Geotechnical Aspects of Underground Construction in Soft Ground, Mair & Taylor (eds)
© 1996 Balkema, Rotterdam. ISBN 90 5410 856 8

Centrifuge modelling of ground movements due to tunnelling in layered ground

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ABSTRACT: Research is described which uses geotechnical centrifuge model tests to investigate ground movements caused by tunnelling in a clay stratum overlain by a layer of coarse grained material. Observations of ground deformations indicate that current methods of prediction underestimate the width of the subsurface settlement troughs.

1 INTRODUCTION

Increasing pressure on land use within urban areas has lead to an escalation in the number of tunnelling projects for services and transportation purposes world-wide. The need for accurate predictions of tunnelling induced ground movements and potential damage to structures has led to growing research interest and resulted in many detailed studies of the problem. However, remarkably little attention has been paid to the development of movements in layered ground. Also, the assessment of subsurface movements has largely been ignored.

Current practice for the prediction of near surface settlement profiles is based on an empirical approach formulated largely as a result of field observations of settlement due to tunnelling in uniform ground conditions. Little is known about ground response to tunnelling in non-uniform or multi-layered ground.

The geology of many urban environments throughout the world consists of a layer of coarse grained material overlying overconsolidated clay. Services and structures are founded at various depths within the soil mass and it is therefore desirable to develop a method of predicting surface and subsurface settlement troughs in ground with more than one soil layer.

Centrifuge testing is a powerful tool for investigating such geotechnical problems. Small scale soil models can be subjected to prototype effective stress distributions. From such testing, a realistic representation of ground movements induced by tunnel excavation can be obtained. A research project is in progress at City University in which centrifuge testing is being used to investigate the development of ground movements caused by tunnelling in layered ground. Recent advances in measurement techniques at City University have provided an opportunity to investigate in detail the pattern of subsurface movements. The results of one of the many tests undertaken will be presented to illustrate the type of results being obtained and the implication of those results in current design practice.

2 BACKGROUND AND DEFINITIONS

Following work by Schmidt and others, Peck (1969) described the distribution of surface settlement profiles above tunnels to be of Gaussian form. Further data from sites in the UK and from work conducted at Cambridge University in the 1970s, co-ordinated by the then Transport and Road Research Laboratory (TRRL), confirmed this. O'Reilly and New (1982) presented a simple approach to predicting surface settlement troughs (above single long tunnels) based on the Gaussian distribution as:

$$S = S_{max} \exp\left(\frac{-x^2}{2i^2}\right) \tag{1}$$

where S is settlement

S_{max} is the maximum settlement at the tunnel centreline

x is the horizontal distance from the tunnel centreline in the transverse direction

i is the distance from the tunnel centreline to the point of inflexion.

The definition is illustrated in Figure 1 for a two layer soil where suffix "s" refers to the sand and suffix "c" refers to the clay.

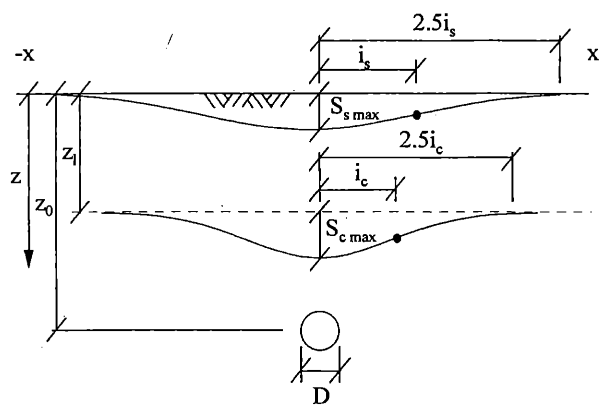

Figure 1. Definition of settlement profiles of Gaussian form.

The settlement distribution defined by equation 1 requires the determination of the values of S_{max} and i. By integrating the Gaussian curve (equation 1), the values of S_{max} and i for a unit length of tunnel can be shown to be related by:

$$S_{max} = \frac{V}{\sqrt{2\pi}.i} \qquad (2)$$

where V is the volume loss per unit length of tunnel. V is commonly considered as the volume of soil lost at the tunnel, which for undrained conditions equates to the volume of the surface settlement trough above. However, it is important to note that volume changes will occur for drained conditions and therefore the volume lost at the tunnel will not equate to the volume of the surface settlement trough where there is a layer of coarse grained material above the tunnel crown.

