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PRESS-IN PILING: THE INFLUENCE OF PLUGGING ON DRIVEABILITY

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1 Introduction

1.1 Urban construction of deep foundations

The decreasing availability of land in urban areas, and the increasing demand for urban development is leading to the construction of taller and heavier buildings at increasingly marginal sites. Such structures usually require founding at depth and modern construction methods are making the economics of piled foundations ever more attractive (Salgado, 1995).

However, the urban environment is not suited to pile driving. Conventional dynamic installation techniques (i.e. drop hammer or vibro-piling) induce vibrations and settlement in a zone close to the pile, and cause noise and dust pollution. Damage to existing structures resulting from pile driving vibrations is not uncommon (Tomlinson, 1986).

1.2 Press-in pile driving

Press-in pile driving machines provide an alternative method for installing pre-formed tubular or sheet piles. These machines use hydraulic rams to press piles into the ground using the negative shaft friction of previously-driven piles to provide reaction force (fig. 1.1). This technique eliminates the vibration, hammering and noise pollution associated with dynamic installation methods. Table 1.1 compares the noise created during installation of steel sheet piles using dynamic and press-in methods.

The press-in method is particularly suited for retrofit renewal of existing structures where the disruption of existing services must be avoided. Press-in pilers are capable of operating alongside live railways for embankment pile stabilisation and beneath bridge structures for pier strengthening.

Installation method	Observed noise level	Distance of observation
Press-in piler[1]	61 dB	7 m
Vibratory[2] (medium freq.)	90 dB	1 m
Drop hammer[2]	98-107dB	7 m
Light diesel hammer[2]	97 dB	18 m

[1] Giken Seisakusho 100 tonne 'Silent Piler'.
[2] Tomlinson, 1986

Table 1.1: Noise developed during pile installation using various methods

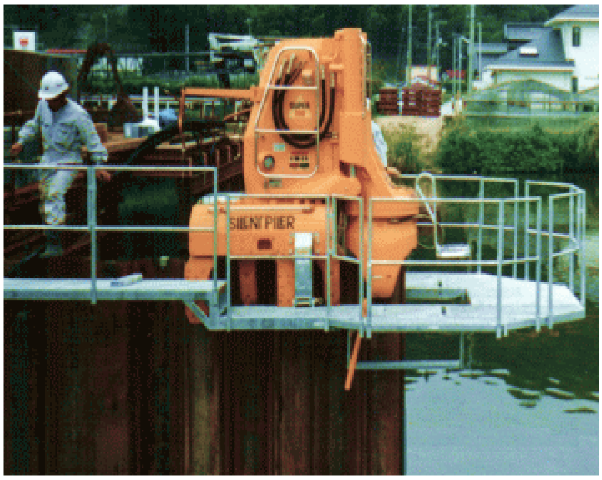

Figure 1.1: A press-in pile driver

Appropriate selection of machinery is more critical to operators of press-in pilers than conventional pile driving equipment. The driving load of a press-

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in pile is limited by the force capacity of the hydraulic rams. If unexpected resistance is met when driving with an undersized pile hammer or vibrator, penetration will be slowed, but total refusal is rare. In contrast, if the unexpected resistance exceeds the force capacity of the press-in piler, penetration is impossible.

1.3 Driving load prediction

When using dynamic installation methods, appropriate pile driving equipment is selected by predicting penetration resistance through dynamic analysis or an energy method (eg. Hiley, 1925). In contrast, since press-in pile installation applies a quasi-static load to the pile, static equilibrium analysis can be carried out to select an appropriate size of machine. Driving load can be predicted in the same manner as ultimate load by summing shaft friction and base resistance.

A variety of methods exist for this purpose, notably Bustamante & Gianeselli (1982), Kraft (1990), Toolan *et al* (1990), API RP2a (1993), Fleming *et al* (1992), Randolph (1994), Jardine & Chow (1996), and Eslami & Fellenius (1997). Recent reviews of these methods are presented by Hossain & Briaud (1993), Bandini & Salgado (1998), and Chow (1997). A common observation (Brucy *et al*, 1991; Hight *et al*, 1996; Randolph *et al*, 1994; De Nicola & Randolph, 1997) is that the plugging behaviour of tubular piles is not satisfactorily captured by existing design guidelines.

1.4 Plugging of tubular piles

An open-ended tubular pile can penetrate in an unplugged or a plugged manner. During unplugged penetration, the pile moves downwards relative to the internal soil column, in the manner of a sampler tube (fig. 1.2a). Penetration is resisted by shaft friction on the inside (Q_{si}) and outside (Q_{so}) of the pile and by base resistance on the annulus of pile wall (Q_w). During plugged penetration the internal soil column is dragged downwards (fig. 1.2b), and the pile assumes the characteristics of a closed-ended pile (Paikowsky *et al*, 1989). Penetration is resisted by shaft friction on the outside of the shaft (Q_{so}) and by base resistance on the pile wall (Q_w) and the soil plug (Q_p).

$$Q_{unplugged} = Q_{so} + Q_{si} + Q_w \qquad (1)$$
$$Q_{plugged} = Q_{so} + Q_w + Q_p - W_p \qquad (2)$$

When a tubular pile is being installed by the press-in method, (or is being loaded to failure- these events are analogous), penetration will occur by whichever mechanism is the weakest (Raines *et al*, 1992). If the shaft friction on the inside of the pile (Q_{si}) (plus the weight of the soil column) is greater than the base resistance of the soil column (Q_p), the pile will penetrate in a plugged manner. Hence, when predicting the driving load in order to select a machine type, the profiles of both unplugged and plugged penetration resistance with depth should be estimated. The lower bound of these two mechanisms will form the driving load.

