory test results are published. Some of these are briefly discussed below:

Correlations with cone resistance

The peak skin friction that is mobilized along a pile shaft segment (f_s) is usually related to cone resistance (q_c) by a relationship of the following form:

$$fs = q_c/a \qquad \text{where } a = \text{constant} \tag{1}$$

Published values for "a" vary between 20 and 400, depending on the nature of the soil and compressive or tensile pile loads (Beringen et al, Meyerhof, Tong et al, Mohan et al)

Correlations with SPT 'N' values

The SPT 'N' value is usually related to peak skin friction by a relationship of the form

$$f_s = N/b \qquad \text{where } b = \text{constant} \tag{2}$$

Published values for b are usually of the order of 50 to 60 (Meyerhof).

Although many correlations of side friction with laboratory test results are published, these were not used in the analysis described here.

Modelling of pile load settlement behaviour requires a knowledge of the mobilization of skin friction with pile movement. Vijayvergiva has published the following correlation for driven and bored piles in both clay and sand (Beringen, et al)

$$f = fmax\ (2\sqrt{Z/Zc} - Z/Zc) \tag{3}$$

where f = unit friction mobilized along a pile segment at movement Z.
fmax = peak unit friction
Zc = critical movement of a pile segment at which fmax is mobilised.

Berigen shows that the value of Zc is difficult to define, but in general appears to be in the range 5 to 13mm.

In the analysis described here, the mobilization of end bearing is modelled by a linear function. It was assumed that the maximum unit end bearing is mobilized over a distance of 5% to 7% of the equivalent pile diameter. The maximum end bearing was estimated using two methods as follows:

i) $$q_l = 50\ Nq \tan \phi \tag{4}$$

where q_l is the limiting effective stress at pile point at failure (after Meyerhof)

ii) q_u = 2.5N for silts (5a)
4N for sands (5b)
6N for gravels (5c)

where qu = ultimate point resistance
N = SPT 'N' value

BACK ANALSIS OF PILE TEST RESULTS

The results of the pile tests performed before production piling started were used to back analyse the static load transfer at the site and the dynamic driving characteristics of the piles.

STATIC LOAD TRANSFER

Of the in situ testing carried out at the site, the electrical piezometer cone penetration testing gives the highest quality results. The equipment is described by Jones (1982), and the test provides a continuous record of cone resistance and pore water pressure response with depth thus enabling accurate determination of boundaries between different soil horizons. Where possible therefore, results of electrical cone testing were used to estimate peak skin friction values. This was not possible in the dense sand at the base of the piles as the cone refused on this horizon. In these areas, qc values were estimated by comparing the SPT values for the upper and lower horizons.

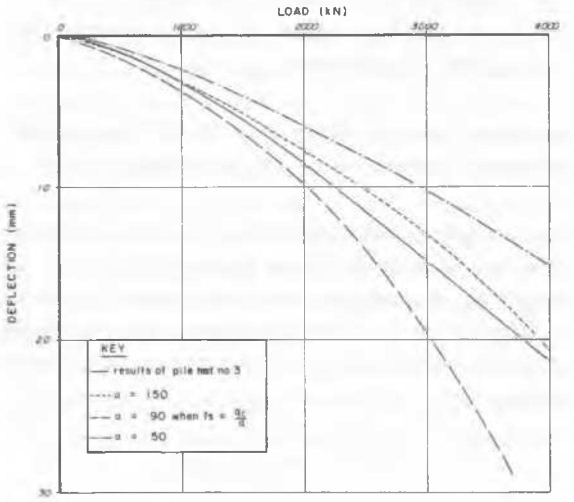

FIGURE 2: COMPARISON OF PREDICTED AND MEASURED LOAD SETTLEMENT CURVES

Analyses were carried out using different values of the constant "a" (Eqn 1) and an average value for the estimated point resistance. Results of these analyses carried out for the soil profile recorded at Test Pile 3 are presented in Figure 2. The sensitivity of the results to assumed point resistance is shown in Figure 3. In carrying out the above analysis, it was assumed

