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PRESS-IN PILING: THE INSTALLATION OF INSTRUMENTED STEEL TUBULAR PILES WITH AND WITHOUT DRIVING SHOES

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The results of three pile installation tests carried out in Japan using the Press-in Method are presented. An instrumented double-skin tubular steel pile allowed independent measurement of internal and external shaft friction and base resistance. The pile was installed with and without internal and external driving shoes. The consequent reduction in shaft friction is examined.

keywords: press-in method, tubular pile, instrumented, driving shoe, shaft friction, temperature effects

1 Introduction

1.1 Press-in pile driving

The technique of press-in pile driving makes use of hydraulic rams to provide the force necessary to jack a pre-formed pile into the ground. The hydraulic rams form part of a robotic machine known as a 'Silent Piler' which uses previously jacked piles to provide a reaction force (figure 1). This technique of pile installation is known as the 'press-in' method.

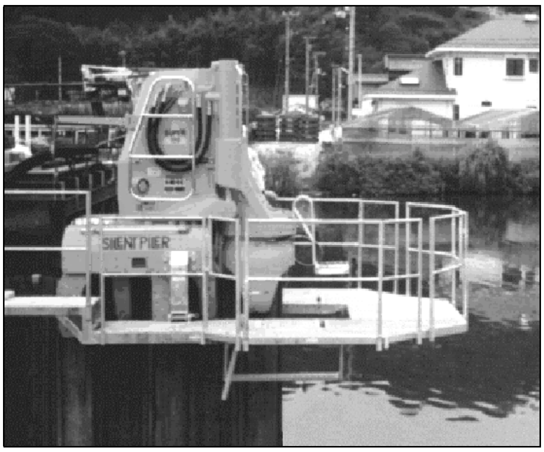

Figure 1; Silent piler with safety walkway

The technique's main advantages over conventional methods of pile installation are a reduction in local disturbance by ground-borne vibrations, a sharp reduction in noise pollution (table 1) and the ability to install piles in confined space.

Plant Type	Equivalent sound power level at 10m range (dB)
Pneumatic drill	100-110
Diesel hammers	102-109
Hydraulic impact hammers	90-97
Vibrodrivers	80-97
Giken silent piler	55
Street corner traffic	70-80
Business office	50-60

Table 1; Noise levels induced by piling [1]

1.2 Prediction and relief of hard driving

A 'Silent Piler' operator must select the most suitable machine size for a particular job in order that the hydraulic rams do not reach their force limit before the pile has been jacked to the required depth. Currently, the most powerful 'Silent Piler' has a maximum force capacity of 400 tonnes.

If unexpected resistance is met when driving with an undersized pile hammer or vibrator, penetration will be slowed, but refusal is rare. In contrast, when using the press-in method, if the unexpected resistance exceeds the force capacity of the piler, further penetration by press-in alone is impossible; the use of a driving aid is required. A corollary to this point is that in difficult soil conditions, jacking does not generate the high driving stresses and possible pile damage associated with conventional methods. Pile sections can be selected based on their design loading rather than a driveability criteria.

The prediction of jacking force is, therefore, essential to the successful application of the press-in method. Any driving aid which can provide a reduction in jacking load without compromising the performance of the installed foundation or retaining wall is highly desirable.

2 Techniques for the relief of hard driving

There are a number of techniques which can be used to reduce the jacking force required to install a pile using the press-in method. These include:

i) The use of driving shoes
ii) Water jetting during installation
iii) Pre-augering to weaken the ground
iv) Cycling the pile up and down during installation ('surging')

Water-jetting, pre-augering, and surging have all been successfully used during the installation of sheet piles in retaining wall applications throughout Japan. In these applications the design loading is primarily lateral, so any reduction in jacking force (or axial capacity) does not affect the ability of the piles to satisfy the design criteria. However, water-jetting and pre-augering require significant additional equipment, slow the installation process, and their influence on the surrounding ground is not well understood. Jetting or augering may not be permitted close to an existing structure. The use of driving shoes is preferable since it does not require additional equipment and is less intrusive.

The mechanism by which driving shoes can reduce jacking force is hypothesised in figure 2. The altered flow of soil around the pile tip leads to a reduced horizontal effective stress acting on the pile shaft (C), and a corresponding reduction in shaft friction. This may lead to a significant reduction in jacking force since shaft friction represents a large proportion of the resistance. The earth pressure coefficient, K, within a soil plug created without the use of driving shoes is typically 0.6-1.0 [2]. Since K_a is typically 0.25-0.3, a driving shoe might be expected to reduce internal shaft friction by a factor of 2-4.

This paper describes a series of field tests which investigated the influence of both internal and external driving shoes on the jacking force required to install a tubular pile. An assessment is made of the ability of current design methods to predict the observed behaviour.

