ABSTRACT: A series of centrifuge model tests has been undertaken as a preliminary investigation into the influence of spiles placed as reinforcement into a tunnel face. The results show that the presence of spiles both improves the stability of a tunnel heading and reduces associated ground movements.

1 INTRODUCTION

A number of different excavation techniques are available for bored tunnelling in soft ground. Unstable ground conditions often require the face to be supported at all times (closed-face tunnelling). However, whenever possible, it is more convenient to operate with an open face using ground treatment from within the tunnel including sprayed concrete linings, jet-grouting techniques forming pre-lining umbrella arches and pre-cutting.

Another means of face stabilisation is via fibreglass nails or spiles inserted ahead of the face. This type of ground treatment has been used effectively in many cases (Arsena et al. 1991; Grasso et al. 1991; Lunardi et al. 1992; Van Walsum 1992). However, the design of the face reinforcement is often conducted on an empirical basis and, at the moment, little research has been carried out to find out the real working conditions of the spiles (Barley and Graham 1997).

Lunardi et al. (1992) proposed limiting the zone of potentially significant ground movement immediately ahead of the face by means of fibreglass soil nails. Although Barley and Graham (1997) found the more extensible fibreglass nails to be less efficient than steel ones, they have the advantage of being more easily cut during tunnel excavation. Soil nailing is now widely used, particularly in France and Italy (Schlosser and Guilloux 1995; Lunardi et al. 1992), as a short term stabilisation technique for difficult ground conditions and it is sometimes coupled with other techniques such as jet-grouting and mechanical pre-cutting.

The evaluation of the behaviour of different reinforcement layouts (i.e. number of spiles and their location in the face) from reported in situ measurements is difficult because the data available refer to very different ground conditions and excavation methods. Results from numerical models aiming to simulate the ground and the reinforcement system (Peila et al. 1996) can only realistically be compared with the one set of in situ data from the real tunnel to which they refer. Thus, physical models and in particular centrifuge modelling can be very helpful.

2 CENTRIFUGE TESTS

The study was conducted on small scale models using the geotechnical centrifuge facility at City University. Soil behaviour is very much influenced by its stress state and stress history and if a model is to correspond to a prototype at a different scale, then the stresses in the two situations must correspond at homologous points. Centrifuge tests achieve this fundamental feature by increasing the acceleration field to \( N_g \) acting on a 1:\( N \) linear scale model. The arrangement of the geotechnical centrifuge and model used for these tests is shown in Fig.1.

A series of three tunnel heading model tests in kaolin clay was undertaken. The clay sample was prepared in
schematic of the model is shown in Fig.2. Only half of the prototype was modelled because the vertical plane passing through the tunnel axis represents a plane of symmetry for the problem. A semicircular tunnel (D = 50mm) was cut into the front face. Part of the tunnel was supported by a stiff lining while the heading was supported by air pressure within a rubber lining. The heading had a structurally unsupported length P = 0.5D and the clay cover above the tunnel crown was C = 3D. When required, the model spiles (plastic rods of diameter 1mm) were inserted ahead of the tunnel face using a template as a guide.

![Schematic of the model](image)

**Figure 2. Schematic of the model**

The experiment proceeded by first increasing the centrifuge speed to give the test acceleration $N = 125g$ (so that the model corresponded to a prototype tunnel 6.25m in diameter) while at the same time increasing the tunnel air support pressure so that it corresponded to the vertical overburden stress at tunnel axis (372kPa at 125g). The model was left running for sufficient time to come into effective stress equilibrium with a water table near the top ground surface. The test phase could then begin. To simulate tunnelling induced movements the support air pressure was gradually reduced over a relatively short period of about 4 minutes. During this phase images from the various CCTV cameras, viewing both the front and top of the soil model, were grabbed and stored for later analysis.