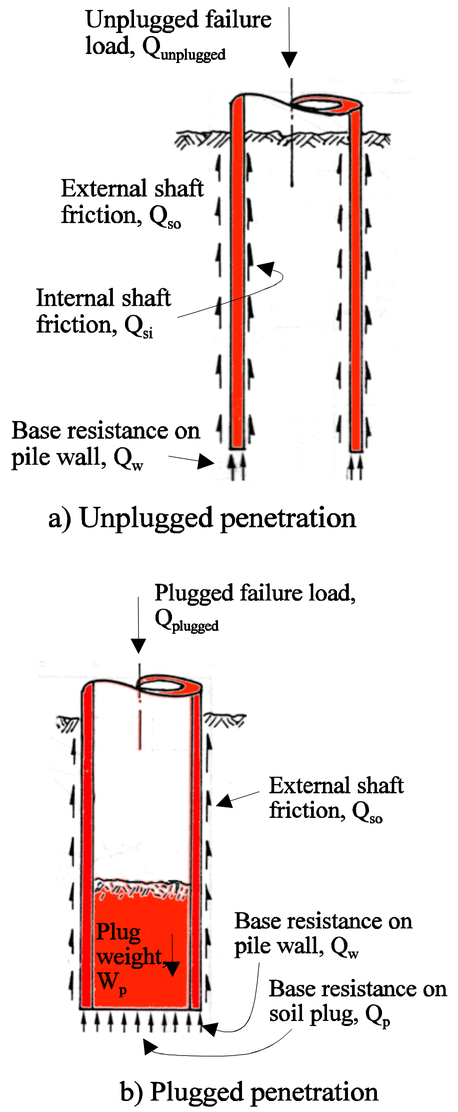


Figure 1.2: Alternative penetration mechanisms: unplugged and plugged

This paper presents the results of a series of field tests which shed light on the issues of driving load prediction and plugging. Two instrumented piles were used to observe the development of a soil plug during press-in installation and measure the stresses in the internal soil column.

A procedure for predicting the driving load and hence selecting an appropriate machine type is outlined. In conclusion, two novel construction techniques for optimising the use of press-in pilers are proposed in the light of this field data.

2 Analysis of tubular piles

When selecting a press-in machine, or predicting ultimate load, the task is to calculate the continuous profile of pile capacity (or driving load) from the ground surface to the installed depth. A simple example of this analysis is presented below, for a 800mm diameter tubular pile with 15mm wall thickness, installed at a hypothetical site consisting of uniform medium dense sand (fig. 2.2).

The driving load of a tubular pile is the sum of the shaft friction and the base resistance and is the lower bound of $Q_{unplugged}$ and $Q_{plugged}$ (eq^n 1,2) (ignoring the weight of the pile). The profile with depth of each component of these loads is calculated below.

Conventional design practice is to predict base resistance ($Q_{w,}$ Q_p) using a correlation with either friction angle and relative density (API RP2a, 1993; Fleming *et al*, 1992; Berezantzev *et al*, 1961), CPT resistance (Bustamante & Gianeselli, 1982; Jardine & Chow, 1996) or through a cavity expansion analysis (Randolph *et al*, 1994). In this example, the variation of base resistance with depth is illustrated using a simple correlation with CPT resistance.

Local shaft friction, τ_s, is generally predicted using the Coulomb equation (eq^n 3). To find the total external shaft friction, (Q_{so}), equation 3 is integrated over the pile surface area (eq^n 4). It is often assumed that the earth pressure coefficient, K, and pile-soil angle of friction, δ, remain constant along the length of the pile (Fleming *et al*, 1992; API RP2a, 1993) with limiting values of τ_s applying. Appropriate values from the API RP2a are used in this example.

$$\tau_s = \sigma_h' \tan\delta = K\,\sigma'_v \tan\delta \qquad (3)$$
$$Q_{so} = \int \tau_s\, \pi D\, dz \qquad (4)$$

Recent methods for predicting the capacity of driven piles (Toolan *et al*, 1990; Lings, 1997; Randolph *et al*, 1994; Jardine & Chow, 1996) include 'friction fatigue', to describe a progressive decrease in earth pressure coefficient and hence local shaft friction at a given soil horizon as the pile penetrates further. This effect can be attributed to compaction of the soil under the high cyclic loading caused by driving. There is evidence that friction fatigue is less significant when the pile is installed by jacking (Chow, 1997; De Nicola & Randolph, 1997), and it has been ignored in this simple analysis.

Internal shaft friction, Q_{si} is the most difficult component of pile capacity to evaluate. API RP2a (1993) assumes that the internal and external shaft friction profiles are equal. However, since the soil column is dragged downwards on all sides, 'arching' occurs. This is analogous to the arching within silos first described by Janssen (1894). Equilibrium analysis of a horizontal slice of soil shows that vertical stress increases exponentially with depth (eq^{ns} 5,6) (fig. 2.1).

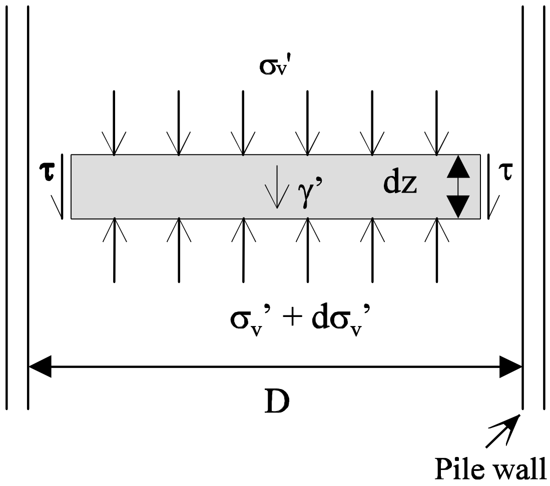


Figure 2.1: Equilibrium of a horizontal slice of soil within a tubular pile

$$d\sigma_v' / dz = \gamma' + 4\,K\,\sigma_v' \tan\delta / D \qquad (5)$$
$$\sigma_v' = \gamma' D\,(e^{4Kh\tan\delta / D} - 1) / 4K\tan\delta \qquad (6)$$

The total internal shaft friction can be found by integrating equation 5 over the height of the soil column, h, (eq^n 6), and considering the overall equilibrium of the column (eq^n 7):

$$Q_{si} = \sigma_v'\,\pi D^2/4 - W_p = (\gamma' D^3\pi/16K\tan\delta)(e^{4Kh\tan\delta / D} - 1) - \gamma' h\,\pi D^2/4 \qquad (7)$$

This analysis adopts the following recommended design variables for medium dense sand; K = 0.8 (API RP2a, 1993; De Nicola & Randolph, 1997) and δ = 25° (API RP2a 1993).