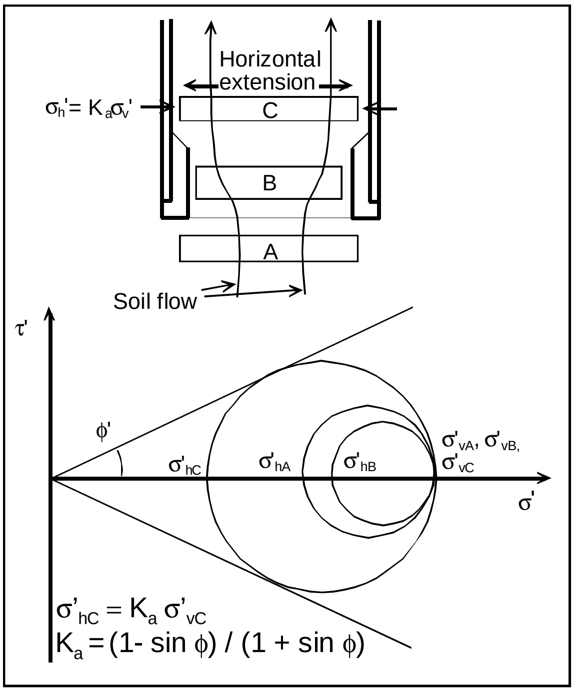

Figure 2; Hypothesised stress changes around a driving shoe [3]

3 Fieldwork

3.1 Site description

An instrumented tubular test pile was installed at the Takasu Research Centre in Kochi on the island of Shikoku, Japan. The stratigraphy consists of made ground overlying silt and sand. The results of laboratory testing of each stratum are shown in table 2. Relative density (I_D) has been deduced from CPT data following Jamilkowski *et al* [4].

Stratum	ϕ_{crit}	D_{10} (mm)	D_{10} (mm)	I_D
A:Made ground	-		-	-
B: Silt	39°		-	40%-
C:Sand & gravel	44°	0.22	1	80%
D: Silty sand	39°	0.047	0.18	30%
E: Sand	42°	0.18	0.3	70%

Table 2; Soil properties at test site

CPT data, q_c is shown in figure 3. In order to allow the CPT data to be used in the prediction of pile resistance, a smoothed CPT profile, $q_{c(+/-0.5m)}$, has been obtained by averaging the data over the neighbouring +/- 0.5m zones in a manner which reflects the differing zones of influence of a small CPT and a large tubular pile. This is similar to the averaging procedures proposed by others ([5],[6],[7]). Heavy rain maintained the water table at ground level throughout the test period. The 1.2m surface layer of made ground was excavated at each test location prior to pile installation.

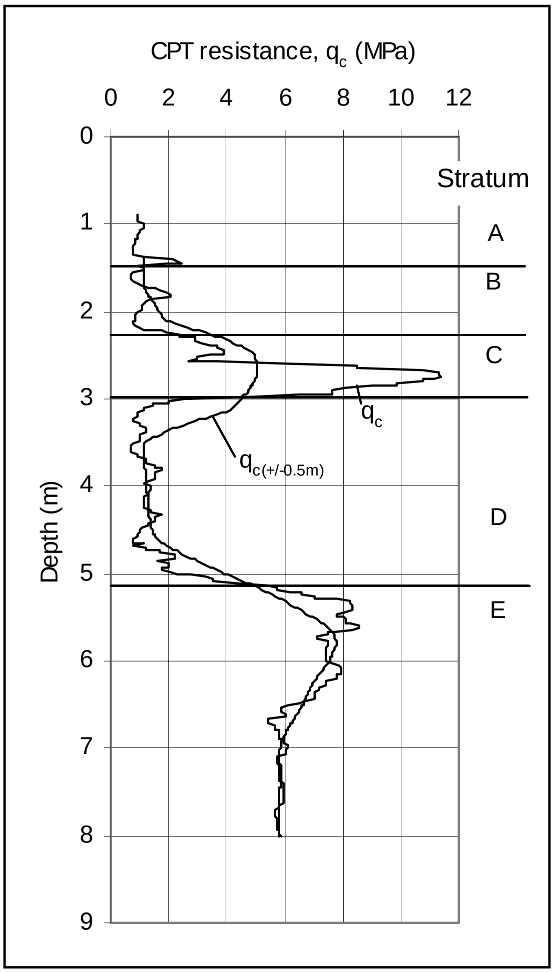

Figure 3; CPT profile and stratigraphy

3.2 Pile and piler specifications

The test pile was constructed from two concentric steel tubes connected at the base. The pile dimensions are shown in table 3. A specially modified press-in piler was used to install the test pile. Continuous calculation of the jacking load was achieved by measurement of the oil pressure in the hydraulic rams. The pile tip depth was measured using a displacement potentiometer and the plug length was measured using a weighted follower within the pile. The piles were installed in a series of 700mm strokes at a rate of approximately 50mm/s

	D_{outer} (mm)	Wall thickness t (mm)	A_{wall} (mm^2)	Length (m)
Outer	318.5	10.3	9973	10
Inner	267.4	6.6	5408	8

Table 3; Pile dimensions

3.3 Instrumented pile

Figure 4 shows the dimensions of the internal and external driving shoes. The test pile was instrumented with strain gauges to allow measurement of the axial and hoop strains on both the inner and outer sleeves along the full length of the pile. 7 heights were selected and at each height an axial and hoop gauge was positioned on both the inner and outer sleeves at two locations on opposite sides of the pile (figure 5).

