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Centrifuge modelling of tunnelling near driven piles

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ABSTRACT: A centrifuge model study was carried out on the Cambridge Geotechnical Centrifuge 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 observed surface settlement profiles are discussed, as well as a zone around the tunnel in which a potential for large settlement exists. Changes in the load distribution on individual piles at various locations during increasing tunnelling related volume loss are presented.

1 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.

A centrifuge model study was carried out on the Cambridge Geotechnical Centrifuge (described by Schofield, 1980) to investigate the effects of tunnelling near driven piles in dense dry sand. Physical modelling, rather than a numerical investigation, was chosen due to 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 tunnel-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 would be most prone to tunnelling-induced settlement. Chen *et al.* (1999) reported small bending moments in piles with their bases located above the tunnel axis. Excessive bending moments in vertical piles designed to carry axial load are unlikely to have a significant effect on the axial load capacity of the piles. Bending of piles due to tunnelling was therefore not investigated in the tests described in this paper.

2 MODEL DESCRIPTION

The centrifuge model is illustrated in Figure 1. The scale of the model was chosen to be 1:75 due to practical considerations, hence all tests were carried out at 75g. The components of the centrifuge model are described below.

2.1 *Strong-box*

The model tunnel, the sand around it and the model piles were contained in an aluminium alloy (Dural, grade HS30-TF) strong-box designed for the project. The box measured 750 x 400 x 470mm deep, equating to 56 x 30 x 35m deep at prototype scale. It was constructed from 16mm thick Dural plates. Two 60mm diameter circular openings in the sides of the strong-box accommodated the model tunnel.

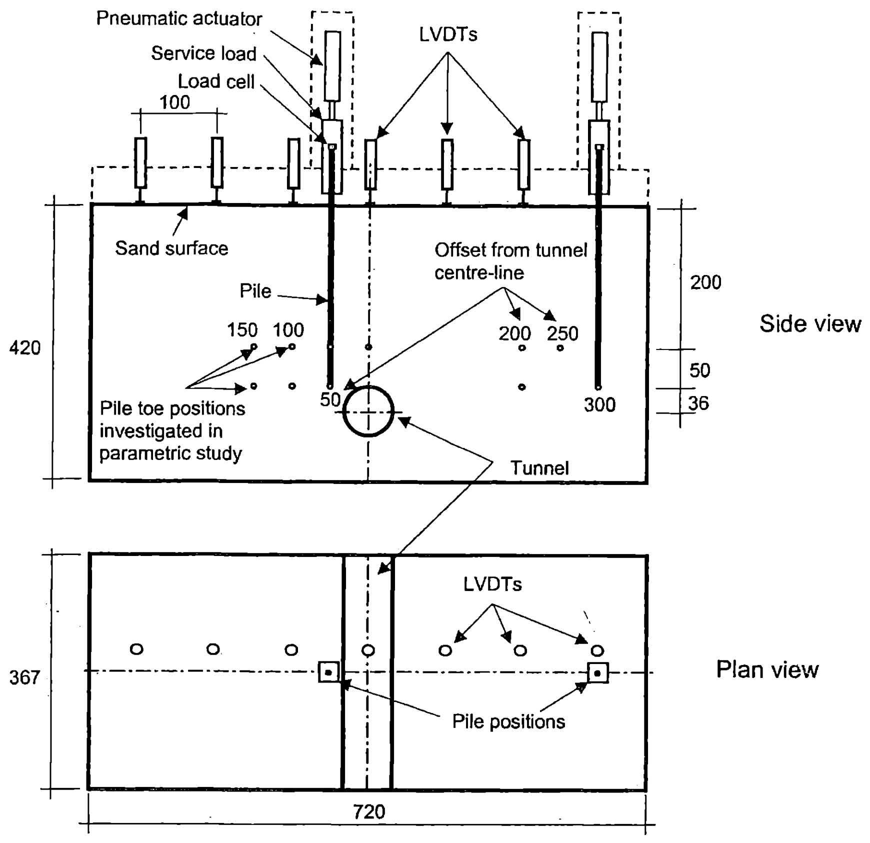

Figure 1. Illustration of the centrifuge model (dimensions in mm).