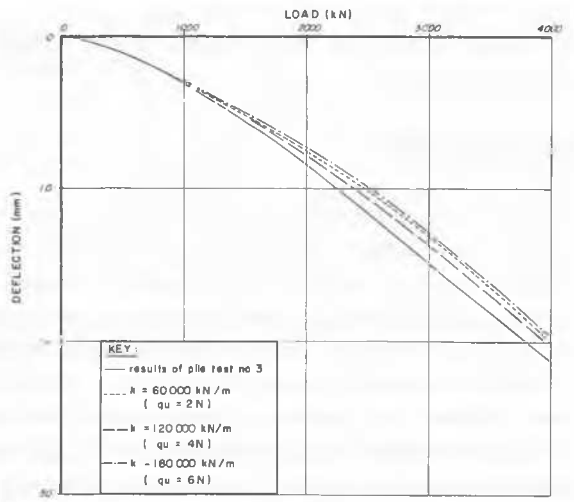

FIGURE 3: SENSITIVITY OF MODEL OUTPUT TO PILE POINT SPRING CONSTANT

that skin friction is mobilized over a distance of 8mm. The sensitivity of the results to this assumption is examined in Figure 4.

These analyses show that the peak skin friction could be estimated using the relationship

$$f_s = q_c / 90$$

DYNAMIC ANALYSIS

Following back analysis of the static load test results, dynamic analysis were carried out using the wave equation method.

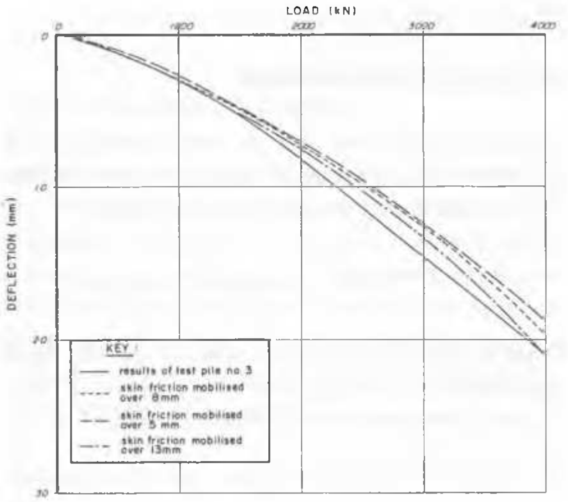

FIGURE 4: SENSITIVITY OF MODEL OUTPUT TO MOBILIZATION DISTANCE FOR SKIN FRICTION

The computer programme used is described by Wiseman and Zeitlin (1983). The soil/pile friction and end bearing parameters from the static back analysis were used as initial input parameters. The ultimate end bearing value was then adjusted to obtain the measured set in the dynamic analysis output. Further analyses were carried out using reduced friction values to estimate the pile capacity for different values of set. In carrying out these analyses, it was assumed that the ultimate end bearing capacity of the pile remains constant after driving whereas the friction values increase due to "freeze". Analyses were also carried out on the same basis using increased and decreased end bearing values. Results of these analysis are presented in Figure 5.

