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CENTRIFUGE MODELLING OF TUNNELLING NEAR DRIVEN PILES

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ABSTRACT

A centrifuge model study was carried out to investigate the effects of tunnelling on nearby single piles in dense dry sand. This paper presents a description of the centrifuge model and the test procedures followed. The surface settlement profiles recorded during tunnelling-induced volume loss are discussed, as well as the settlement of piles installed at various distances from the tunnel. A zone was identified around the tunnel in which a potential for large settlement exists. Changes in the load distribution on individual piles at various locations during increasing volume loss are presented.

Key words: centrifuge, foundations, load distribution, piles, settlement, tunnelling, volume loss (**IGC**: E12)

INTRODUCTION

The impact of tunnelling on nearby piled foundations is not well understood. This problem has to be confronted increasingly in urban areas, e.g. where tunnels are constructed as part of metro systems. As a result of the many uncertainties regarding the potential interaction between tunnels and piled foundations, over-conservative and costly solutions are often adopted.

During the construction of the Channel Tunnel Rail Link in London and the new North-South Metro Line in Amsterdam, tunnels will be constructed close to the piled foundations of a variety of structures. In several instances mitigation measures might be required prior to tunnel construction to prevent damage to overhead structures. This necessitates a thorough understanding of the tunnel-pile interaction problem. However, very limited information is currently available in the literature on the effects of tunnelling on piled foundations.

A number of studies using numerical analyses to investigate the problem have been published (e.g. Lee et al., 1994; Vermeer and Bonnier, 1991; Mroueh and Sharour, 1999) on tunnel-pile interaction problems. With the exception of Mroueh and Sharour (1999), the problems were reduced to plane-strain, introducing considerable simplification. The researchers were mostly interested in tunnelling-induced bending moments in piles close to the tunnels, and generally did not investigate tunnelling-induced pile settlement. However, Vermeer and Bonnier (1991) analysed end-bearing piles in sand, with very soft clay around the shafts. Their model predicted pile settlement to be similar to the settlement of the sand bearing layer. However, tunnelling-induced settlement might have been found to be different for the case where a more competent material surrounded the pile shafts. This was however not investigated.

Chen et al. (1999) presented an analytical method to predict the effects of tunnelling on piles in clay. The method is based on a closed-form solution for tunnelling-induced ground movements by Loganathan and Poulos (1998). Lateral and axial pile responses to the calculated ground movements are computed separately using a boundary element analysis. The analysis showed that lateral pile deflections were very similar to the calculated soil movements. Pile settlement was predicted to be very uniform along the length of the pile due to the pile's high axial stiffness relative to that of the soil. This method essentially assumes elasticity, but allows limits to be placed on the shear and normal stresses that can be transferred to the pile shaft. The method does however not investigate the effects of tunnel excavation on the high end-bearing stresses below nearby pile bases and therefore assumes that the capacity of the material supporting the base remains constant.

This paper reports on a centrifuge model study, carried out on the Cambridge Geotechnical Centrifuge (Schofield, 1980), to investigate the effects of tunnelling near driven piles in dense dry sand. Physical modelling,