Figure 2.2 shows the sharp build up of internal shaft friction and hence driving load, as the length of the soil column increases during unplugged penetration. At the transition to plugged penetration, the soil column will 'lock up' and remain approximately of length, hcrit.

Beyond the transition to plugged penetration, the driving load increases gradually with $Q_{plugged}$. Minimal further slippage may occur in order to maintain sufficient internal shaft friction to balance the increase in $Q_{plugged}$.

The sharply diverging curves of $Q_{plugged}$ and $Q_{unplugged}$ indicate the importance of considering both failure modes during ultimate load prediction. At an embedment of 0.5m less than h_{crit}, $Q_{plugged}$ is 2.5 times greater than the lower bound. In contrast, 0.5m deeper than h_{crit}, $Q_{unplugged}$ is 3 times greater than the lower bound.

The key feature of this curve is the exponential increase in internal shaft friction. The exponential term contains K, and tan δ, and scales with γ'. Whereas many accepted methods exist for predicting external shaft friction and base resistance, the physical processes which govern the plugging mechanism during driving and subsequent static loading are poorly understood (De Nicola & Randolph, 1997). Consequently, selection of the correct values for these crucial exponential parameters remains a significant source of uncertainty.

3 Fieldwork

3.1 Site description

Two tubular test piles were installed at the Takasu Research Centre in Kochi on the island of Shikoku, Japan. The stratigraphy consists of made ground overlying silt and sand (figure 3.1). The results of laboratory testing of each strata are shown in table 3.1. CPT data has been used to estimate a profile of relative density following Jamiolkowski *et al* (1985).

Since Test A took place following recent rainfall, the groundwater table was located 0.75m below ground level. During test B, this had reverted to 2m below ground level.

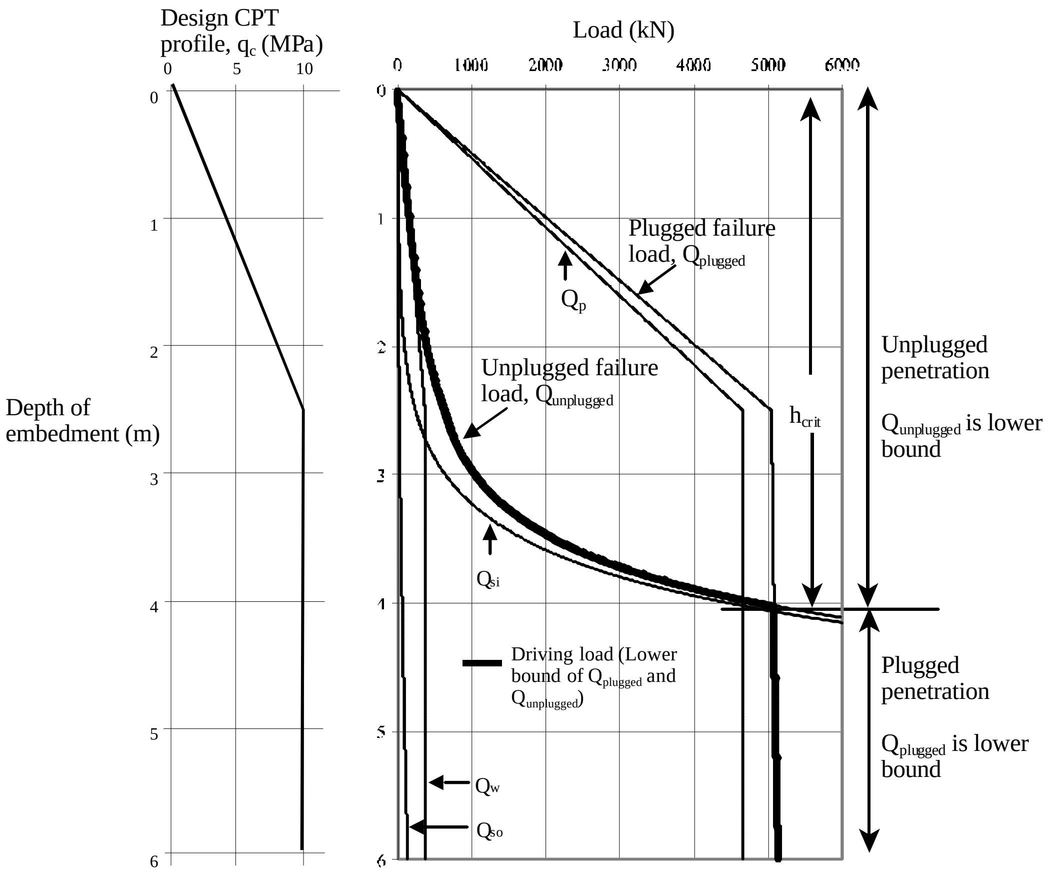


Figure 2.2: Predicted driving load vs. embedded depth at a typical site

Stratum	f_{crit}	D_{10} (mm)	D_{50} (mm)
A: Made ground	-	-	-
B: Silt	39E	-	-
C: Sand & gravel	44E	0.22	1
D: Silty sand	39E	0.047	0.18
E: Sand	42E	0.18	0.3

Table 3.1: Soil properties

3.2 Pile and piler description

The dimensions of the two test piles are shown in table 3.2.

Test pile	D_{outer} (mm)	Wall thickness t (mm)	Wall area, A_{wall} (mm)	Plug area, A_{plug} (mm)
A	318.5	6.9	9731	72918
B	162.5	5.0	5450	18265

Table 3.2: Dimensions of test piles

A specially modified press-in piler was used to install the test piles. Continuous calculation of driving load was achieved by measurement of the oil pressure in the hydraulic rams. Embedded depth was measured using a displacement potentiometer attached to the head of the pile. The level of the soil column within the pile was measured using a displacement potentiometer attached to a weighted follower within the pile.

The piles were installed in a series of 700mm strokes at a rate of approximately 50mm/sec. A pause of 15 seconds between strokes occurred whilst the rams were retracted.