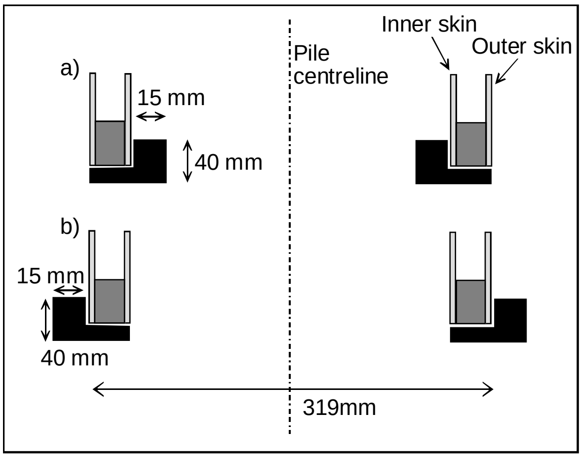

Figure 4; Driving shoe dimensions

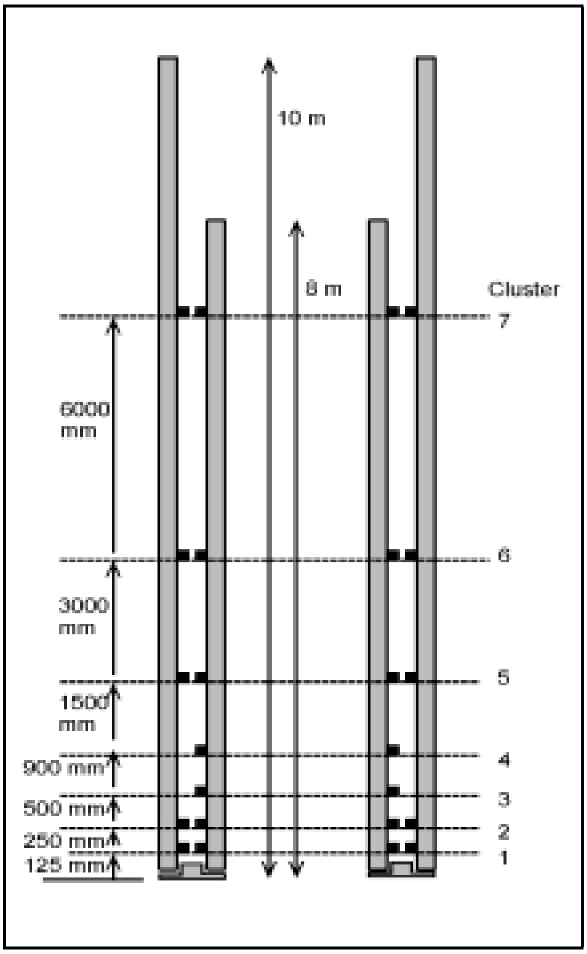

Figure 5; Strain gauge locations

The gauges were within the annular gap between the inner and outer sleeves to provide protection from both soil abrasion and groundwater.

Since independent strains are induced in both the axial and hoop directions, a full bridge measuring strains and Poisson strains on the pile wall with a consequent lack of temperature sensitivity could not be utilised. Two possible methods were envisaged for removing the temperature-induced apparent strains which arise when inserting a sun-heated pile into the cold soil:

i) Pairing the active gauge with a dummy gauge mounted on small unstrained sheet of steel welded to the pile wall to create a half bridge;
ii) Measuring the pile temperature using thermistors mounted on the pile wall and applying a manual correction to the strain gauge data.

Neither method proved to be satisfactory in removing inconsistencies found in the data due to temperature effects. Instead temperature effects were finally overcome by pausing between strokes for 5-10 min to allow temperature gradients in the pile wall to become negligable. Strain changes outside of the stroke duration were assumed to be due to temperature drift and were neglected. Since the loads measured using strain gauges agreed closely with jacking force data, this method of compensation was considered satisfactory.

4 Measurement of load distribution

4.1 Calculation of stress

The axial and hoop strain gauge data was converted to axial and hoop stress using the elastic properties of steel, assuming plane stress conditions. The calculated stresses on either side of the pile were combined to give average stresses throughout each cross-section. This removed the influence of bending moments in the pile.

4.2 Calculation of load distribution

The data from the strain gauges nearest the base of the pile was used to determine the distribution of load between base resistance and inner and outer skin friction. The following assumptions were made.

i) External shaft friction, Q_{so}, is the difference between the jacking force and the compressive force at the bottom of the outer sleeve.
ii) Internal shaft friction, Q_{si}, is equal to the tensile force measured at the bottom of the inner sleeve.
iii) Base resistance, Q_w, is the difference between the stresses measured at the bases of the inner and outer sleeves.