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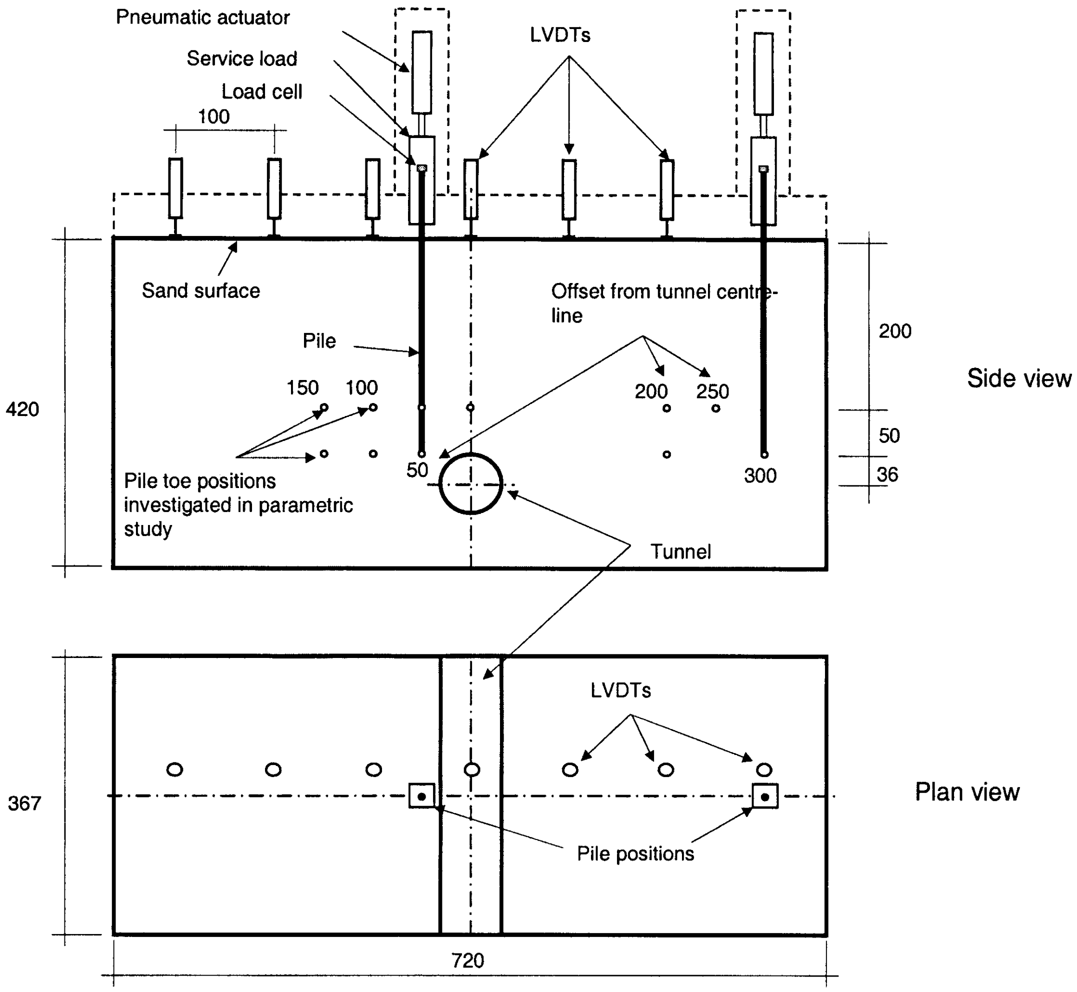

Fig. 1. Illustration of the centrifuge model (dimensions in mm)

rather than a numerical investigation, was chosen because of the highly three-dimensional nature of the problem and the complexities involved in modelling factors such as pile driving, the distribution of load on a pile and tunnel deformation. Advances in software development and computing power might perhaps enable realistic three-dimensional numerical analyses of this problem to be carried out in the future, but at present the viability of such analyses is questionable.

The study initially focussed on the settlement of single piles in response to tunnelling-induced volume loss. This enabled a zone of influence around a tunnel to be identified in which a potential for large settlements exists. Instrumented model piles were developed to measure changes in the load distribution on single piles as volume loss was imposed.

All piles were installed with their bases located at depths above the tunnel crown, since piles at these locations are most prone to tunnelling-induced settlement. Chen et al. (1999) reported small bending moments in piles with their bases located above the tunnel axis. However, the main emphasis in this study concerned the settlement and change in load distribution along the pile: bending of piles due to tunnelling was not investigated in the tests described in this paper, as this aspect relates to the structural design of the pile which fell outside the scope of the research.

MODEL DESCRIPTION

The centrifuge model is illustrated in Fig. 1. The dimensions of the model were determined by various practical constraints, and all tests were carried out at 75 g, so that the tests modelled a 4.5 m diameter tunnel at a depth of 21.5 m at prototype scale. The components of the centrifuge model are described below; comprehensive details are given by Jacobsz (2002).

Strong-Box

The model tunnel, the sand around it and the model piles were contained in an aluminium alloy strong-box designed for the project. The box measured $750\times400\times470$ mm deep, equating to $56\times30\times35$ m deep at prototype scale. It was constructed from 16 mm thick plates with local stiffeners to minimise deformation. Two 60 mm diameter circular openings in the sides of the strong-box accommodated the model tunnel.