3.3 Pile instrumentation

The aim of these tests was to investigate the phenomenon of plugging, and shed light on the distribution of stress within the plug. The internal shaft friction is governed by the horizontal stress on the pile wall, as well as the pile/soil interface angle of friction. Previous pile tests using closed-end tubular piles have measured horizontal stress using delicate total pressure cells embedded in the pile wall (eg. Coop, 1987; Chow, 1997).

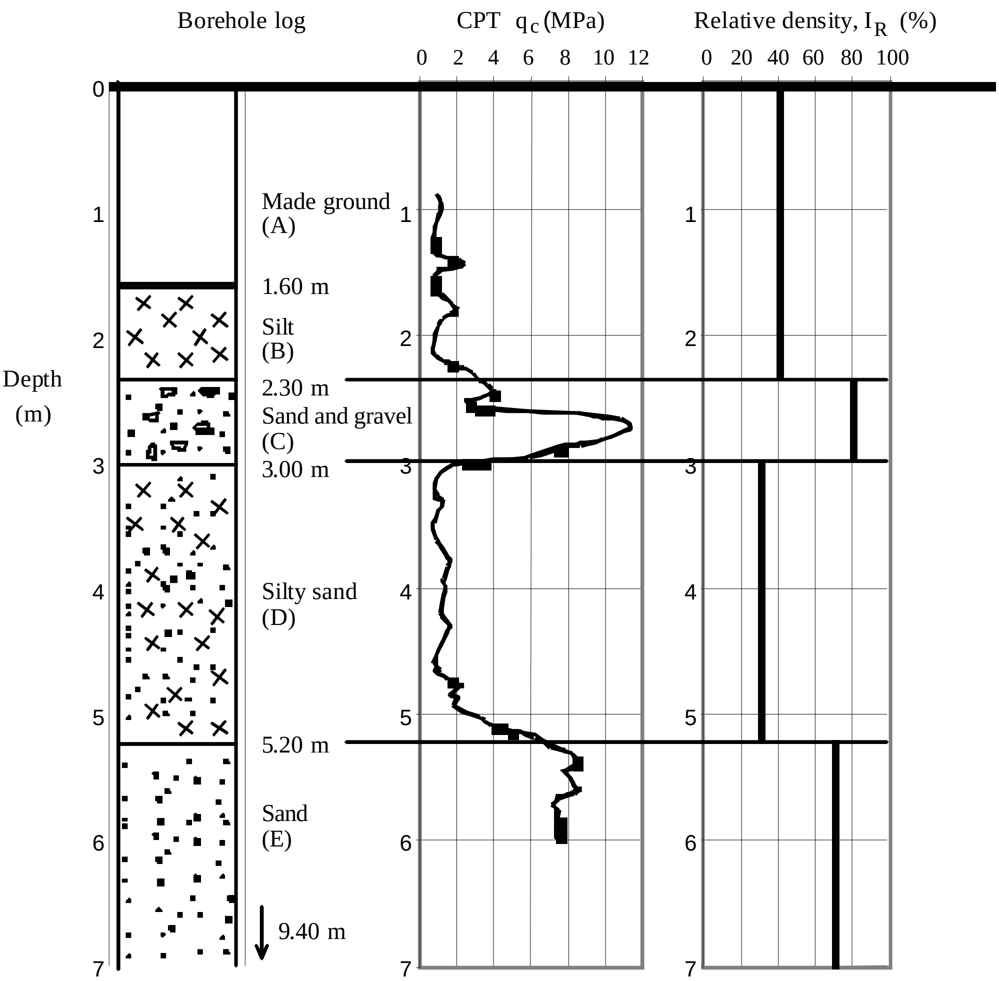
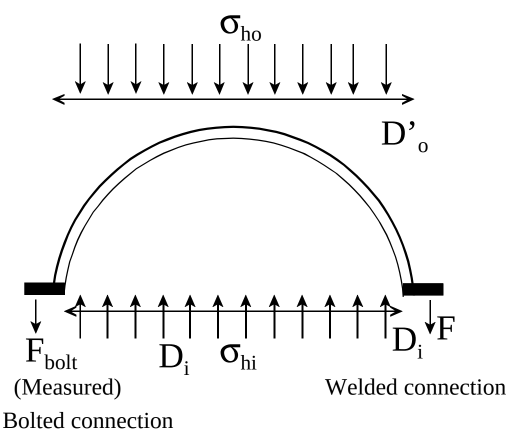


Figure 3.1: Site stratigraphy and CPT profile

However, since external shaft friction (and hence horizontal stress) is very much lower than internal shaft friction, a differential pressure is created across the wall of a tubular pile. This differential pressure causes a hoop stress within the pile wall (figure 3.2). This hoop stress is much larger than the horizontal stress within the soil. Consequently it is preferable to use instrumentation which infers the horizontal stress from the hoop stress taking advantage of the increased signal size.



$$\sigma_{hoop} = (\sigma_{hi} D_i - \sigma_{ho} D'_o) / 2t \quad (8)$$
$$F = \sigma_{hoop}\, t\, L \quad (9)$$
$$\sigma_{hi} = (2 F/L + \sigma_{ho} D'_o) / D_i \quad (10)$$

L = longitudinal bolt separation

Figure 3.2: Calculation of internal horizontal stress from bolt force

The relative magnitudes of the external and internal horizontal stresses can be estimated by comparing design method profiles for external shaft friction with the results of equation (6) which predicts the distribution of internal shaft friction. Even when external shaft friction is conservatively assumed to increase sharply close to the pile toe (eg. Randolph *et al*, 1994), the external horizontal stress in this zone is typically only 2-5% of the internal horizontal stress. Hence, the hoop stress induced in the pile wall is almost entirely due to the internal horizontal stress. During analysis of the instrumented bolt data, the minor correction for external horizontal stress depicted in equation 10 has been carried out assuming external shaft friction varies as predicted by API RP2a. Pore pressures are assumed to be hydrostatic.

Each test pile was split longitudinally and the two sections were re-assembled using strain-gauged bolts at 300mm intervals (figure 3.3). Whereas the detection of horizontal stress using a total pressure cell requires careful manufacture of the device and consideration of cell action effects, the instrumented bolts are relatively cheap and quick to manufacture.

