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Numerical modelling of a NATM tunnel construction in London Clay

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ABSTRACT: The finite element program CRItical State Program (CRISP) has been used to model the New Austrian Tunnelling Method (NATM) in London Clay. The non-linear behaviour of the London Clay was modelled by a Strain Dependent Modified Cam Clay (SDMCC) model and the tunnel lining was modelled by constant and time-dependent elastic models. The construction process was modelled in two and three dimensions by removing soil elements in sequence. The tunnel lining was either assumed wished-in-place or introduced after the excavation of each panel. The results obtained from plane strain and three dimensional analyses are compared to assess the importance of arching of soil ahead of tunnel face.

INTRODUCTION

The New Austrian Tunnelling Method (NATM) is a technique in which ground exposed by excavation is lined with shotcrete to form a temporary lining. Rapid and consistent support of freshly excavated ground, easier construction of complex intersections, and lower capital cost of major equipment are some of the advantages of NATM. The successful use of the method is reliant upon high quality working by a skilled work force under continuous engineering supervision. Some of the limitations of this method are that it is slow compared to shield tunnelling in uniform soils, dealing with water ingress can be difficult, and it demands skilled man power. In particular, instability at the tunnel face, unless positive support is provided, can endanger the work force. Kuhnhenn (1995) carefully analyses the typical collapses of NATM tunnels constructed in hard and soft rocks in Germany. He highlights the importance of workmanship and limiting the length of the unsupported section ahead of the shotcrete. Although NATM was primarily developed for rocks, it is now being used in clayey soils. Therefore, it is important to understand this method in clayey soils.

There have been many empirical methods developed to calculate surface settlements due to tunnelling (Schmidt, 1974; Attewell, 1978; O'Reilly and New, 1982; Mair *et al.*, 1993). These methods have the limitations of being specific to soil type and unable to take account of soil-lining interaction. On the other hand the finite element method (FEM) can model all these influences reasonably, if appropriate constitutive models with correct input data are used. Due to limitations both of software and hardware three dimensional (3D) situations are often analysed as though they were two dimensional (2D). A number of 2D FE simplifications have been developed to model 3D tunnels problems, e.g. axi-symmetric case (Rowe and Lee, 1992), cross sectional plane strain (Mair *et al.*, 1981; Rowe and Lee, 1989; Leca and Clough, 1992; Atzl and Mayr, 1994) and longitudinal plane strain analyses (Romo and Diazm, 1980; Guo *et al.*, 1994).

The approximations made to model the 3D construction sequence in each type of 2D analysis to account for the 3D redistribution of stresses around the heading broadly may be classified into three categories:
i) percentage unloading methods (Panet and Guenot, 1982; Allouani *et al.*, 1994), where the lining is introduced after removing a certain percentage of the initial stresses.
ii) volume loss methods (Stallebrass *et al.*, 1994), where the initial stresses are reduced until a given volume loss is achieved, and the rest of the load is left in place.
iii) gap parameter methods (Rowe and Lee, 1992), where the deformation prior to the contact of the lining (hence the surface settlement) is controlled by 'a gap parameter'.

Ground movements due to tunnelling have been analysed in 3D by various researchers (Swoboda *et al.*, 1989; Lee and Rowe, 1990; Lee and Rowe, 1991; Chen and Baldauf, 1994; Akagi, 1994). Swoboda *et al.* (1989) analysed a NATM tunnel in rocks using a rheological model in order to understand the time-dependent interaction between shotcrete and ground displacements. Lee and Rowe (1991) analysed the Thunder Bay tunnel in 3D using the gap parameter method. A gap may be physically meaningful for overbreak produced by tunnel boring machine, or for inward displacements in NATM which occur prior to shotcreting. But the selection of 'a value' for the gap parameter has significant effect on predictions. Akagi (1994) analysed the progressive advance of a shield tunnel in soft ground in 3D. He concludes that ground displacement and pore pressure predictions depend much on changes in the inclination of the shield machine. The general lesson for the analysis of tunnelling is that the actual construction activity should be modelled as closely as possible.

In this paper the aim was to simulate construction of NATM in two and three dimensions without introducing any major arbitrary approximations.