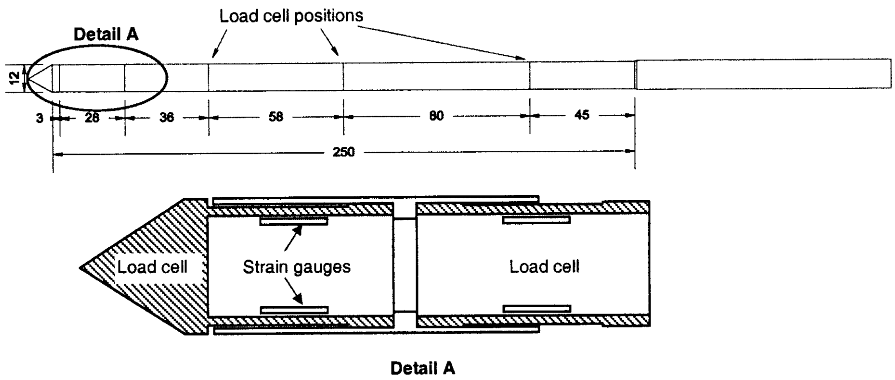

Fig. 2. Segmental instrumented model pile with axial load cells

Model Tunnel

The model tunnel consisted of a brass pipe surrounded by a 1 mm thick latex membrane. A 4 mm thick annulus between the pipe and the membrane was filled with water that could be extracted to simulate volume losses from 0% to about 20%. A pressure transducer incorporated into the tunnel control system enabled the pressure in the annulus to be monitored. The outer diameter of the model tunnel (D) was 60 mm, representing a 4.5 m diameter tunnel at prototype scale.

The tunnel was connected via a solenoid valve to a standpipe in which a constant water level was maintained to balance automatically the tunnel pressure with the overburden pressure during the acceleration of the centrifuge and driving of the piles. After the desired acceleration (75 g) had been achieved and the model piles driven, the solenoid valve was closed and volume loss imposed by slowly extracting water from the tunnel.

Instrumented Model Piles

Segmental instrumented model piles, machined from aluminium alloy tubing (Dural, grade HE30-TF, $E \approx 70$ GPa), were used to examine the effects of volume loss on the axial pile load distribution. The outer diameter of the piles was 12 mm over a length of 250 mm. The diameter scaled to 900 mm at prototype scale and the pile length installed into the sand to 18.75 m. The total length of the piles was 360 mm, the upper 110 mm being left unmachined.

Brass weights were slid over the unmachined length of the piles to exert realistic service loads during the tests. The service loads were sized to exert a load equal to roughly 50% of the penetration resistance of the model piles, a value (i.e. providing a factor of safety of 2) often adopted in pile design. Piles installed to a depth of 200 mm (*see* Fig. 1) were loaded with 1.4 kg service loads, exerting approximately 1 kN at 75 g and piles installed to 250 mm were loaded with 1.7 kg loads corresponding to about 1.25 kN at 75 g. Load cells, incorporated into the service weights, enabled the load transferred to the piles to be measured at all times. The piles were fitted with 60° conical tips, also machined from Dural. Linear Variable Differential Transformers (LVDTs), resting on the service weights, enabled pile settlement to be monitored.

The instrumented piles were each equipped with five or six axial load cells along their lengths as shown in Fig. 2. The axial load cells were cylindrical and joined the pile segments together. They measured 20 mm in length with an internal diameter of 9 mm. The wall thickness of the instrumented part of the load cells was 0.75 mm. All load cells were internally instrumented with strain gauges, connected in full Wheatstone bridges to produce a temperature compensated system.

The sensitivity of the axial load cells was approximately 4×10^{-6} V/N (before amplification of the signal). The strain gauge circuits were powered by a 5V DC source and the output signals were amplified 100 times before logging on the centrifuge's on-board data acquisition system. The pile components were bonded together using an adhesive and gaps between the segments were filled with silicone rubber to prevent sand ingress during testing.

After assembly, the model piles were calibrated by supporting them in a brass tube of practically the same diameter as the pile, while known loads were applied to the top of the piles using a hanger system.

Pile Driving Actuators

Pneumatic actuators (i.e. piston devices in which the applied air pressure driving the piston can be controlled), mounted on support frames, were used to push down onto the weights resting on the piles to drive the piles approximately 25 mm in flight (i.e. at 75 g) to fully mobilise the base capacity prior to inducing volume loss. The piles were driven by gradually increasing the air pressure in the actuators while monitoring the settlements. The rate of penetration was kept roughly constant between tests (typically 15 mm/min). After driving, the actuator pistons were retracted to leave only the service loads acting on the piles.

Sand

The strong-box was filled with dry fine silica sand (Leighton Buzzard sand) with a grading ranging between about 90 μm and 150 μm. The uniformity coefficient (U_c) of the sand was 1.6, the specific gravity of the grains 2.67 and the minimum and maximum dry density respectively 1357 kg/m^3 and 1633 kg/m^3 (Lee, 2001).

Model Preparation

Sand was pluviated into the strong-box from a hopper from a constant height and at a constant flow rate. The sand travelled approximately 600 mm through a flexible hose before falling an additional 300 mm to the sand surface. A fairly uniform sand density was achieved, generally of around 1560 kg/m^3 (75% ± 2% of maximum relative density). To ensure that sand could be placed at a uniform density in the strong-box, especially around the model tunnel, it was poured parallel to the tunnel axis by turning the strong-box on its side.

The model piles were installed after completion of the sand pouring procedure once the box had been turned upright. The piles were pushed into the sand to a position 25 mm above their final depths using a lead-screw assembly, equipped with a rotary bearing to prevent rotation of the piles.