O'Reilly and New (1982) suggested that the value of i could be related to the depth to the tunnel axis and a parameter K as:

$$i = Kz \qquad (3).$$

They suggested that values of K vary from 0.4 to 0.7 for stiff to soft clays and 0.2 to 0.3 for coarse grained materials. More specifically they stated that a value of K = 0.5 is reasonable for London clay.

It should be noted that the statements above are principally concerned with the surface settlement trough of a single tunnel, of infinite length, being driven through a single layered homogeneous soil.

Selby (1988) conducted numerical analyses of a two layered soil by superimposing the settlement trough from the lower strata, on the basis of it being a free surface, on the bottom of the upper strata. From this he concluded that superposition, based on the values of K given above, was reasonable. This was further stated by New and O'Reilly (1991) who also suggested superposition of settlements caused by adjacent tunnels.

The need to predict subsurface settlements, in particular when considering the damage to structures, has lead to the extrapolation of the results from surface settlement troughs to the subsurface region. Mair et al (1993) presented data which suggested this extrapolation was inappropriate and showed that the subsurface troughs in clay were considerably wider than predicted by this method.

3 CENTRIFUGE TESTS

Centrifuge model testing is a powerful tool for investigating geotechnical problems. With this facility it is possible to create a stress distribution which increases with depth from zero at the model surface, such that stresses correspond at homologous points in the small scale model and the full scale prototype. Soil strength and stiffness is governed by the current state of stress within the soil and the stress history to which it has been subjected and both of these can be taken into account by using careful sample preparation techniques combined with centrifuge modelling.

A series of centrifuge model tests has been conducted at City University to investigate the movements caused by the contraction of a cylindrical cavity in a two layered soil, consisting of coarse grained material overlying clay. The scaling laws applicable to this problem are those of length and time. If the gravity scaling factor for the model is N and the stresses in the soil of the model and the prototype are to be equal, the scale factor for linear dimensions in the model must be 1/N. Time and therefore the dissipation of excess pore pressures, consolidation, is subject to a scale factor of $1/N^2$ in the model. Further details of centrifuge testing, scaling laws and sources of error may be found in Schofield (1980) and Taylor (1995).

A typical model used for the tests is shown in Figure 2. It is representative of the plane strain conditions assumed by much of the analysis described in Section 2.

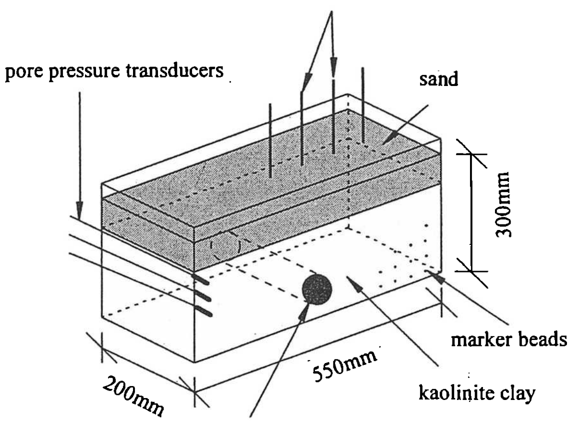

Figure 2. Schematic diagram of centrifuge model

The model resembles those used by Mair (1979) and Taylor (1984) who undertook earlier investigations into aspects of soil behaviour due to tunnelling.

Sample preparation commences by preconsolidating reconstituted kaolinite clay in the plane strain box, usually to a vertical pressure of 500kPa. Consolidation is practically complete after 7 to 10 days when the soil is allowed to swell back under a vertical stress of 250kPa. At this stage miniature pore water pressure transducers are installed through the rear of the container around the region that the tunnel will later occupy. The soil remains at this pressure for a further 4 days to allow excess pore water pressures to dissipate. On the day of the centrifuge test the consolidation pressure is removed from the soil, having first removed any free water from the model container to minimise swelling. The front wall of the container is removed and a cylindrical tunnel cavity cut through the clay which is then lined with a latex rubber membrane. A grid of small black marker beads (3mm diameter) are pushed into the vertical face of the clay to act as targets for the digital image processing described in Section 5. A thick perspex window, highly greased, replaces the front wall of the model container to allow monitoring via CCTV cameras, mounted on the centrifuge arm, whilst the centrifuge is in flight. Sand is rained onto the surface of the clay to produce a medium dense material. Typically, two arrays of 9 displacement transducers (linearly variable differential transformers, LVDTs) are used to monitor vertical movements at the clay/sand interface and at the upper sand surfaces. Perspex sleeves are used to line the holes through the sand layer through which the LVDT probes pass to the clay/sand interface.