Since the lowest strain gauges were located 125mm (~2/5 of a pile diameter) above the pile base, a small amount of internal and external shaft friction was attributed to base resistance.

5 Analysis of pile tests

The results of the three pile tests are presented and analysed below.

Test 1: No shoe
Test 2: External driving shoe (figure 4b)
Test 3: Internal driving shoe (figure 4a)

Figures 6 and 7 show profiles of jacking force and plug length against embedded depth

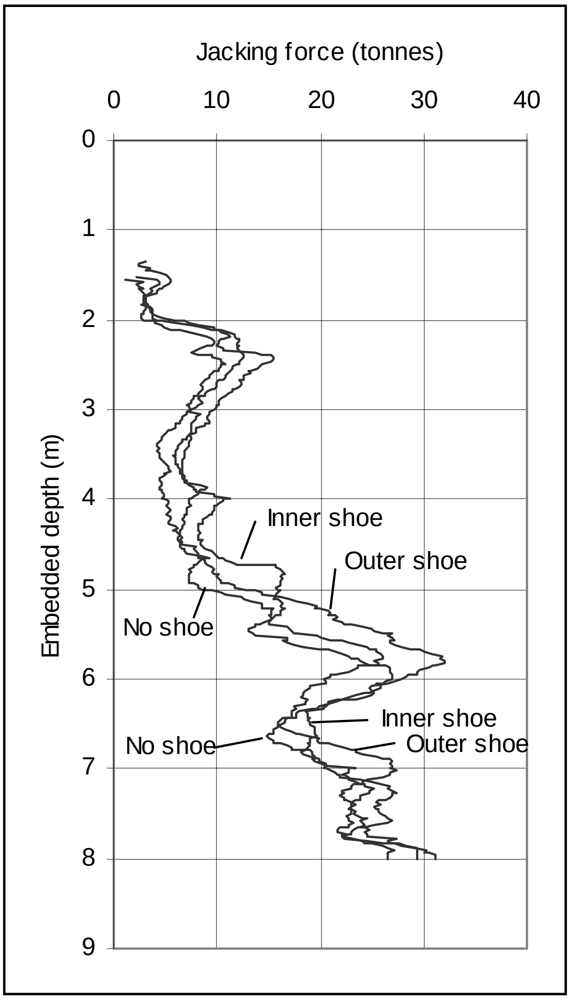

Figure 6; Jacking force vs Embedded depth

Figure 7 allows the mode of penetration (plugged or unplugged) to be deduced. Distinct transitions from unplugged to plugged penetration are evident, and there is no tendency for the pile to penetrate in a partially plugged manner.

The degree of plugging is conventionally analysed by defining the incremental filling ratio (IFR). This is defined as the increase in plug length divided by the increase in embedded depth over a small increment of penetration. Dynamically-installed piles usually exhibit an

IFR, throughout installation, of 0.4-1.0 (eg [8], [9]), indicating partial plugging. In contrast, figure 6 shows that jacked piles alternate between the fully plugged (IFR = 0) and unplugged (IFR = 1) modes of penetration.

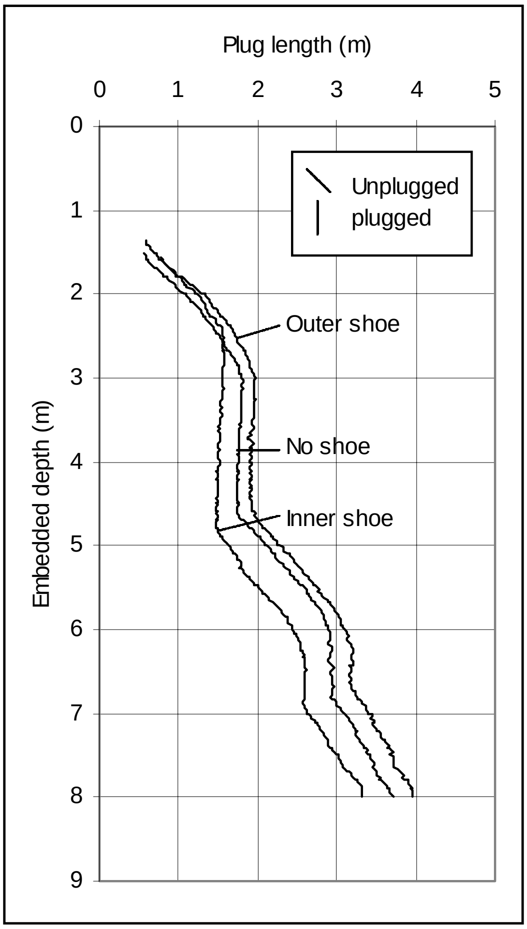

Figure 7; Plug length vs Embedded depth

By considering the static equilibrium of plugged and unplugged penetration (see figure 8) the following equations are derived [3];

$$Q_{unplugged} = Q_{so} + Q_{si} + Q_w \quad (1) \text{ (figure 8a)}$$
$$Q_{plugged} = Q_{so} + Q_w + Q_p - W_p \quad (2) \text{ (figure 8b)}$$

The lower bound of $Q_{unplugged}$ and $Q_{plugged}$ will govern the jacking force and hence the mode of penetration. Plugging will occur if $Q_{si} + W_p > Q_p$. This is illustrated on figure 7 by the transition to plugged penetration at 2.5m depth on entering a loose silty sand which is associated with a drop in base resistance. Penetration reverts to an unplugged manner on reaching the dense sand layer (hence high base resistance) at 5m depth.