Test Procedure

A typical centrifuge test comprised the following steps.

- Acceleration of centrifuge to 75 g.
- Driving model piles 25 mm to final depth.
- Retraction of pile driving actuators.
- Closure of the solenoid valve connecting the model tunnel to the standpipe.
- Extraction of water from the model tunnel to impose volume loss at a rate of 0.7% per minute up to 10% volume loss and then at a rate of 2.8% per minute to 20% volume loss.
- Centrifuge stopped.
- Pile settlement and load distributions and surface settlements were monitored throughout each test.

Retraction of the pile driving actuators was accompanied by a small amount of pile rebound, resulting in a significant reduction in the mobilised shaft friction.

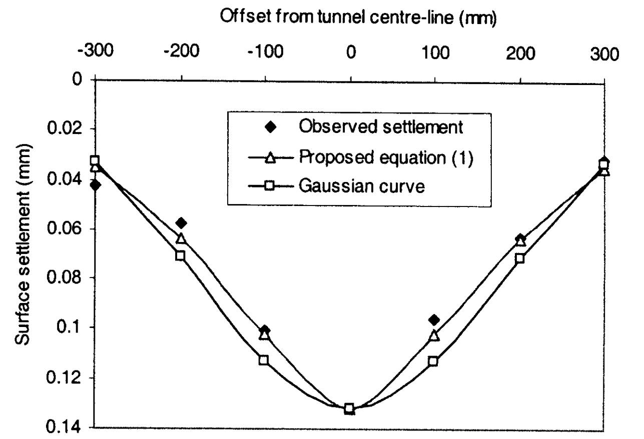

Fig. 3(a). The modified settlement trough equation and the "traditional" Gaussian curve compared to centrifuge data

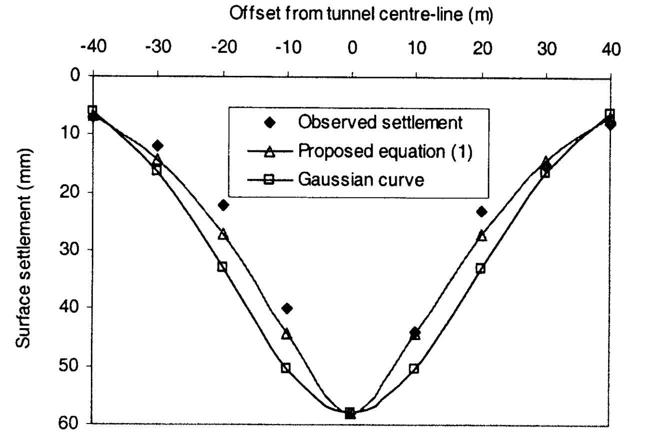

Fig. 3(b). The modified settlement trough equation and the "traditional" Gaussian curve compared with field data obtained in sand (Gioda and Locatelli, 1999)

SURFACE SETTLEMENT

Surface settlement was measured with an array of seven LVDTs spaced at 100 mm (7.5 m at prototype scale). The settlement troughs resembled the classical Gaussian-shaped settlement trough, although they were somewhat narrower, similar to the findings of various authors who have reported on settlement troughs in sand (e.g. Mair and Taylor, 1997). Data points on a settlement trough that closely resembles the Gaussian curve produce a straight line when the logarithm of the normalised settlement is plotted against the square of the offset from the tunnel centre-line. However, settlement data observed in the centrifuge tests produced a parabolic shape when plotted in this way. It was found that the logarithm of the normalised settlement data were better represented by a linear relationship when plotted against the offset from the tunnel centre-line raised to the power 1.5. This observation led to a modified equation for the settlement trough being proposed:

$$S = S_m \exp\left(-\frac{1}{3}\left(\frac{|x|}{i'}\right)^{1.5}\right) \qquad (1)$$

where S is the settlement at any point, S_m the maximum settlement, x the offset from the tunnel centre-line and i' the distance of the inflection point from the tunnel centre-line. The value of i' is determined from a simple regression analysis of the settlement data, taking S_m as the maximum settlement observed. Note that typical values for the trough width parameter K (defined as i/z_0, with z_0 the depth to the tunnel centre-line) do not hold for i'-values from Eq. (1). Based on the centrifuge settlement data the following expression was found to relate K-values determined from the Gaussian curve to those from Eq. (1).

$$K' \approx 0.85K - 0.12 \qquad (2)$$

where K refers to the Gaussian curve and K' to Eq. (1). In the centrifuge tests K-values typically ranged between 0.48 and 0.58. The validity of Eq. (2) outside this range was not investigated.