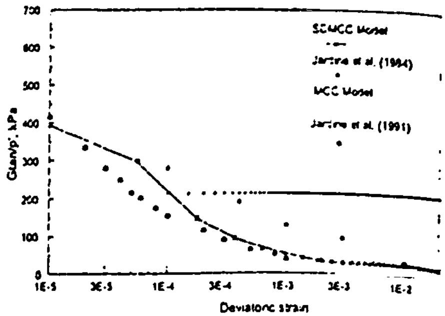

Fig. 1 Shear stiffness-strain variation for London Clay

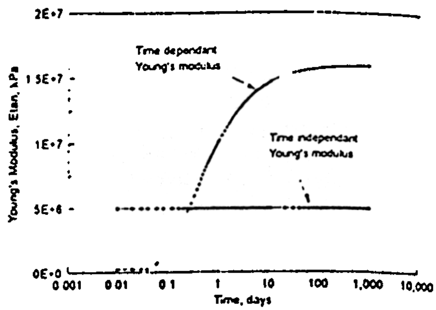

Fig. 2 Assumed Young's modulus-time relation for shotcrete

2. FE PROGRAM, CONSTITUTIVE MODELS AND PROBLEM DEFINITION

A general purpose finite element program [CRItical State Programs (CRISP)-developed at Cambridge, Britto and Gunn (1987)] which can perform 2D and 3D geotechnical analyses was used. Pre and post processing was carried out using FEMGEN/ FEMVIEW (Femsys, 1995).

Recent research (Jardine *et al.*, 1986; Simpson, 1992; Bolton *et al.*, 1993) has shown that stiffness-strain variation is important in analysing any boundary value problem in overconsolidated clays. The non-linear stiffness variation of the London Clay has been modelled by a Strain Dependent Modified Cam Clay (SDMCC) which has been incorporated into CRISP (Bolton *et al.*, 1994). The variation of shear and bulk stiffnesses in the SDMCC model were approximated by power functions (Bolton *et al.*, 1994). Fig. 1 gives the shear stiffness-strain variation predicted by the SDMCC model compared with experimental data, Jardine *et al.* (1984), Jardine *et al.* (1991) and Hight and Higgins (1994). The shotcrete has been modelled as linear elastic and with either constant or time-dependent stiffness (Fig. 2). The time dependency of the shotcrete has been modelled using a non-linear equation based on the data of Fischnaller (1992).

The problem considered here is the analysis of a NATM tunnel. The mesh adopted for 2D analysis is shown in Fig. 3, which is a cross section of the 3D geometry shown in Fig. 4. In the 2D mesh about 90 consolidating linear strain quadrilaterals were used. In the 3D mesh about 1500 consolidating 20 noded linear strain brick elements were used. Each analysis was undrained. The tunnel was assumed to be 8m in diameter with a cover of 21m. A 50m wide and 50m deep section was chosen for the analysis. This mesh approximately represents one half of the Heathrow trial tunnel Type 2 (Deane and Bassett,1995). The actual tunnel construction technique was not symmetrical but symmetry was assumed here mainly to compare 2D and 3D results. Soil properties were assumed to be representative of London Clay. Typical shotcrete properties were chosen for use in the modelling. The in-situ stress state assumed is given in Fig. 5, and the water table was assumed to be at ground level.

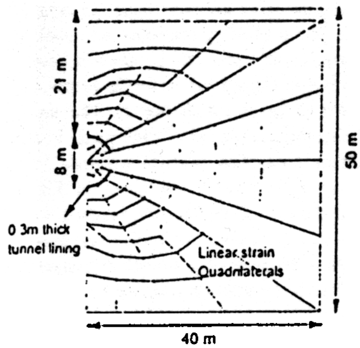

Fig. 3 Mesh adopted for 2D analysis

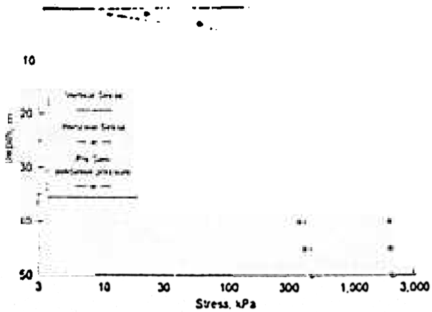

Fig. 5 In-situ effective stress state for London Clay

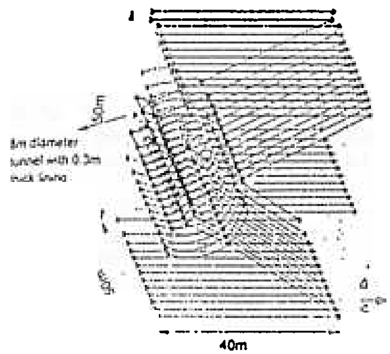

Fig. 4 Geometry for 3D analysis

3. TWO DIMENSIONAL MODELLING

A cross section of the NATM tunnel was analysed in a plane strain mode. The CRISP analysis started from the in-situ stress state shown in Fig. 5. Two types of construction techniques were modelled:

(a) the lining was assumed to be wished-in-place
(b) the lining was constructed sequentially as
   i) excavate top half of panel
   ii) install lining of top half
   iii) excavate bottom half of panel
   iv) install lining of bottom half

The findings from these two idealised cases will bracket the real construction process.

3.1 Effect of Construction Sequence

The settlement profiles obtained following the two construction simulations are shown in Fig. 6. The actual measurements in the Heathrow Trial Tunnel Type 2 (New and Bowers, 1994) lie within these two extremes. Observations are closer to the wished-in-place simulation case (a), although the sequential construction simulation in case (b) might have been expected to be closer. It is suggested that the wished-in-place lining is closer to reality because 3D arching in the field is approximately simulated in 2D by having the lining already in place. The support provided by a wished-in-place lining is obviously more than the 3D arching, therefore the predicted settlements are smaller than the observations. A 2D lining placed after excavation gives larger settlements because of its inability to model arching ahead of the tunnel face.

The computed settlements extend much further and gave a flatter trough than the site measurements. Though the settlement curve predicted by non-linear models like the SDMCC is deeper and narrower than linear elastic models, field observations are found to

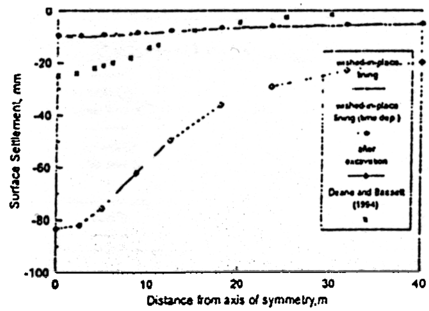

Fig. 6 Effect of construction sequence on surface settlements

be deeper and narrower still. The reasons for this are not fully known, but the following may be possible explanations:
i) uncertainties associated with small strain stiffness measurement (Fig. 1)
ii) effect of anisotropy of soil (Rowe and Lee, 1989)
iii) effect of recent stress history (Bolton *et al.*, 1994; Stallebrass *et al.*, 1994).

3.2 Stiffness of Shotcrete

There are recent developments in shotcrete technology to get higher stiffness/strength. But the usefulness of these high strength concretes for tunnel lining is uncertain. A few analyses were conducted to find out the effect of dependency of settlements on shotcrete stiffness. The shotcrete is modelled as linear elastic with a Young's modulus of 5 x 10^6 kPa, 50 times stiffer, and 50 times softer. Another analysis was carried out with the time dependent stiffness shown in Fig. 2. It is assumed that shotcrete is wished-in-place, as this case is the most affected by stiffness of shotcrete. The results in Fig. 7 show that there is a significant reduction of ground movements due to an initial increase in stiffness, after which there seems to be little effect. Therefore, there is an optimum value of the Young's modulus of lining for a given soil. Time dependence of stiffness does not have much influence as it crosses the optimum value 5×10^6 kPa value in a short time. Figs. 6 & 7 suggest that it is the early placing and hardening of the shotcrete which reduces surface settlements rather than the long-term stiffness.

4. THREE DIMENSIONAL MODELLING

A 3D analysis was carried out with the same material parameters, the same soil model and the same program as in 2D. The modelling sequence was similar to the 2D analysis of sequential construction, and excavation was carried out in the Z-direction (Fig. 4) up to 40m (5 times the diameter). The lining was placed after the excavation of each panel in segments. The surface settlement profiles obtained in the 3D analysis at the section-1 are given in Fig. 8. The surface settlement at this section reached its ultimate plane strain condition after the excavation had progressed a distance of 2 tunnel diameters past the section. The surface settlement due to excavation of the section-1 was about 14 mm and the final settlement was about 29 mm. This means that 42% of surface settlement was due to excavation of that section and the remaining 58% of settlement occurs after the excavation has passed.