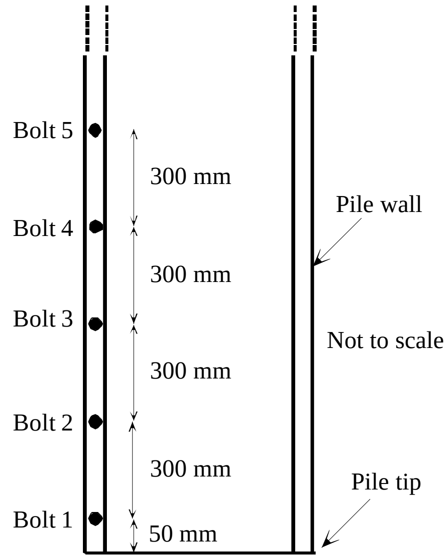


Figure 3.3: Location of instrumented bolts

4 Driving load and plugging measurements

Figure 4.1 shows the measured profiles of driving load and soil column length with embedded depth. Measurement of soil column length failed at 5m depth during the installation of pile B; the test was repeated using an uninstrumented pile to obtain trace C.

The driving load follows the trend of the CPT data by increasing at depths of 2.3m and 5.5m. At the end of each 700mm press-in stroke, a drop in driving load can be detected. This could be attributed to pore pressure dissipation at the base leading to a reduction in base resistance.

Whereas the hypothetical example shown in figure 2.2 predicted a single transition from unplugged to plugged penetration, the field data shows that multiple transitions can occur in layered soil. After an initial period of unplugged penetration, the soil column becomes sufficiently long to form a plug at approximately 0.8m depth. Penetration continues in a plugged manner (with a brief period of slippage during Test A between 1.4 and 1.7m) until the hard layer at 2.3m depth is reached.

Upon reaching a hard layer, plugged resistance increases sharply since base resistance follows the CPT profile. In contrast, unplugged resistance is less influenced by a hard layer. Hence, the lower bound mechanism reverts to unplugged penetration. This causes the soil column to increase in length, accompanied by an exponential increase in internal shaft friction. At approximately 2.7m depth this increase in internal shaft friction causes the lower bound mechanism to revert back to plugged penetration. On reaching the sand layer at 5.2m depth, penetration again becomes unplugged.

5 Measurement of stresses in soil column

The output from the instrumented bolts was converted to internal horizontal stress (σ'_{hi}) according to the calculation method shown in equations 8-10. Fig. 5.1 shows the variation of horizontal stress with embedded depth at each bolt location. The quantity s/D indicates the distance of each bolt from the pile toe (s) normalised with pile diameter (D).

The similarity between the profiles of horizontal stress with the variation of soil column length with depth is notable. Changes in soil column length are associated with large increases in horizontal stress at a given bolt location. These profiles of horizontal stress are converted to plug force (Q_{si} + W_p) by integration over the length of the pile (fig. 5.2).

The measured curves of plug force closely match design predictions found using arching theory (eq^n 7) with input parameters, K and tan δ, chosen as proposed by De Nicola & Randolph (1997). These guidelines are derived from centrifuge test data of jacked model piles. The correlation with these full-scale results is remarkable, particularly since the exponential curve is very sensitive to the input parameters.