5.1 Measured base resistance on pile wall

Figure 9 shows the variation of base resistance acting on the annulus of pile wall (Q_w) with embedded depth. The general trend matches the CPT profile and the pile with an outer shoe experiences the greatest base resistance due to the large cross-sectional area.

To isolate the influence of base area, Q_w is converted to unit base resistance, q_b by dividing by the cross-sectional area of the pile (and shoe, where applicable. See table 4). In figure 10, q_b is normalised with respect to the smoothed CPT resistance ($q_{c(+/-0.5m)}$). This shows a general trend of $q_b/q_{c(+/-0.5m)}$ decreasing with depth from 0.8 to 0.4.

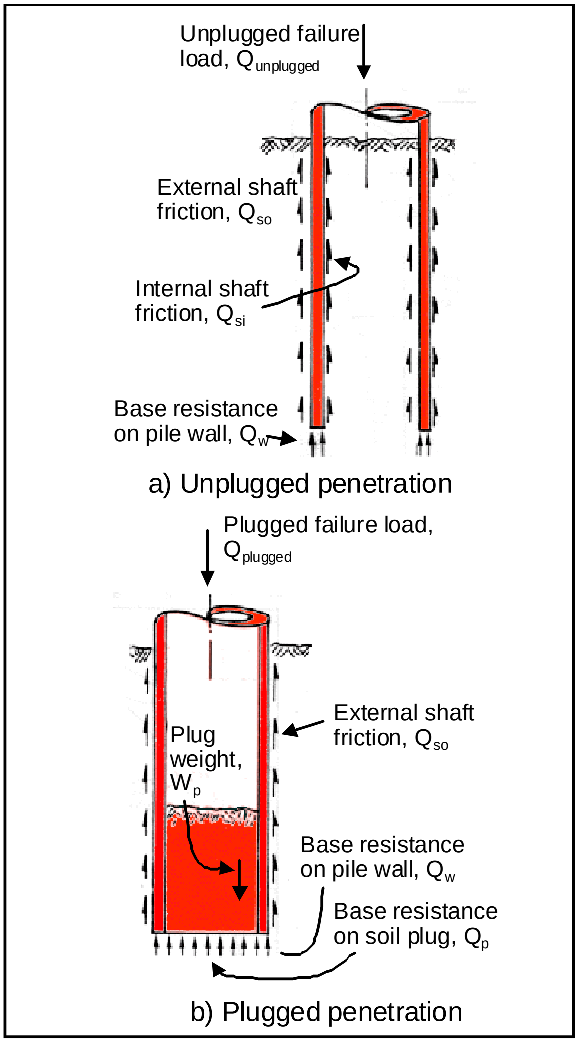

Figure 8; Plugged and unplugged modes of penetration

Between 3.5m to 5m depth a sharp increase in $q_b/q_{c(+/-0.5m)}$ is evident. This is a region of soft soil with a corresponding decrease in q_c. q_b is less influenced by this zone of soft soil due to the relative sizes of a CPT and the pile, leading to an anomalous increase in $q_b/q_{c(+/-0.5m)}$. An alternative explanation is that the small contribution of internal and external shaft friction acting on the 125mm length of pile below the lowest strain gauges leads to an overestimate of q_b which is most noticeable in the zone of low base resistance.

Pile tip geometry	Base cross-sectional area (mm^2)
No shoe	28922
Inner shoe	40194
Outer shoe	44638