The surface settlements obtained at Section-2, 8 m (1 tunnel diameter) inside the mesh are shown in Fig. 9. From the results it can be observed that again the plane strain condition at this section was reached after the excavation had progressed 2 tunnel diameters from this section. The ultimate displacement which

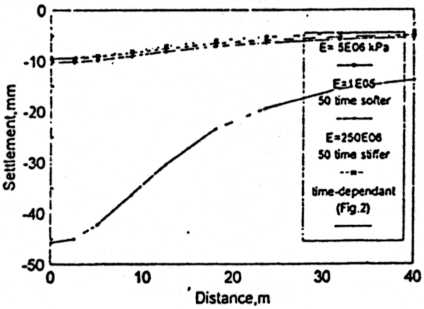

Fig. 7 Effect of shotcrete stiffness on settlement profiles

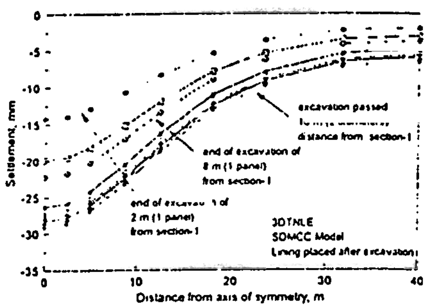

Fig. 8 The surface settlement profiles at the cross section-1

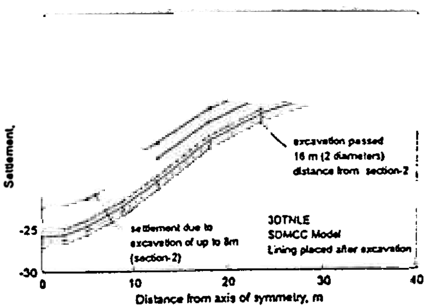

Fig. 9 The surface settlement profiles at Section-2

occurred at the section-1 is more than the section-2, this is due to the support of lining on one side and arching of the soil on the other side.

The ultimate surface settlements obtained in 2D and 3D analysis are compared in Fig. 10. The surface settlements obtained in 2D plane strain analysis of sequential construction are about 3 times greater than the corresponding 3D analysis.

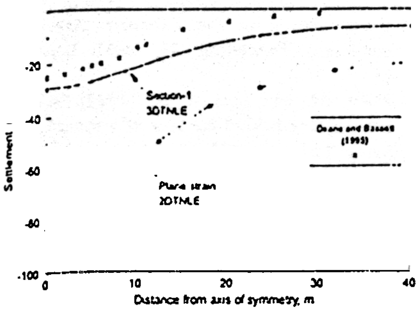

Fig. 10 Ultimate surface settlement obtained in 2D and 3D analyses together with field observations.

5 DISCUSSION AND CONCLUSIONS

A review of existing methods of analysis of tunnels in the light of NATM has been carried out. The analysis of NATM is a complex problem to analyse by any single method and predictions depend to a large extent on the assumptions made in modelling. Because of these complexities FEM may not be used for the direct design of NATM tunnels. However, the FEM is an extremely powerful analysis tool, and its ability to predict true engineering performance is dependent upon the quality of the calibration exercises undertaken against trial tunnels or high quality case records (AGS, 1994).

NATM has been analysed under 2D and 3D conditions. It was shown that the construction sequence has a significant effect on the predicted ground movements. Analyses carried out to study the effect of varying the stiffness of shotcrete indicate that there is an upper limit to the useful stiffness (or lining thickness) for a given soil. In order to reduce movements it is the early placement of the shotcrete which is more important than the eventual stiffness.

If an unlined elastic tunnel is analysed as 2D plane strain and progressive construction in 3D, then the ultimate settlements reached in both cases are the same. However, if a lined tunnel is analysed in 2D and 3D using a non-linear soil model, as in the present study, the results will differ, because of the interaction between the soil and the tunnel lining. The introduction of lining elements at a section restricts further deformation in a 3D analysis. This is important because it is wrongly thought that ultimate conditions due to tunnelling can be obtained with a 2D analysis (Allouani *et al.*, 1994).

The present study demonstrates that the ultimate conditions reached in both cases differ by a factor of three in terms of settlements (Fig. 10) for typical properties of London Clay. This explains the reasons for reduction of only 34% of nodal forces to obtain observed settlements in an approximate 2D plane strain analysis e.g. Stallebrass *et al.* (1994).

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