Equation (1) fits the recorded settlement data better than the conventional Gaussian curve, as illustrated in Fig. 3(a), where observed settlement data from one of the tests at 1% volume loss are presented. Figure 3(b) shows

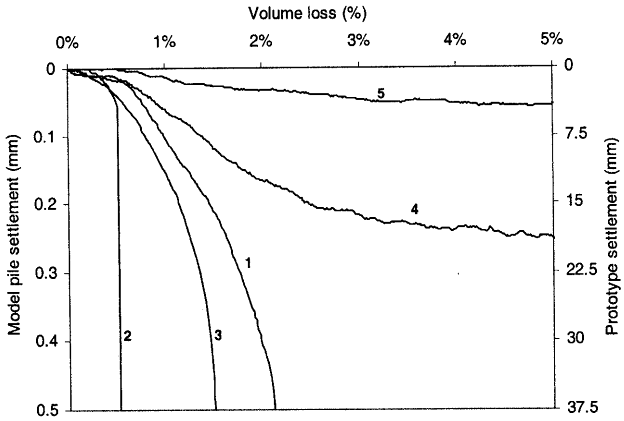

Fig. 4. Pile settlement in response to volume loss (pile positions are shown in Fig. 5)

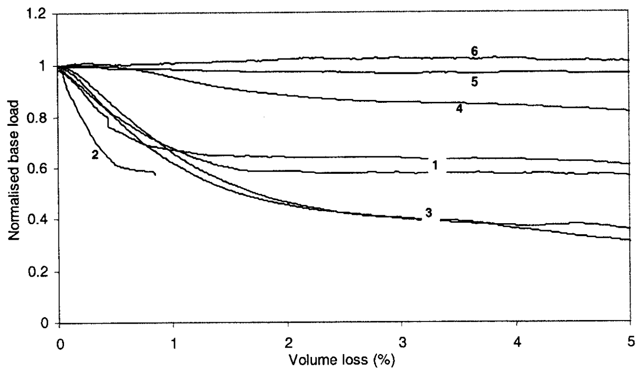

Fig. 6. Normalised pile base loads with volume loss

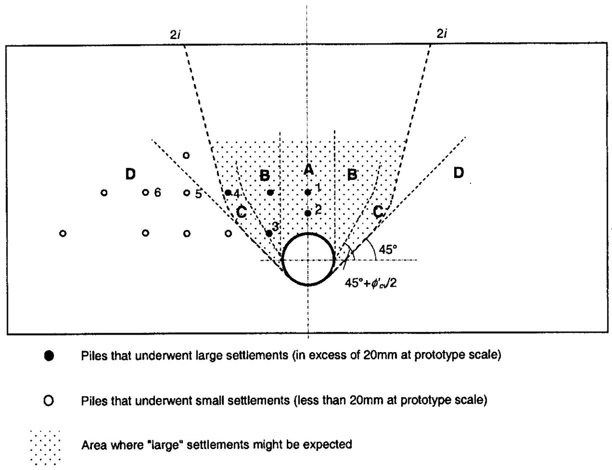

Fig. 5. Zone of influence around tunnel in which potential for large pile settlements exists

that Eq. (1) also fits settlement data obtained from a tunnel in sand (*see* Gioda and Locatelli, 1999) better than the Gaussian curve.

ZONE OF INFLUENCE

A parametric study was carried out with single piles installed at different locations near the tunnel. The positions to which the pile bases were installed during the parametric study are indicated in Fig. 1.

The settlement of piles installed at selected locations near the tunnel, plotted against volume loss, are presented in Fig. 4. The numbers alongside the different curves refer to the locations of the pile toes, which are shown in Fig. 5. More detailed settlement records were presented by Jacobsz (2002). Piles installed near the tunnel initially underwent little settlement, but beyond a certain volume loss large settlements occurred rapidly. Settlements reduced with increasing separation between the pile bases and the tunnel.

An examination of the settlement that piles underwent at various base positions revealed a roughly parabolic-shaped zone of influence in which a potential for large settlements exists at volume losses greater than approximately 1.5% (*see* Fig. 5), with the exception of pile 2 which was very close to the tunnel and settled rapidly at 0.5% volume loss. For the purposes of this discussion "large" settlements arbitrarily refer to settlements in excess of 20 mm at prototype scale. The settlement of piles generally accelerated between about 1% to 1.5% volume loss, with the exception of pile 2, where the critical volume loss was 0.5%. This magnitude of volume loss (i.e. 1.5%) is a typical value often encountered in practice.

The zone of influence can be sub-divided as shown in Fig. 5 according to the amount of settlement that the piles underwent at 1.5% volume loss compared with the surface settlement. Piles with their bases installed in zone D, settled less than the surface. Piles with their bases in zone B, settled more than the surface. In zones A and C the pile and surface settlements were very similar.