Table 4; Area of driving shoes

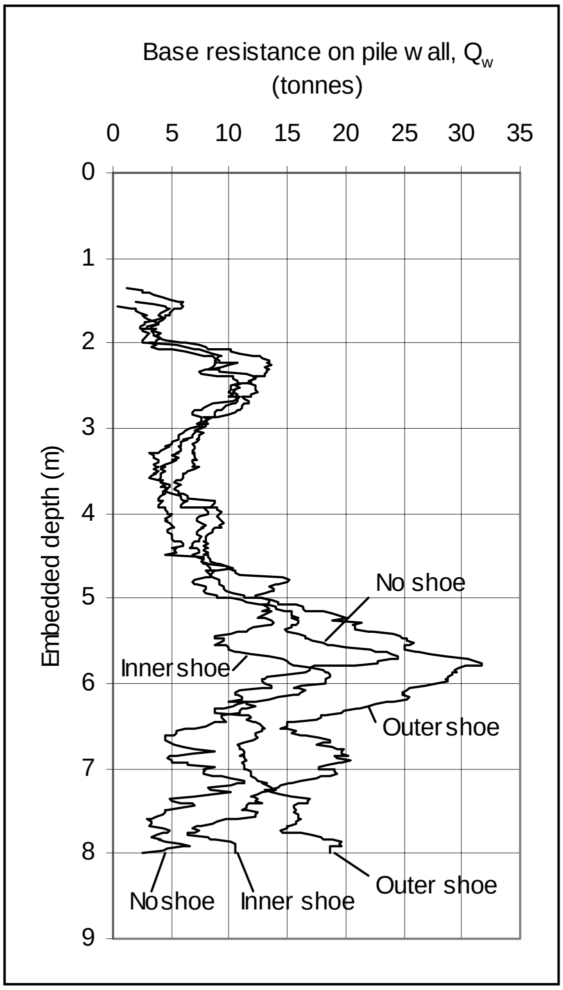

Figure 9; Base resistance on pile wall, Q_w vs Embedded depth

This scale effect between CPT resistance and pile base resistance is of importance to designers, and has been widely observed in the literature (eg. [10], [11]). A number of empirical methods exist for the selection of a reduction factor, α (equation 3). However, some design methods continue to recommend using $q_b = q_c$ for prediction of pile capacity (eg. [12], [13]), which is shown by this data to be unconservative.

Other authors have proposed empirical methods for selecting α in design through curve-fitting to field or centrifuge test data. The observed trend of α varying from 0.8 at the ground surface to 0.4 at 8m depth compares to a trend of 0.7 to 0.5 predicted by de Nicola [11]. Jardine and Chow [5] recommend a reduction factor of 0.36 based on pile diameter alone. It should be noted that this correlation is for a head displacement of D/10 instead of ultimate (or 'plunging') failure and so an underestimate is to be expected. The de Nicola method links the decrease in q_b/q_c to an increase in confining stress (or depth) whereas pile diameter is the only variable in the Jardine & Chow method. Borghi *et al* [14] propose that the scale effect is due to interaction between the shaft and the base, varying with both depth and diameter.

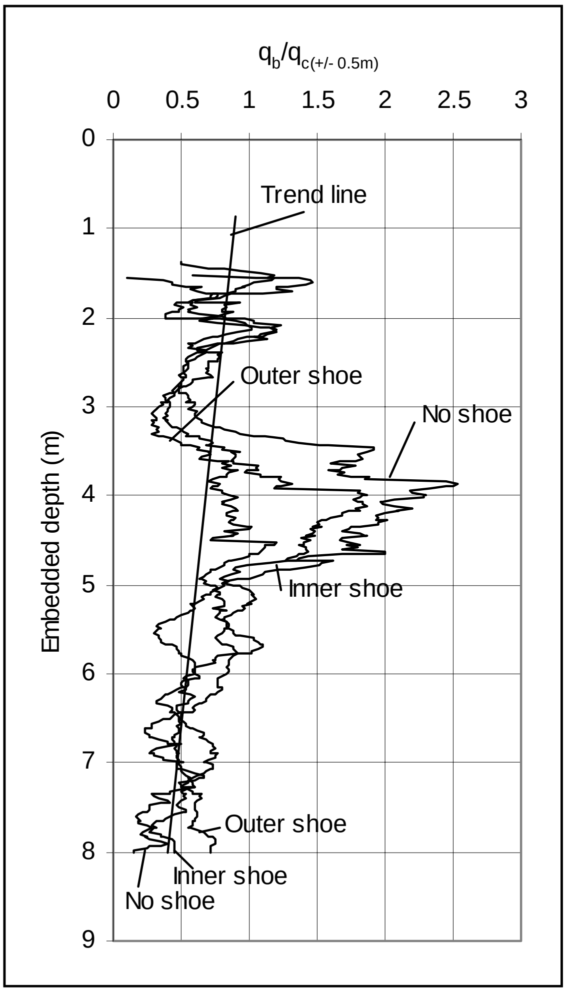

Figure 10; Normalised unit base resistance ($q_b/q_{c(+/-0.5m)}$) vs Embedded depth

$$q_b = \alpha\, q_c \qquad (3)$$

5.2 Measured external shaft friction

The total external shaft friction, Q_{so}, plotted against depth is shown in figure 11. It is dependent on the external embedded surface area, the angle of friction between the pile and the soil and the effective horizontal stress acting on the pile. Both the area in contact with the soil and the angle of friction will be constant for a given embedded depth so any variation in outer friction between tests will be due to differences in the effective horizontal stress acting on the pile.