2.2 Model tunnel

The model tunnel consisted of a brass mandrel surrounded by a 1mm thick latex membrane. During centrifuge tests the 4mm thick annulus between the mandrel and the membrane was filled with water that could be extracted accurately 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 was 60mm, representing a 4.5m 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 automatically balance the tunnel pressure with the overburden pressure during the acceleration of the centrifuge and driving of the piles. After the desired acceleration (75g) had been achieved and the model piles driven, the solenoid valve was closed and volume loss imposed by extracting water slowly from the tunnel.

2.3 Instrumented model piles

Segmental instrumented model piles, machined from aluminium alloy tubing (Dural, grade HE30-TF), were used to examine the effects of volume loss on the axial pile load distribution. The outer diameter of the piles was 12mm over a length of 250mm. The diameter scaled to 900mm at prototype scale and the pile length installed into the sand to 18.75m. The total length of the piles was 360mm. The top 110mm was not machined. Brass weights were slid over this length to exert realistic service loads on the piles. The service loads were sized to exert a load roughly equal to 50% of the penetration resistance on the model piles. Piles installed to a depth of 200mm (see Figure 1) were loaded with 1.4kg service loads, exerting approximately 1kN at 75g and piles installed to 250mm were loaded with 1.7kg loads to exert about 1.25kN at 75g. 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 Figure 2. The axial load cells were cylindrical and joined the pile segments together. They measured 20mm in length with an internal diameter of 9mm. The wall thickness of the instrumented part of the load cells measured 0.75mm. All load cells were internally instrumented

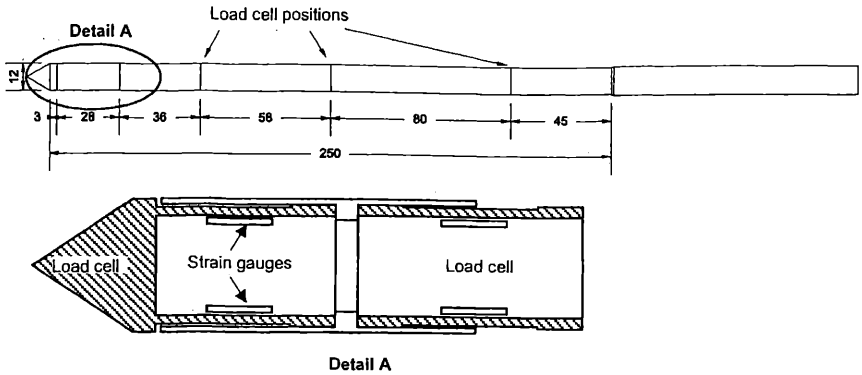

Figure 2. Segmental instrumented model pile with axial load cells

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 superglue and gaps between the segments were filled with silicone rubber to prevent sand ingress during testing. Due to the segmental nature of the piles they were rather flexible with regard to bending.

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

2.4 *Pile driving actuators*

Pneumatic actuators, mounted on support frames, were used to push down onto the weights resting on the piles to drive the piles approximately 25mm in flight (i.e. at 75g), prior to inducing volume loss. The piles were driven by gradually increasing the air pressure in the actuators while monitoring the settlements. After driving, the actuators were retracted to leave only the service loads acting on the piles.

2.5 *Sand*

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

2.6 *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 600mm through a flexible hose before falling an additional 300mm to the sand surface. A fairly uniform sand density was achieved, generally varying around 1560 kg/m^3 (75%±2% of maximum dry 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 25mm above their final depths using a lead-screw assembly, equipped with a rotary bearing to prevent rotation of the piles.

2.7 *Test procedure*

A typical centrifuge test comprised the following steps.

- Acceleration of centrifuge to 75g.
- Driving model piles 25mm to final depth.
- 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.

3 SURFACE SETTLEMENT

Surface settlement was measured with an array of seven LVDTs. 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. This observation had led to a modified equation for the settlement trough being proposed:

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

where S is the settlement at any point, S_m the maximum settlement, x the offset from the tunnel centre-line and i the offset from the tunnel centre-line of the inflection point. This equation fits the recorded settlement data better than the conventional Gaussian curve, as illustrated in Figure 3, where observed settlement data from one of the tests at 1% volume loss are presented.