2.1 Digital Image Processing

During the tests movements of the ground surface and subsurface movements on the vertical plane of symmetry through the longitudinal axis of the tunnel were determined by analysis of images from the CCTV cameras; no other instrumentation was used. The automation advantages offered by digital image measurements and analysis have enhanced the potential and measurement opportunities offered by photogrammetric techniques, by which precise measurements made in the image are mathematically transformed into real displacements (Cooper and Robson 1996). The image analysis system at City University uses techniques of close range photogrammetry to determine movements in centrifuge models (Taylor et al. 1998; Robson et al. 1998). Digital images are stored directly onto a PC hard disk using an on-line frame grabber, with a frame rate of 1Hz for the single camera and 0.5Hz for the three upper surface viewing cameras. In-house software is then used to determine the target positions from the images.

![Digital images from camera](image)

**Figure 3. Front camera view for test 1**

The upper ground surface has movements in three dimensions and at least two cameras are needed to evaluate these. However three cameras were used to give a better measurement accuracy and allow a target’s coordinates to be determined even if it is could not be seen by all of the cameras (e.g. poor light conditions can obscure some areas of the target grid). In Figure 4 images grabbed from the three top cameras are shown. The distortion of the images results from the relatively short focal length of the lenses necessary to obtain a complete view of the model.

The overall precision of the results was determined to be about ±60μm for the front face analysis and about ±100μm for the surface vertical displacements (Taylor et al. 1998).
2.2 Soil Reinforcement

Many methods of tunnel design and construction rely on a plane strain simplification of a tunnel. But to evaluate the effectiveness of longitudinal reinforcement, it is necessary to consider the development of ground movements in three dimensions around a tunnel heading.

In this series of three tests, test 1 had no face reinforcement and tests 2 and 3 used different configurations of nails so that the efficiency of the spiles with respect to their spacing and length, $L$, could be studied.

Fig. 5 shows the patterns of spiles used in the three models. The spiles were pushed in the ground ahead of the tunnel face. They were modelled using 1 mm diameter plastic rods. Sixteen spiles were used for both tests 2 and 3, and thus the ratio between their area and the tunnel cross section was always 1.3%. The two supported tests, however, differ in both the ratio between their length and the diameter of the tunnel ($L/D=1$ for test 2 and $L/D=2$ for test 3) and pattern of installation. In test 3 the spiles were longer, so that they extended into the ground in front of the tunnel face to beyond the zone of significant ground movements, and they were placed within a thin strip at the outer circumference of the face, in order to create a protective shell around the ground to be excavated. This is similar to the strong and continuous zone created by mechanical pre-cutting.

Figure 5. Spile patterns: length and spacing

3 GROUND MOVEMENTS

The aim of the test series was to evaluate the effect of soil nailing in reducing both risk of collapse of a tunnel excavation and movements of ground above the tunnel. Broms and Bennermark (1967) defined the stability ratio $N$ as:

$$N = \frac{\gamma \left( C + \frac{D}{2} \right) - \sigma_T}{s_u}$$

where:
- $\gamma =$ unit weight of the soil
- $C =$ depth of clay cover
- $D =$ tunnel diameter
- $\sigma_T =$ tunnel support pressure
- $s_u =$ undrained soil strength near tunnel

The geometry of the model and the material properties in the three tests were the same. Thus, a reduced risk of collapse would be identified by a lower support pressure at failure.

For open-face tunnelling in clays, a major component of ground movement is deformation towards the face due to stress relief. In London clay,
for example, significant ground movements are observed ahead of the face (Ward 1969; Mair and Taylor 1993). Reinforcing that portion of the ground should both reduce the magnitude and change the distribution of surface and subsurface settlement troughs.

3.1 Subsurface Movements

The observed surface displacements which developed as the tunnel pressure was reduced are shown in Fig. 6. The three curves refer to vertical displacement near the ground surface directly above the tunnel face plotted against support pressure. During all the tests the ground surface showed negligible movements until $\sigma_T$ was about 100kPa. However, further reductions of pressure lead to significant differences between the three tests. The collapse of the heading could be defined by the support pressure at which the displacements most rapidly increased (Mair 1979; Kimura and Mair 1981). The support pressure at failure for the three tests, determined in this way, was equal to 60, 40 and 10kPa respectively. It was found that other face displacement data such as extrusion into the face and convergence of the cavity indicated very similar values of support pressure at failure.