The model is then installed on the arm of the centrifuge and the acceleration increased. Compressed air is fed to the tunnel to balance the increasing overburden pressure as the speed of the centrifuge increases. For the tests described here a gravity scaling factor N = 100 has been used. On reaching the required speed the model is left for approximately 16 hours to allow excess pore pressures to dissipate. Monitoring of the movements and pore pressures is carried out throughout this phase using the instrumentation and CCTV cameras via a slip ring arrangement on the centrifuge axis.

In this paper a single test is described which relates to a model in which the total soil cover above the tunnel crown is 3 times the diameter, D, of the tunnel. This soil cover is layered with 1.5D of sand overlying 1.5D of clay. At an acceleration of 100g the model represents a prototype situation of a tunnel 5m in diameter and overlain by a total soil cover of 15m, consisting of 7.5m of clay overlain by 7.5m of sands/gravels, with the water table approximately 2.5m below the ground surface.

When the model has reached equilibrium the test phase can commence.

4 GENERAL MOVEMENTS AT INTERFACES

To simulate tunnelling induced ground movements, the support pressure within the model tunnel is gradually reduced over a period of around 4 minutes. A typical pressure reduction phase is illustrated in Figure 3: the settlements are those measured by the LVDTs.

As anticipated the movements above the centreline of the tunnel are greater at the clay/sand interface than at the upper sand surface.

Normalised transverse settlement troughs, determined from LVDT measurements, for a range of volume losses are plotted in Figure 4a for the clay/sand interface and Figure 4b for the upper sand surface. It should be noted that the measured data

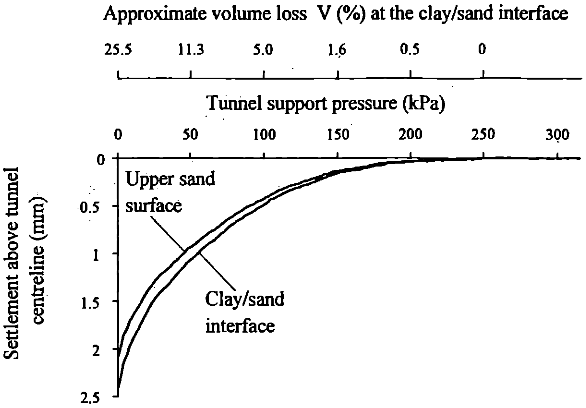

Figure 3. Settlements above tunnel at clay/sand interface and upper sand surface during the test.

for the two figures are for identical stages during the test and the volume losses quoted have been evaluated from measurements at the clay/sand interface. Since volumetric straining can occur in the sand layer, the volume of the settlement trough at the sand surface is unlikely to be identical to that in the clay.

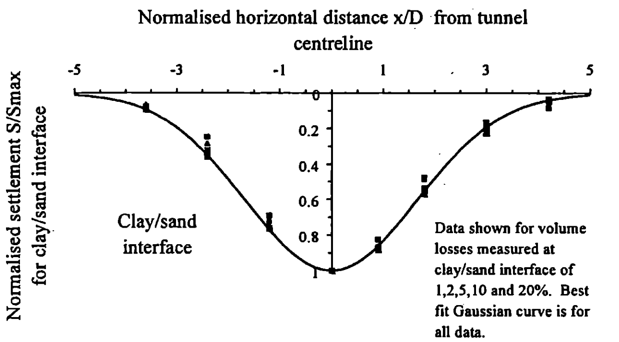

Figure 4a. Normalised troughs at the clay/sand interface.