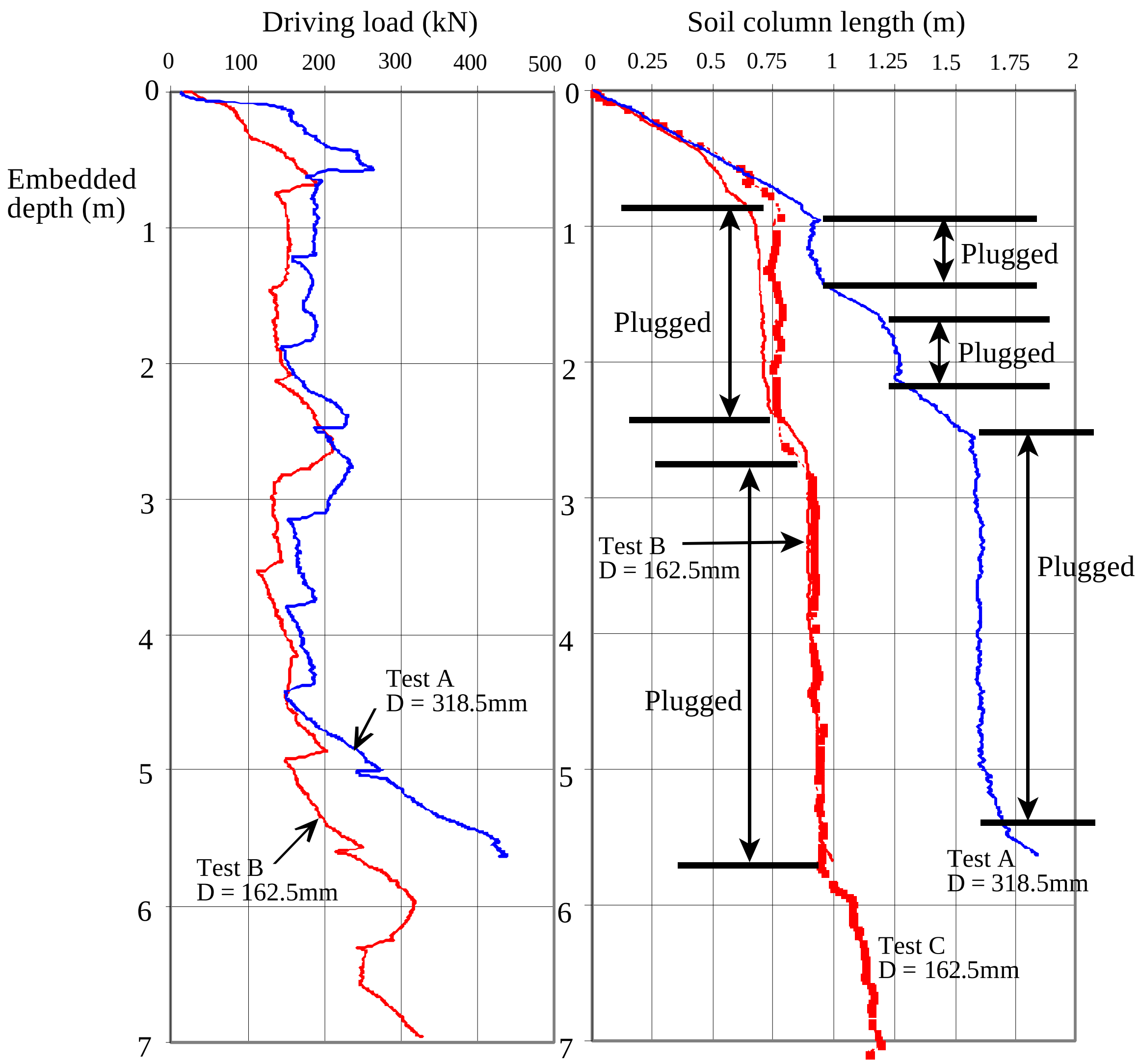


Figure 4.1: Driving load and soil column length vs. embedded depth

The analysis of dynamically-installed piles often assumes that a pile plug consists of a 'wedged' and 'unwedged' zone (Murff *et al*, 1990; Randolph *et al*, 1991). The 'unwedged' zone is considered not to mobilise shaft friction and act only as surcharge on the lower 'wedged' zone. The physical explanation for this phenomenon is that the cyclic loading created during installation will loosen the top of the soil column. Since press-in installation is monotonic, this loosening will not occur. The design predictions shown on figure 5.2 consider that the entire plug is 'wedged'.

If further research confirms that arching theory provides a robust technique for predicting internal shaft friction, this will provide a valuable technique for estimating the strength of pressed-in piles. Reliance on empirical methods for prediction of plugging can be eliminated.

6 Implications for the press-in construction technique

6.1 Sensitivity of plug capacity

The exponential build of internal horizontal stress observed during these field tests and predicted by arching theory indicates the sensitivity of internal shaft friction (Q_{si}) to the governing parameters K, tan δ and γ' in equation (7). This phenomenon could be exploited to improve the driveability of pressed-in piles.