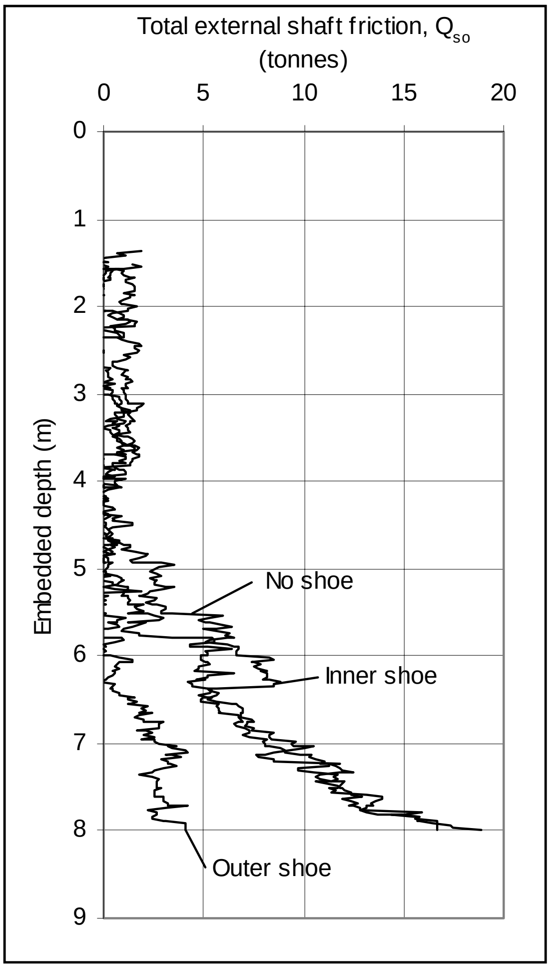

Figure 11; Total external shaft friction vs embedded depth

It can be seen from figure 11 that the external driving shoe leads to a significant reduction in external shaft friction especially at large depths. The driving shoe was successful in reducing the effective horizontal stress acting on the pile by a factor of 4.

Note that the use of an internal driving shoe has no influence on external shaft friction. The close agreement between the profiles of Q_{so} for no shoe and an internal shoe indicate the reliability of this data.

5.3 Measured internal shaft friction

Whilst external shaft friction acts on the entire embedded depth of the pile, internal shaft friction acts only of the lower section of the inside surface which is in contact with the internal soil column (or plug). Extending the statements made in section 5.2, it can be said that internal shaft friction is a function of pile-soil friction angle, the horizontal effective stress within the soil column, and the length of the internal soil column. An internal driving shoe will alter both the horizontal stress distribution within the internal soil column, and also its length.

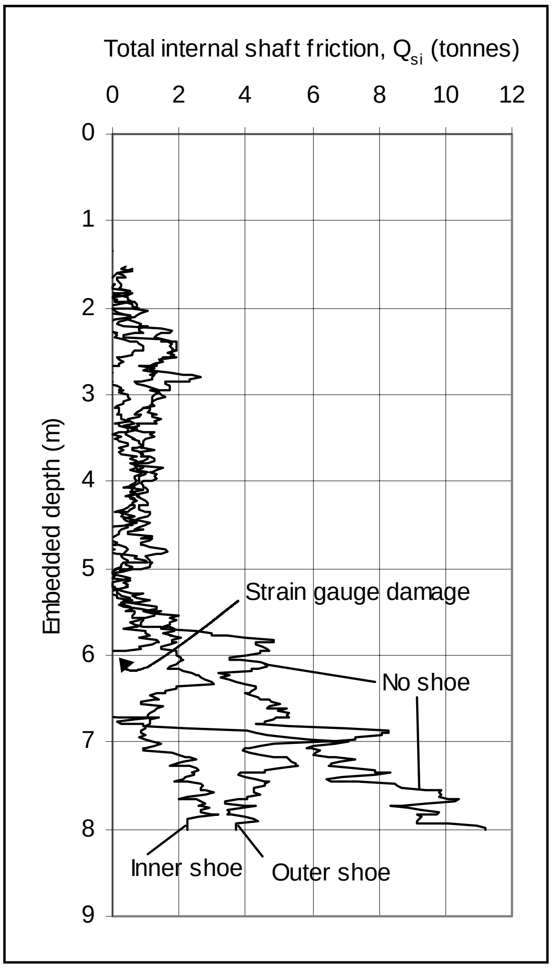

Figure 12; Total internal shaft friction vs embedded depth

Figure 12 shows that a large reduction in internal shaft friction is achieved by using an internal driving shoe. Beyond a depth of 5 metres (corresponding to a plug length greater than 2 metres), significant internal shaft friction is mobilised, and the influence of the internal driving shoe is clearly evident.

For a given embedded depth, internal shaft friction is generally reduced by a factor of 3. Concurrent examination of figures 7 and 12 allows internal shaft friction for a given plug length to be compared. The corresponding reduction in internal shaft friction lies in the range 2.5-3.

The profile of Q_{si} when using an external driving shoe appears anomalous. The sharp drop to a negative value at 6 metres depth suggests that the axial strain gauge at the bottom of the inner sleeve became damaged or debonded. Beyond 6 metres depth the readings fluctuate, and are considered unreliable. This gauge is also used to calculated base resistance. However, since the axial strain at the bottom of the inner sleeve is much smaller than the axial strain at the bottom of the outer sleeve, this damage has only a small influence on the base resistance profile shown in figure 9.