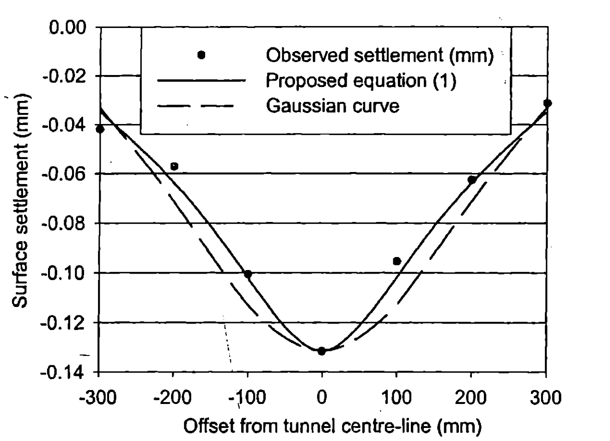

Figure 3. The modified settlement trough equation compared with the "traditional" Gaussian curve.

4 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 Figure 1.

The settlement of piles installed at selected locations near the tunnel, plotted against volume loss, are presented in Figure 4. More detailed settlement records were presented by Jacobsz *et al.* (2001a) and Jacobsz *et al.* (2001b). Piles installed near the tunnel initially underwent little settlement, but beyond a certain volume loss settlement occurred rapidly with large settlements observed. Settlements reduced with increasing separation between the pile bases and the tunnel.

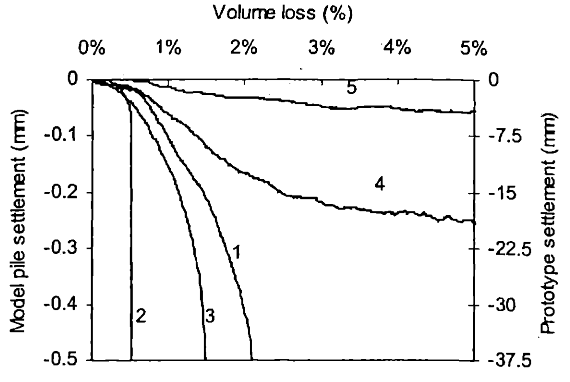

Figure 4. Pile settlement in response to volume loss (pile positions are shown in Figure 5)

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 1.5% (see Figure 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 20mm at prototype scale. A benchmark volume loss of 1.5% was selected because the settlement of piles generally accelerated beyond this value, 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 recorded in practice.

The zone of influence can be sub-divided as shown in Figure 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. These findings are similar to those of the Adviesbureau Noord/Zuidlijn (1999) who conducted a full-scale tunnel construction trial near piled foundations with extensive instrumentation and monitoring.

5 CHANGES IN PILE LOAD DISTRIBUTION

5.1 *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 volume loss, at volume losses from 0% to 5% for single piles installed at various locations around a tunnel as indicated in Figure 5.

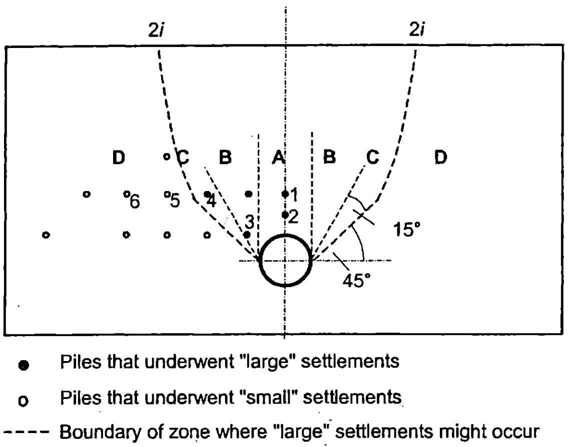

Figure 5. Zone of influence around tunnel in which potential for large pile settlements exists

Piles 1 to 3, with their bases inside sub-zones A & 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 bases in the zone of influence. Negative skin friction developed on piles with bases outside the zone of influence (zone D) as the soil around the pile shaft settled more than the piles themselves in response to volume loss.