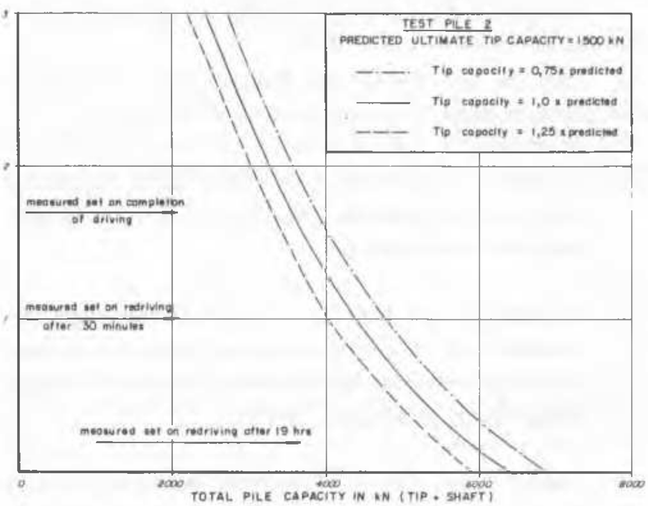

FIGURE 5: SENSITIVITY OF WAVE EQUATION ANALYSIS TO TIP CAPACITY

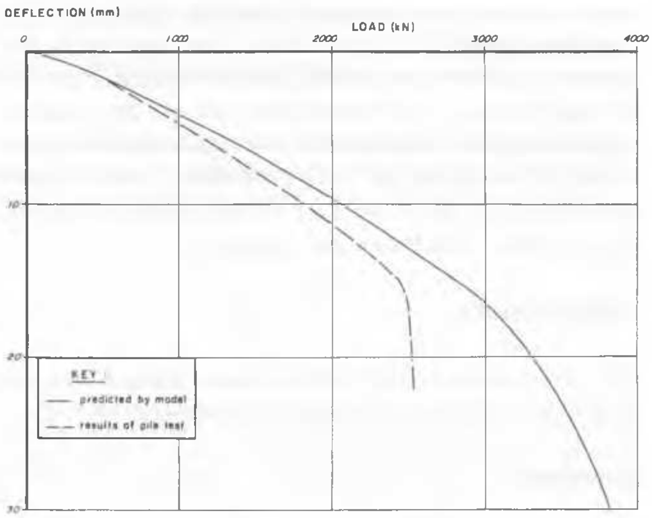

FIGURE 6: RESULTS OF PILE TEST CONDUCTED ON "SOFT BOTTOM" PILE

Test pile no 2 was redriven at various times after initial driving. The improvement of pile capacity with time as predicted by the wave equation method on the basis of observed sets is also shown on Figure 5.

COMPARISON OF PREDICTIONS AND PERFORMANCE

Some piles at various locations across the site did not achieved the desired set. Driving was however stopped to prevent the piles from founding too close to the underlying clay. A third pile test was therefore scheduled on one of these "soft bottom" piles to check the load carrying capacity. A static analysis was carried out using input information from a nearby cone penetration test. The looser sand pocket was not present at this test site, however, and the static analysis was expected to overpredict the pile capacity. The predicted and measured load deflection curves are presented in Figure 6. Dynamic analyses were then performed and a set/capacity curve generated as described above. The pile load capacity at completion of driving was estimated from this curve on the basis of the measured set. This was then uprated to take account of freeze by the same ratio observed for Test Pile 2. The actual pile failure load was found to be in agreement with that predicted by this method to within 10%.

CONCLUSIONS

A case history has been described in which precast piles were driven into estuarine sands. Back analyses of preliminary pile test results has shown that both the static and dynamic analyses computer models give a good indication of pile behaviour when calibrated using pile test information. The availability of this information during construction provided invaluable pile driving control data.

Back analysis of the static load testing of the piles indicates that the peak skin friction achieved may be estimated from cone resistance as follows

$$f_s = q_c/90$$

The analysis is not sensitive to the distance over which the peak friction is mobilised. It was found that the piles tested in this project carry load predominantly by shaft friction. The prediction of pile performance is therefore not very sensitive to the assumed mobilization of end resistance.

Pile capacity can be predicted within reasonable accuracy on the basis of the driving record using the wave equation method. The programme used in this project is suitable for adaption to microcomputers and can therefore be easily available on site. Although the correlation between predicted and measured pile capacity agreed to within 10% here, this is considered to be good fortune rather than sophistication. On the basis of published information, correlations to within more than 30% cannot be consistently expected.

Comparison of calculated and observed pile behaviour have been made for 3 test piles. The good agreement achieved in these comparisons confirms the applicability of the models. The interactive use of the computer models and site measurements for individual piles at construction stage not only provides a rational basis for decisions regarding pile driving control criteria, but may offer significant cost savings.

ACKNOWLEDGEMENTS

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