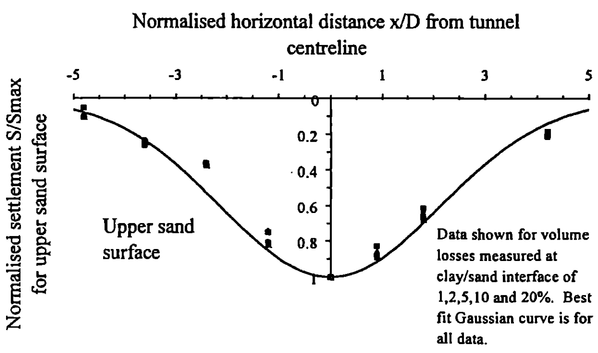

Figure 4b. Normalised troughs at the upper sand surface.

Gaussian distribution curves have been superimposed onto the data. They were produced using the average value of i calculated from LVDT displacement data throughout the entire prefailure period of the test. It can be seen that not only do the data conform well with a Gaussian distribution but also the form of that distribution is almost constant for a wide range of volume losses.

Figure 5 shows the variation in the best fit value of i throughout the test. The degree of scatter at volume losses of less than 1% is considerable but nevertheless it is evident that the values of i at the clay/sand interface and at the upper sand surface stay practically constant, and independent of the degree of support provided, until failure is effectively achieved.

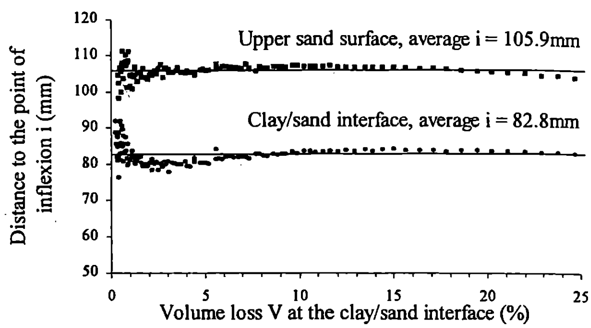

Figure 5. Variation of i during the test.

5 DIGITAL IMAGE PROCESSING

Recent advances in digital image processing techniques at City University has increased the quantity of high quality data that can be obtained from centrifuge model tests. Figure 6 shows a view of the model on the centrifuge swing. The image is taken from the digital record of the test captured from the CCTV camera mounted on the centrifuge arm.

Analysis of the digital images captured during the test produces co-ordinates for the centres of the marker beads seen in the images such as that shown in Figure 6. In order to locate the targets in real space it is necessary to calibrate for lens distortion, camera alignment relative to the model container and distortion due to the perspex window. Details of this and other aspects of the image analysis system are given by Robson et al (1996).

Figure 7 shows the target centroids determined from the image in Figure 6 for the clay layer only,

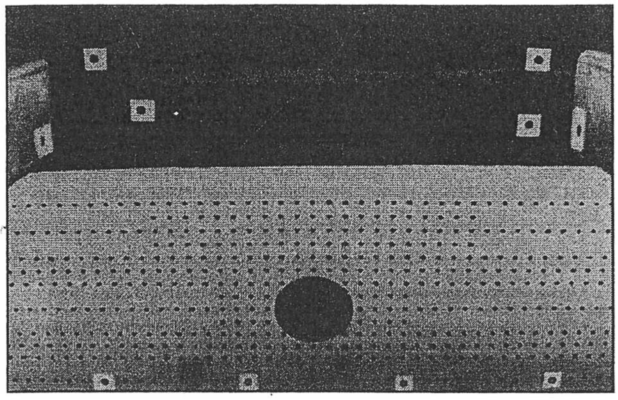

Figure 6. View of the model from the CCTV camera.

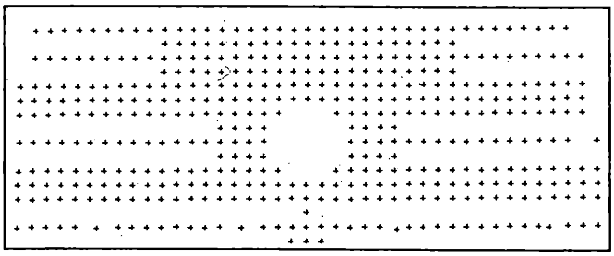

Figure 7. Calibrated positions of targets, in the clay layer only.

and calibrated as described above. The analysis is conducted using software developed in-house.