A tunnel at considerable depth below a pile base is unlikely to have a significant effect on the behaviour of such a pile, implying that the zone of influence illustrated in Fig. 5 would not necessarily extend to the surface. The upper extent of the zone of influence was not investigated in the tests described in this paper. However, should the boundaries of the zone of influence be extended towards the surface, they appear to intersect the surface at an offset of roughly $2i'$ from the tunnel centre-line.

CHANGES IN PILE LOAD DISTRIBUTION

Base Load

Load cells located at the base of the model piles enabled the pile base loads to be monitored continuously in response to volume loss. Figure 6 presents the base load, normalised by the base load value after the final pile driving prior to imposing volume loss, at volume losses from 0% to 5% for single piles installed at various locations around a tunnel as indicated in Fig. 5. Note that there are two lines shown for pile locations 1 and 3 as repeat tests were performed with piles at these localities. The repeatability of the tests is evident from the closeness of the pairs of lines.

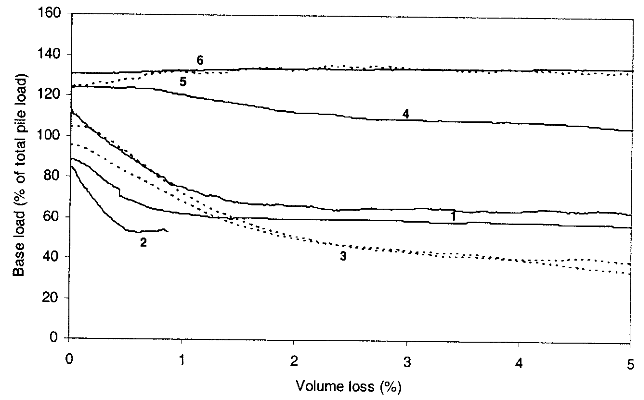

Fig. 7. Percentage of pile load supported by the base with volume loss

Piles 1 to 3, with their bases inside sub-zones A and B of the zone of influence, suffered considerable reductions in base load during volume loss, while pile 4 in sub-zone C suffered a significantly smaller base load reduction. Piles 5 and 6 in zone D registered only very small base load changes. In response to the reduction in base load, positive shaft friction developed on the piles with their bases in the zone of influence. Negative shaft friction developed on piles with bases outside the zone of influence (zone D) as the soil around upper part of the pile shafts settled more than the piles themselves in response to volume loss. This resulted in a small increase in the base load on pile 6.

Zone A

The normalised base loads of two piles, monitored in two separate tests, installed directly above the tunnel centre-line to a depth of 200 mm (56 mm or 0.93*D* above the tunnel crown; position 1 in Fig. 5), are shown in Fig. 6. The base loads reduced rapidly between volume losses of 0% and 1.5%. Beyond 1.5% the base loads remained fairly constant as the pile settlement accelerated rapidly with increasing volume loss. At failure the normalised base loads were about 60%, while the percentages of the *total* pile load (i.e. including shaft friction) carried by the bases were 65% to 70% respectively (*see* Fig. 7, showing the base load as percentage of the total pile load.)

The effect of having the pile base closer to the tunnel is illustrated by the result for the pile installed to a depth of 225 mm (31 mm or 0.52*D* above the tunnel crown; position 2 in Fig. 5). This pile suffered a very rapid reduction in base load as volume loss commenced and settled very rapidly by a large amount at 0.5% volume loss. Although the normalised base load was similar to the piles at position 1 (Fig. 6), the percentage of the *total* pile load carried by the base at failure was 54% (Fig. 7). This is lower than the values for the piles installed to a shallower depth, reflecting the larger load that could be supported by the shaft due to its greater length of shaft embedment into the sand. The test on pile 2 finished at a much lower volume loss than other tests because of excessive pile movement, but, as can be seen, there was sufficient volume loss for the base load to have levelled off.

Zone B

The base load on piles installed at an offset of 50 mm (0.83*D*) from the tunnel centre-line and to a depth of 250 mm (position 3 in Fig. 5) reduced at a similar rate to that of the piles at position 1, as shown by the results from two tests in Fig. 6. Due to the greater depth of these piles, higher loads could be supported by the shafts, so that the amount of base load reduction that they suffered before failure was larger. The percentage of the *total* load supported by the base amounted to about 41% at failure, reflecting the greater pile depth and shaft length. It should be noted that the base loads did not stabilise as in the case of piles in zone A, but continued to reduce with volume loss. This is discussed further in Section Discussion.