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 200mm (56mm or 0.93 tunnel diameters (D) above the tunnel crown; position 1 in Figure 5), are shown in Figure 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 carried by the bases were 65% to 70% respectively.

The effect of having the pile base closer to the tunnel is illustrated by the result for the pile installed to a depth of 225mm (31mm or 0.52D above the tunnel crown; position 2 in Figure 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 (Figure 6), the percentage of the *total* pile load carried by the base at failure was 54%. 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 embedment into the sand.

Zone B

The base load on piles installed at an offset of 50mm (0.83D) from the tunnel centre-line and to a depth of 250mm (position 3 in Figure 5) reduced at a similar rate to that of the piles at position 1, as shown by the results from two tests in Figure 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 (shaft length).

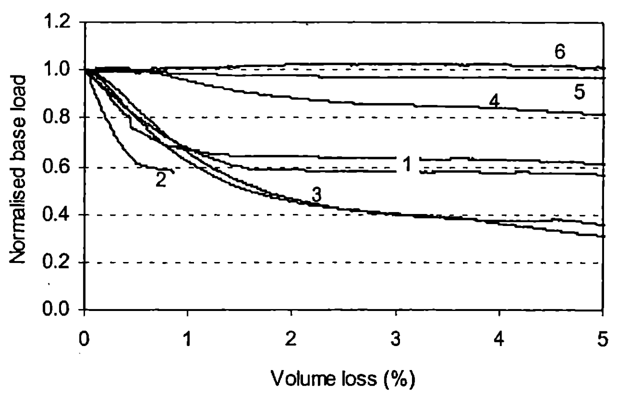

Figure 6. Normalised pile base loads with volume loss

Zone C

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 unaffected zone (zone D).

5.2 *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.

$$\tau_s = \sigma'_n \tan\delta \qquad (2)$$

where τ_s is the shaft friction, σ'_n the normal effective stress on the pile shaft and δ the interface friction angle.

Small pile settlements are required to mobilise shaft capacity (Fleming *et al.*, 1985). As the pile settles, mechanisms such as interface dilation cause an increase in the normal stress on the pile shaft (Lehane *et al.*, 1993). The amount of normal stress that can act on the pile shaft is finite and the friction angle is essentially a fixed value; therefore the shaft capacity is limited to a certain maximum.

When the magnitude of the base load reduction approaches the maximum load that can be supported by the pile shaft, equilibrium cannot be maintained and the piles settle to counteract further base load reduction. By the time that the shaft friction has been fully mobilised the stress level around the pile base has been considerably reduced, resulting in a significant reduction in the ultimate bearing capacity of the soil supporting the pile base. Significant pile settlement may then be required to maintain equilibrium as the pile attempts to maintain the stress level by settling.

Figure 7 presents the shaft loads recorded on the numbered piles in Figure 5 as a percentage of the total pile load against pile settlement. The shaft load was taken as the total pile load minus the base load. The percentage values marked on the curves refer to volume loss. The data show that for piles in zones A and B the pile settled by a small amount as shaft friction was mobilised. 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. The shaft friction on pile 5 became more negative as it settled as the soil around it settled more.

6 CONCLUSIONS

The following conclusions are drawn from the centrifuge model study.

- A zone of influence near a tunnel exists in which there is a potential for large settlements.
- Tunnelling-related volume loss results in a transfer of load from the pile base to the pile shaft. 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.

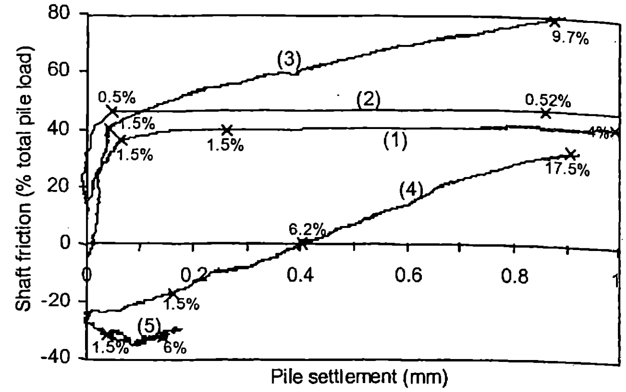

Figure 7. Mobilisation of shaft load with pile settlement

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