5.4 Measured reduction in jacking force

Despite reducing shaft friction by factors of 3 and 4 respectively, the internal and external driving shoes did not lead to a significant reduction in jacking force in this instance. This is because the reduction in shaft friction was partially balanced by an increase in base resistance created by the additional area of the driving shoes.

The instrumented double-skin pile used in this test series has a disproportionate base/shaft area ratio and was only installed to a shallow embedment. Consequently, shaft resistance comprised a smaller proportion of the total resistance than would be expected in an actual application. The relief of hard driving is more typically required when installing 36 inch tubular piles to depths of 15 metres or more. These piles have a base/shaft area ratio of which is 5 times greater than the test pile.

Furthermore, the undesirable increase in base resistance could be reduced if a smaller driving shoe flange can be used. The reduced shaft friction observed during this testing may be obtainable using a driving shoe flanges smaller than 15mm. The hypothesised mechanism shown in figure 2 requires that the soil is unloaded horizontally until the active condition is reached. The strains to mobilise active failure are very small due to the high unload stiffness of soil . Further testing with smaller flange sizes is required to optimise the driving shoe design.

6 Conclusions

Three field tests using the press-in method of pile installation have been carried out using a double-skinned tubular pile. These tests consisted of a control test followed by tests with internal and external driving shoes. Strain gauge instrumentation allowed independent measurement of internal and external shaft friction and base resistance. Independent measurement of axial and hoop stress was achieved by using a quarter-bridge arrangement, with installation being carried out slowly enough to allow temperature-induced apparent strains to be identified and corrected.

The measured unit base resistance on the pile wall, q_b, was not significantly influenced by the use of driving shoes. However, comparison with the CPT profile, q_c, indicated a base resistance scale effect. The ratio q_b/q_c decreased from 0.8 to 0.4 from the surface to the final embedment of 8 metres. This has implications for design based on CPT data. The assumption that $q_b = q_c$ is unconservative.

The influence of internal and external driving shoes on shaft friction was examined. Internal and external shaft friction were reduced by factors of 3 and 4 respectively. These changes are consistent with a reduction in the earth pressure coefficient (σ'_h/σ'_v) adjacent to the pile shaft to the active condition as the soil flows past the driving shoe.

7 References

[1] - Selby, A. R. (1997) "Control of Vibration and Noise During Piling." *Brochure publication. British Steel, pp12.*

[2] - de Nicola, A. (1996) "The performance of pipe piles in sand." *Ph.D. thesis, University of Western Australia, Nedlands 6907, Australia*

[3] - White, D. J., Sidhu, H.K., Finlay, T.C.R., Bolton, M.D. & Nagayama, T. (2000) "Press-in Piling: The influence of plugging on driveability," *Proc. 8th International Conference of the Deep Foundations Institute. New York*

[4] - Jamiolkowski M., Ladd C. C., Germaine J. T. & Lancelotta R. (1985) "New developments in field and laboratory testing of soils" Proc. 11th Conf. on Soil Mechanics and Foundation Engineering (1): 57-156

[5] - Jardine, R. J. & Chow, F. C. (1996) "New design methods for offshore piles." Imperial College, London. *MTD publication 96/103.*

[6] - Bustamante, M. & Gianaselli, L. (1992) "Pile bearing capacity by means of static penetrometer CPT." *Proc. 2nd Eur. Symp. on Penetration Testing, Amsterdam, 493-500*

[7] - de Ruiter, J. & Beringen, F. L. (1979) "Pile foundations for large north sea structures." *Marine Geotechnology, Vol. 3.*

[8] - de Nicola, A. & Randolph, M. F. (1997) "The plugging behaviour of driven and jacked piles in sand." *Géotechnique 47(4):841-856.*

[9] - Brucy, F., Meunier, J. & Nauroy, J-F. (1991) "Behaviour of a pile plug in sandy soils during and after driving." *Proc. Offshore Technology Conference, Houston, Texas, OTC6514:145-154.*

[10] - Meyerhof, G. G. (1983) "Scale effects of ultimate pile capacity." *J. Geotech. Engng. Div. ASCE 113(5):490-507.*

[11] - Chow, F. C., Jardine, R. J., Brucy, F. & Nauroy, J. F. (1996) "The effects of time on the capacity of pipe piles in dense marine sands." *Proc. 28th Annual Offshore Technology Conf., Houston, OTC7972.*

[12] - Fleming, W. G. K., Weltman, A. J., Randolph & M. F., Elson, W.K. (1992) "Piling Engineering" *(2nd edition) Blackie Academic and Professional*

[13] - Biddle, A. R. (1997) "The steel bearing piles guide." *(5th edition), Steel Construction Institute, Ascot, UK.*

[14] - Borghi, X., White, D. J., Bolton, M. D. & Springman, S. (2001) "Empirical pile design based on cone penetrometer test results: An explanation for the reduction of unit base resistance between CPTs and piles." *Proc. 5th International Conference on Deep Foundation Practice.*