Zones C and D

The base load of the pile installed to position 4 reduced more gradually than in zones A and B, so that the full shaft capacity was not mobilised even at a volume loss of 5%. The result is typical of the transition zone between the main zones of influence (zones A and B) and the zone where base load reduction did not occur (zone D).

Shaft Load

A constant service load was applied to each pile during each centrifuge test. In order to maintain equilibrium as the pile base load reduced, load had to be transferred to the pile shaft. The ultimate shaft capacity can be described by the following expression:

$$\tau_s = \sigma_n' \tan \delta \tag{3}$$

where τ_s is the shaft friction, σ_n' the normal effective stress on the pile shaft and δ the interface friction angle. With the sand in a dense state around the shaft, as the pile settles, interface dilation causes an increase in the normal stress on the pile shaft, reaching a maximum value once the constant volume interface friction angle is mobilised (Lehane et al., 1993). The amount of normal stress that can be generated on the pile shaft is therefore limited and the friction angle is essentially a fixed value; as a result the shaft capacity cannot exceed a certain maximum.

In order to mobilise shaft capacity some differential soil-pile movement is required. Only small pile settlements are required to mobilise shaft capacity (Fleming et al., 1985).

The shaft friction loads recorded on the numbered piles in Fig. 5 are presented as a percentage of the total pile load against pile settlement in Fig. 8. The shaft load was taken as the difference between the total pile load applied at the head and the base load. The low or negative shaft friction values on the piles prior to the onset of volume loss can be explained by the rebound that the piles underwent when the pile driving actuators were retracted. Piles with their bases inside the zone of influence suffered less rebound than piles with their bases outside it. This

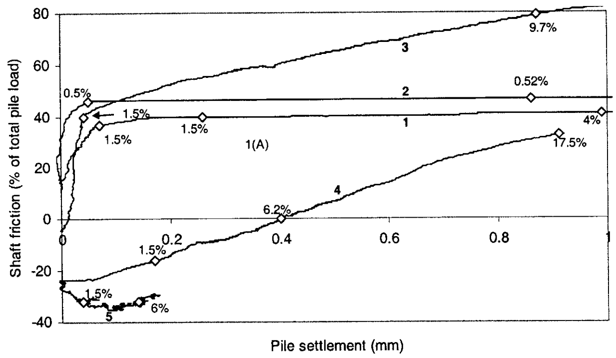

Fig. 8. Mobilisation of shaft load with pile settlement

was the result of the presence of the model tunnel during pile driving, which resulted in some disturbance in the stress regime near the bases of piles in the zone of influence. Negative shaft loads were observed on piles with their bases outside the zone of influence. Larger base loads were recorded during driving of these piles, which resulted in more rebound.

The percentage values marked on the curves in Fig. 8 refer to volume loss. The data show that for piles in zones A and B (pile positions 1, 2 and 3) shaft friction was mobilised during a small amount of pile settlement. Once shaft capacity had been fully mobilised the piles settled rapidly with further volume loss and large settlements occurred. Pile 4 settled gradually as friction was slowly mobilised due to the slow reduction in base load in the transition zone, suggesting that differential movements between the pile and the surrounding soil were small. The shaft friction on pile 5 became more negative as it settled because the soil around it settled more than the pile itself, causing additional down-drag.

DISCUSSION

The processes taking place as a tunnel is constructed beneath a pile are strongly controlled by soil-structure interaction and failure mechanisms within the soil. The research described in this paper has set out to gain a better understanding of these processes.

Immediately after the onset of tunnel pressure reduction, the base loads of piles within the zone of influence began to reduce. As there was a constant load on the piles in the centrifuge tests, base load reduction resulted in a corresponding mobilisation of the pile shaft capacity. When the magnitude of the base load reduction approached the maximum load that could be supported by the pile shaft, equilibrium could not be maintained and the piles started to settle rapidly. By the time that the shaft friction was fully mobilised, the stress level around the pile base had been considerably reduced from tunnel construction, resulting in a significant reduction in the ultimate bearing capacity of the soil supporting the pile base. Significant pile settlement was therefore required to maintain equilibrium.

It is interesting to note that the amount of shaft friction on pile 3 (in zone B of the zone of influence) continued to increase as the pile settled, mirroring the continuous reduction in base load described in Section Base Load. The shaft of pile 3 falls within the zone of most intense shearing propagating towards the surface from the tunnel shoulders and springlines as volume loss increased (*see* for example Cording and Hansmire, 1975). Due to the increased degree of shearing, accompanied by dilation, the normal stress on the pile shaft continued to increase, enabling increasing shaft friction values to be gradually mobilised. This gradual increase in shaft friction was not observed on piles directly above the tunnel (in zone A) where much less shearing, and hence less dilation, is expected.