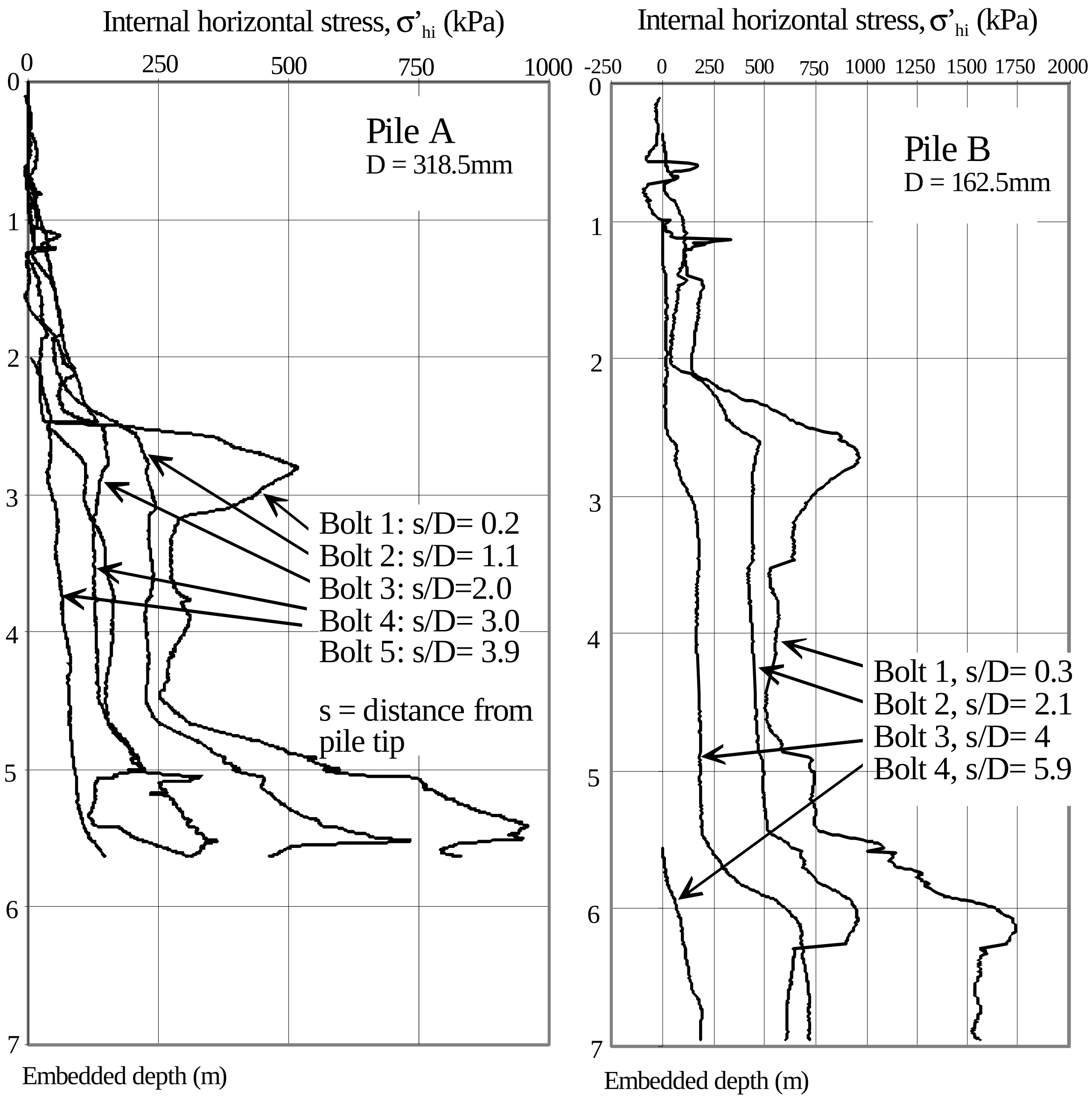


Figure 5.1: Variation of internal horizontal stress, s'_{hi} with embedded depth

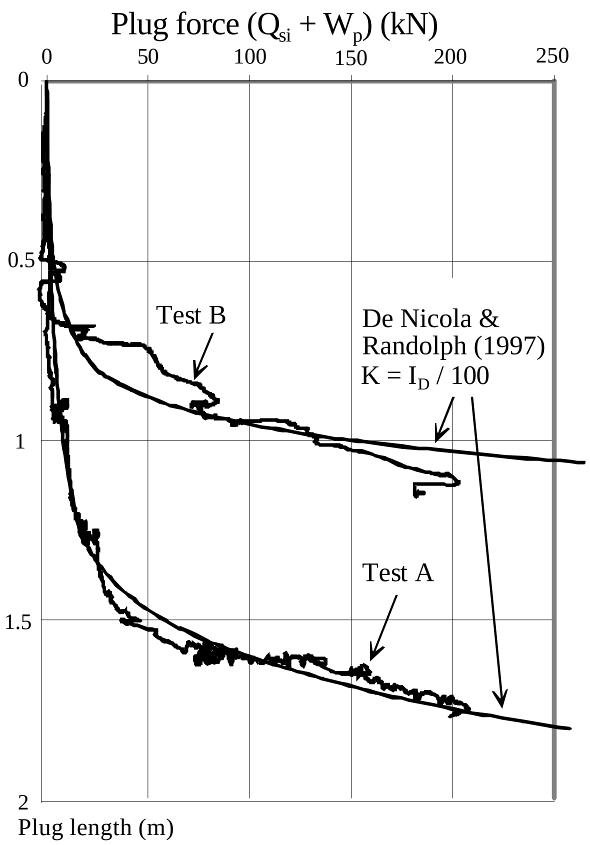


Figure 5.2: Variation of plug force (Q_{si} + W_p) with plug length

The field data shows that internal shaft friction accounts for more than 60% of the driving load of pile A beyond depths of 3m. A construction procedure which reduces any of the controlling variables, K, tan δ or γ', could dramatically reduce the driving load of pressed-in piles. For laterally loaded piles, this decrease in axial capacity is not critical. If the adjustment can be recovered, axially loaded piles of significantly greater capacity than the strength of the press-in piler could also be installed. Two possible construction variations are discussed below.

6.2 Driving shoe to reduce K

The use of a driving shoe can provide relief of internal horizontal stress by reducing the earth pressure coefficient, K (De Nicola & Randolph, 1997). Field tests have indicated that dynamic driving resistance can be significantly reduced through the use of a driving shoe (Raines *et al*, 1992).

The physical mechanism by which K is reduced is hypothesised in figure 6.1. As an element of soil passes through the shoe, the large decrease in horizontal confinement causes a reduction in horizontal stress. The vertical stress remains high, governed by the internal shaft friction of the upper part of the soil column.

Consequently, the lower bound value of K is governed by the active failure criterion (eq^n 11). In the case of the field data presented above, this would represent a reduction of K from 0.8 to 0.4 in the dense sand strata.

$$K_{active} = (1 - \sin\phi) / (1 + \sin\phi) \qquad (11)$$

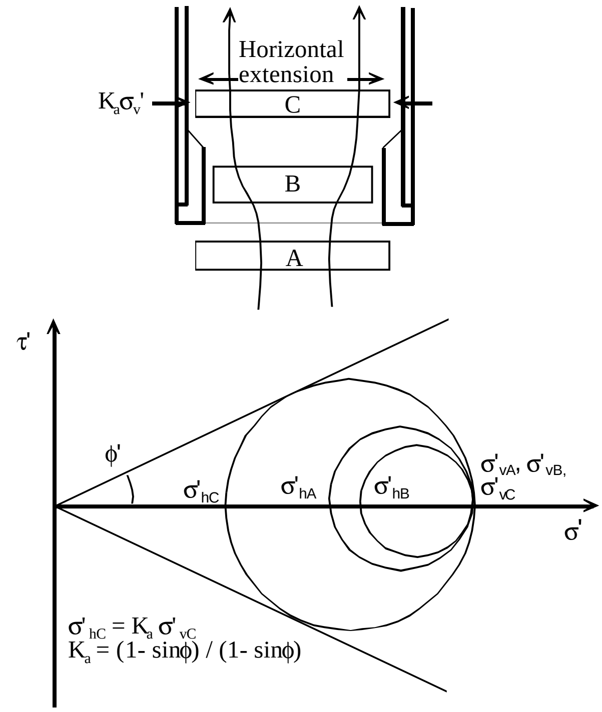


Figure 6.1: Hypothesised stress changes due to driving shoe

This change in K leads to a dramatic reduction of internal shaft friction. This is best illustrated by plotting the stress at the base of the soil column due to internal shaft friction and the weight of the soil, $(Q_{si}+W_p)/A_p$, against depth (figure 6.2).

If $(Q_{si}+W_p)/A_p$ is lower than base resistance, q_b, an unplugged mechanism will form the lower bound, plugging will be avoided, unplugged penetration will occur, and consequently driving load will be reduced.