6 SUBSURFACE MOVEMENTS

The image analysis described above has allowed detailed observations of subsurface movements to be made. Figure 8 shows displacement vectors for the stage corresponding to a volume loss of 12.5% during the centrifuge test. Also, best fit Gaussian distributions are plotted onto three of the planes of movement above the tunnel crown.

Analysis of data at a number of stages during the test has produced the average distribution of i with depth shown in Figure 9.

Also plotted on Figure 9 is the distribution of i assuming commonly used values of K of 0.5 and 0.3 in the clay and sand layers respectively, as discussed in Section 2. It is clear that the test indicates overall a considerably wider zone of influence of tunnel induced movements than suggested by current practice. However, this results largely from the very widespread distribution of movements in the lower

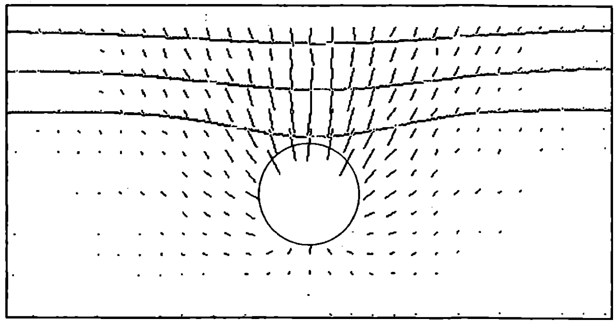

Figure 8. Vectors of subsurface movements, in the clay layer only, corresponding to a volume loss of approximately 12.5%.

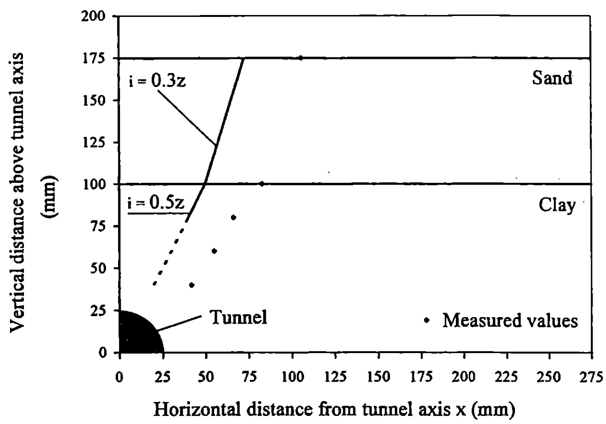

Figure 9. Distribution of i with depth.

clay layer; the relationship between i and z in the sand layer is very similar to the usual design assumptions.

7 DISCUSSION

Mair et al (1993) presented data which indicated that the subsurface distribution of i with depth for tunnels overlain by clay only was wider than the distribution suggested by assuming a constant value of K of 0.5. The field measurements used to determine the values of K of 0.5 for clay and 0.3 for coarse grained material appear to have come entirely from near surface settlement data. The extrapolation to subsurface movements and superposition for ground of more than one soil type is unjustified.

The results presented suggest that a constant value of K of 0.3 for the sand is a reasonable value. However, the overall variation of i with depth depends significantly on the movements throughout the clay layer, and these have been shown to be considerably wider than determined by assuming a constant value of K of 0.5 for the clay stratum. For design calculations, superposition of movements from one layer to another is complicated since it requires prediction of movements in individual layers which are themselves influenced by the properties and behaviour of adjacent layers.

The detailed measurements that can be obtained from a single centrifuge test have been presented. These, combined with data from other tests with different geometries and boundary conditions will lead to a better understanding of subsurface ground movements caused by tunnelling in layered ground, and to the development of procedures for estimating these movements.

ACKNOWLEDGEMENTS

The authors wish to acknowledge the support of the Engineering and Physical Sciences Research Council (EPSRC) for funding the research.

The digital image processing system for the centrifuge is being developed under a separate grant from the EPSRC. The work is being conducted largely by the Engineering Surveying Research Centre and the Centre for Digital Image Measurement and Analysis at City University. Particular thanks are due to Drs. Jin Chen, Stuart Robson and Donning Qu.

The authors are also grateful for the support of colleagues from the Geotechnical Engineering Research Centre at City University, particularly Dr. Sarah Stallebrass, and Messrs. Harvey Skinner and Keith Osbourne.

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