![Figure 6. Support pressure at failure from vertical displacements near ground surface above the tunnel face.](image)

The values of support pressure at failure indicate the effectiveness of this reinforcing technique. The spiles increased the stability of the heading which was reflected in the lower tunnel support pressures at failure.

Fig. 7 shows the overall pattern of subsurface movements in the vertical plane through the longitudinal axis of the tunnel. The crosses indicate the initial coordinates of the targets and the lines show their displacements from the beginning of the tests ($\sigma_T = 372$ kPa) to failure ($\sigma_T = 60$, 40 and 10kPa for tests 1, 2 and 3 respectively). Targets were placed on a regular 10mm grid. In the figures the vectors are magnified by a factor 4, with respect to the grid scale, to show more clearly the patterns of movement. Although the extent of the zone of relatively large surface and subsurface movements do not vary significantly there are some differences in the displacement patterns. In tests 2 and 3 the vectors are slightly more widespread possibly due to strengthening by the spiles of the ground in front of the tunnel face.

![Figure 7. Displacement vectors, magnified by a factor of 4, from beginning of tests to failure.](image)

Figure 8 shows the near surface settlement troughs during the tests for different values of tunnel support pressure. Included in the figure are settlement troughs measured at support pressures deemed to correspond to failure for the three tests. It can be observed that, in spite of a reduction of the final settlement, the behaviour in test 1 and test 2 is similar before failure ($\sigma_T = 60$ and 40kPa respectively) and the order of magnitude of settlements is about the same. In contrast the geometry of spiles used for the second supported
test (test 3) is much more effective in reducing the settlements before failure as well as the final value. The shape of the settlement curves do not vary significantly with the introduction of the spiles.

Figure 8. Vertical displacements at surface for different values of tunnel support pressure

3.2 Surface Movements

Figure 9 shows the soil surface for test 1 computed from the difference in the target coordinates at the beginning of the test and those at failure ($\sigma_T = 60\text{kPa}$) determined by analysis of the measurements from the 3 surface viewing cameras. Despite some background noise in the evaluated settlements, the bowl of depression above the tunnel heading is clearly evident.

Figure 9. Isometric view of ground settlement from beginning of test 1 to failure ($\sigma_T = 60\text{kPa}$)

In Fig.10 ground movements from the beginning of the tests to failure is represented by means of surface contours normalised with respect to the maximum surface settlement. Contour lines are drawn, from 0 to 80% of the maximum settlement, at intervals of 20%. Consistent with Fig. 8, Fig. 10 shows that the centre of the bowl of depression moves slightly from a point directly above the tunnel face in test 1 to a point somewhere ahead of the face in tests 2 and 3. For test 3 the larger settlements (60% and 80% contour lines), undoubtedly the most dangerous for any structure on the ground, are much closer to the tunnel on plan than those for tests 1 and 2. The location and length of the spiles in tests 2 and 3 reduced both ground settlements and the transverse extension of the bowl of depression.

4  CONCLUSIONS

A series of centrifuge model tests has been used to demonstrate the effectiveness of spiles driven into a tunnel face to increase the stability of the heading and reduce the ground movements.

It was found that a more significant reduction in ground movements could by achieved with spiles concentrated near the tunnel periphery and extending beyond the zone of significant movements ahead of the tunnel face. This also had the effect of concentrating the surface bowl of settlement over the tunnel heading.

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(a) test 1, \( \sigma_T = 60\text{kPa} \)

(b) test 2, \( \sigma_T = 40\text{kPa} \)

(c) test 3, \( \sigma_T = 10\text{kPa} \)

Figure 10. Normalised surface settlement contours at failure.

REFERENCES