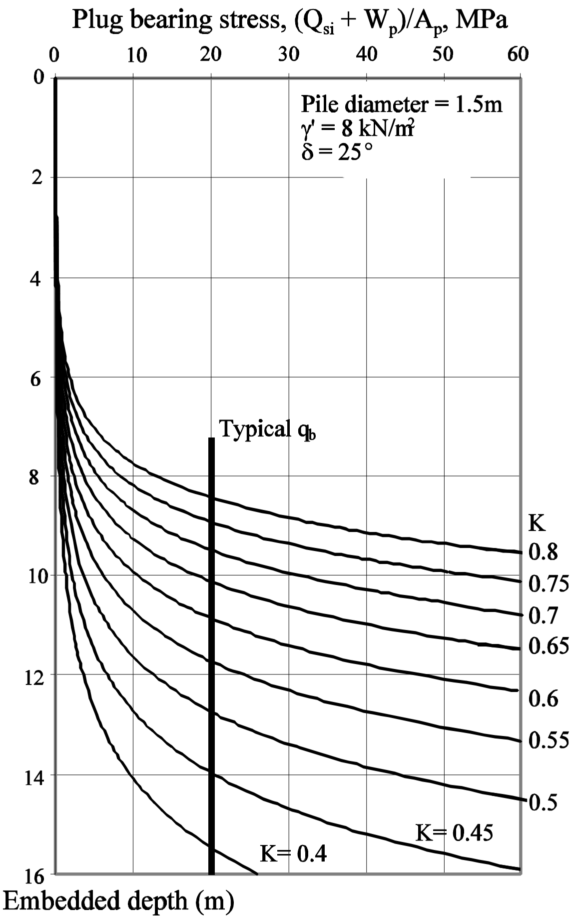


Figure 6.2: Effect of reducing K on plug force

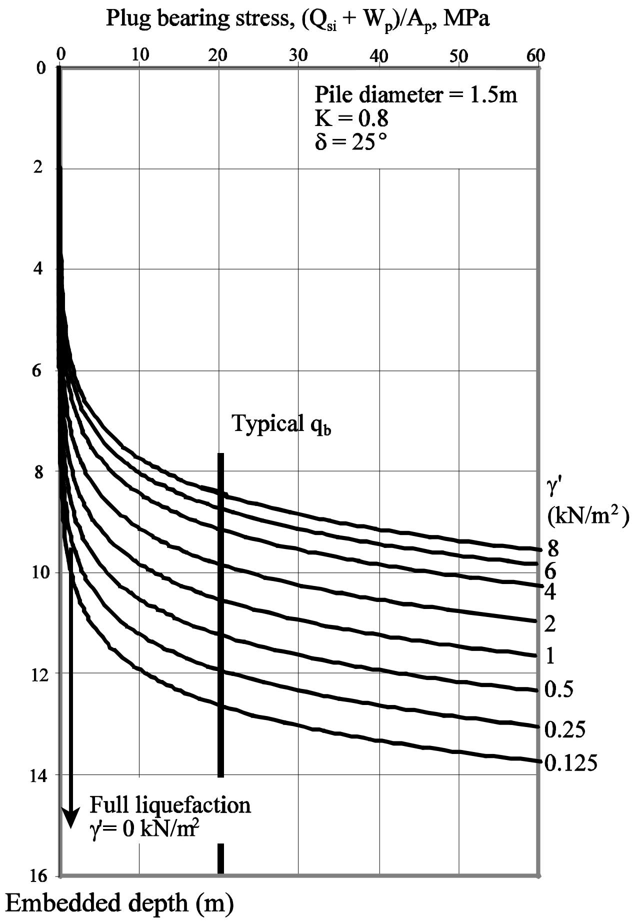


Figure 6.3: Effect of reducing g' on plug force

6.3 Water injection to reduce g'

A drawback of reducing driving load by using a shoe is that the change in capacity is difficult to recover. An alternative method to ease driving is to inject water close to the base of the soil column which then flows up the column creating an upward hydraulic gradient. This upward hydraulic gradient reduces the buoyant weight of the soil column, with a consequent reduction of internal shaft friction.

Figure 6.3 demonstrates how varying γ' could reduce internal shaft friction with a resulting reduction in driving load. The soil column need not be fully liquefied. Following installation, the pore pressures will return to hydrostatic and the lost internal shaft friction will become available.

Field testing is required to confirm this hypothesis, and to ensure than long term external shaft friction and base resistance are not adversely affected by the water injection.

A further method of improving plug strength post-installation to increase the ultimate capacity beyond the driving load is described by Randolph *et al* (1991). Placing a relatively small surcharge onto the soil plug after installation leads to a dramatic increase of internal shaft friction. Equation 6 can be rewritten as shown in equation 12 to calculate the increase in vertical stress within the soil plug caused by a surcharge, p, applied to the top of the soil plug.

$$\sigma_v' = p + (p + \gamma' D / 4K \tan\delta)\,(e^{4Kh\ \tan\delta / D} - 1) \qquad (12)$$

7 Conclusions

The novel installation technique used by press-in pile drivers allows construction to proceed without the environmental problems associated with dynamic pile installation methods. However the deployment of press-in pile drivers is currently limited to sites where the static load capacity of the machine is above the maximum expected resistance.

Predicting this resistance remains particularly problematic when installing tubular piles. Conventional design methods either ignore internal shaft friction, or use an empirical correlation to determine the critical failure mode; plugged, or unplugged. A more robust analysis technique is to evaluate driving load as the lower bound of plugged and unplugged failure loads.

Field tests on two pressed-in instrumented tubular piles indicate that internal shaft friction is well predicted by arching theory (Janssen, 1894). Design values of K derived from centrifuge test data (De Nicola & Randolph, 1997) closely fit the field results.

If further field tests confirm these results over a wider range of ground conditions and pile types, arching theory will provide a valuable tool for predicting the strength of pressed-in piles based on static analysis, without recourse to empirical relationships to understand plugging behaviour.

Since internal shaft friction evolves according to an exponential relationship, large reductions can be achieved through only a small change in the governing parameters, K, tan δ and γ'. It is hypothesised that novel construction techniques which take advantage of this phenomenon could lead to wider applications of pressed-in piles.

The use of a driving shoe is proposed as a technique for permanently reducing internal shaft friction by decreasing the horizontal earth pressure coefficient within the internal soil column. Water injection is proposed as a technique of temporarily reducing the buoyant weight of the soil within the pile during installation. The resulting loss of internal shaft friction would be recoverable after installation.

Field testing is required to assess the viability of these techniques. Little quantitative research has been previously conducted into the influence of driving shoes and water injection.

The field tests described in this paper were carried out in sand. The field data was analysed under the assumption that the press-in installation took place under drained conditions, with pore pressures remaining hydrostatic. Any error would be minimal since typical increases in pore pressure induced by pile installation would not significantly alter the large stresses measured within the soil plug.

However, further investigation of the pore pressures generated during press-in installation is needed to assess whether the shaft friction and base resistance encountered during driving are different to the ultimate values.

8 Acknowledgements

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