The low or even negative shaft friction values recorded on the pile shafts prior to volume loss (discussed in Section Shaft Load) resulted in the piles being initially essentially end-bearing. They therefore possessed a significant reserve shaft capacity. If the piles had not been driven (say for example bored piles) and consequently not suffered rebound, much higher shaft friction values would have been mobilised prior to volume loss, resulting in less reserve shaft capacity. It appears reasonable to expect that such piles would suffer large settlements at lower volume losses than discussed in this paper. It is therefore important to have an understanding of the load distribution on piles prior to volume loss in order for the effects of nearby tunnelling to be assessed.

It should be noted that one test was carried out in which one pile above the tunnel axis was driven 50 mm instead of 25 mm to investigate whether driving length would affect the volume loss-settlement behaviour. The effect was found to be negligible.

Mechanisms explaining what happens around the vicinity of the pile toe and the tunnel have not been clearly established at this stage and are difficult to postulate as they depend on a number of factors such as geometry and relative pile to tunnel diameter ratio (Jacobsz, 2002). This remains an area for further investigation.

CONCLUSIONS

The following conclusions are drawn from the centrifuge model study where tests were performed in dense dry sand.

- A zone of influence exists above a tunnel in which there is a potential for large pile settlements due to tunnelling related volume loss. In this context, ''large'' refers to settlements in excess of 20 mm at the prototype scale.
- Tunnelling-related volume loss results in a transfer of load from the pile base to the pile shaft for piles with their bases located within the zone of influence. The rate and amount of load transfer depends on the location of the pile base within the zone of influence.
- Little pile settlement initially occurs as load is transferred to the pile shaft, but once the full pile shaft load has been mobilised large settlements can occur. The onset of large pile settlement may be rapid once a certain amount of volume loss is exceeded.

- The settlement of piles with volume loss presented in this paper applies to essentially end-bearing piles, possessing a significant amount of reserve shaft capacity. Should piles possess little or no reserve shaft capacity, large settlements may occur more rapidly than presented here.

ACKNOWLEDGEMENTS

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REFERENCES

1) Chen, L. T., Poulos, H. G. and Loganathan, N. (1999): Pile responses caused by tunnelling, *J. of Geotech. and Geoenvironmental Engrg.*, ASCE, **125**(3), 207–215.
2) Cording, E. J. and Hansmire, W. H. (1975): Displacement around soft ground tunnels-General report, *Proc. 5th Pan-American Conf. on Soil Mechanics and Foundation Engrg.*, Buenos Aires, Session IV, 571–632.
3) Fleming, W. G. K., Weltman, A. J., Randolph, M. F. and Elson, W. K. (1985): Piling engineering. Surrey University Press and Halsted Press.
4) Gioda, G. and Locatelli, L. (1999): Back analysis of the measurements performed during the excavation of a shallow tunnel in sand, *Int. J. for Numerical and Analytical Methods in Geomechanics*, **23**, 1407–1425.
5) Jacobsz, S. W. (2002): The effects of tunnelling on piled foundations, *PhD thesis*, University of Cambridge.
6) Lee, S. W. (2001): The effects of compensation injections on tunnels, *PhD thesis*, University of Cambridge.
7) Lee, R. G., Turner, A. J. and Whitworth, L. J. (1994): Deformation caused by tunnelling beneath a piled structure. *Proc. 13th ICSMFE*, University Press, London, 873–878.
8) Lehane, B. M., Jardine, R. J., Bond, A. J. and Frank, R. (1993): Mechanisms of shaft friction in sand from instrumented pile tests, *J. of Geotech. Engrg.*, ASCE, **119**(1), 19–35.
9) Loganathan, N. and Poulos, H. G. (1998): Analytical prediction for tunnelling induced ground movements in clays, *J. of Geotech. and Geo-environmental Engrg.*, ASCE, **124**(9), 846–856.
10) Mair, R. J. and Taylor, R. N. (1997): Theme lecture: bored tunnels in the urban environment, *Proc. 14th ICSMFE*, Hamburg, Balkema, 2353–2385.
11) Mroueh, H. and Shahrour, I. (1999): Three dimensional analysis of the interaction between tunnelling and pile foundations. *Numerical Methods in Geomechanics* (eds. by Punde, Pietruszczak and Schweiger), Balkema, 397–402.
12) Schofield, A. N. (1980): Cambridge Geotechnical Centrifuge Operations, *Géotechnique*, **30**(3), 227–268.
13) Vermeer, P. A. and Bonnier, P. G. (1991): Pile settlements due to tunneling, *Proc. 10th European Conf. on Soil Mechanics and Foundation Engrg.*, Florence, Balkema, **2**, 869–872.