軟弱地盤에서의 近接터널 사이의 相互擧動에 대한 硏究
Interaction Between Closely Spaced Tunnels in Soft Ground

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개요(SYNOPSIS) : 이 연구는 旣存터널과 近接하여 새로운 터널을 施工할 境遇 이에 따른 터널의 相互擧動에 대한 硏究이다. 특히 이 硏究는 軟弱地盤내에서의 近接터널 施工에 따른 旣存터널 라이닝의 變位및 應力 影響에 대하여 遂行되었다. 이와 같은 터널擧動을 硏究하기 爲하여 試驗室 模型試驗을 遂行하였다. 이 試驗 結果는 터널의 相互 擧動에 對한 보다 正確한 結果를 얻기 爲한 시뮬레이숀 技法 開發을 위하여 有限要素技法을 利用하여 數値解析을 遂行하여 比較 分析 하였다. 硏究結果로 부터 터널의 中心軸을 基準으로 두 터널 사이의 距離가 터널直徑의 1.5배 以上이 될 境遇 터널사이의 相互擧動은 微細한 것으로 나타났으며, 터널掘鑿에 對한 시뮬레이숀에 있어서 Hydrostatic gap modelling은 試驗結果와 잘 一致함을 보여 주었다.

1 INTRODUCTION

In recent years many new tunnels have been designed or constructed in urban areas in order to develop or extend underground transportation systems. In the design of new tunnels, it is important to ensure that any existing underground transportation systems in close proximity to the proposed tunnelling activities can continue to operate safely both during, and after, construction. The stresses and displacements in existing tunnel liners may be affected by the new tunnelling work when the distance between the tunnels is small. The influence of new tunnelling activities on nearby existing tunnels depends on various features of the problem including the magnitude of the in-situ stresses in the ground, the pillar width, the liner stiffness, and the method used to install the tunnel.

A certain amount of information about the interaction that occurs between closely spaced tunnels is given by various reported field measurements and numerical studies. Terzaghi (1942) and Ward and Thomas (1965), for example, reported a set of field measurements made on tunnels constructed with a centre-line spacing of 1.425 tunnel diameters in Chicago Clay and 1.6 tunnel diameters in London Clay respectively. In both cases, the two tunnels were installed consecutively. The measurements indicated that significant liner deformations occurred in the first of the tunnels to be installed as the second tunnel was constructed. The maximum radial displacements, expressed as a percentage of tunnel radius, were measured to be 0.10 % and 0.12 % respectively. Deere *et al.* (1969) give an excellent summary of the available field data prior to 1969. In most cases, measurements were made as a second tunnel was driven past a test section.

Typical numerical analyses of this interaction problem were described by Ghaboussi and Ranken (1977) and Leca (1989). In these studies, a variety of tunnel spacings and procedures to model tunnel construction were adopted. In both cases a two-dimensional approach was used in which the soil model was elastic. The results indicated that, for the configurations investigated, the computed interactions between two parallel tunnels were small when the centre-line spacing was greater than about two tunnel diameters. Addenbrooke and Potts (1996) report nemerical analyses of the interaction between two tunnels constructed within a month of each other. These analyses were based on a small strain non-linear soil model. They concluded that the interaction between two adjacent tunnels depends on relative tunnel position (to the side or vertically above) as well as spacing. Driving a new tunnel above an existing tunnel was shown to have significantly less influence on the existing tunnel lining than was the case for equivalent side-by-side tunnels.

In this paper, the influence of shield tunnel construction on the moments and displacements induced in the linings of existing nearby tunnels are studied by a set of carefully controlled physical model tests. In these tests the tunnels were installed using a miniature shield tunnelling machine. The tests were supplemented by a limited amount of two-dimensional finite element analysis. These computations are based on a similar approach to that described by Ghaboussi and Ranken (1977), with the exception that special numerical procedures were used to model the ground loss associated with tunnel installation.

2 PHYSICAL MODEL TESTS

A set of five tests have been carried out using samples of kaolin clay consolidated in the plane strain tank shown in Figure 1. Three tunnels were installed in each of the tests in order to carry out two interaction experiments in one clay sample. One of these experiments was for a 'distant' tunnel (centre-line spacing of 2.0D) and the other was for a 'close' tunnel (centre-line spacing of 1.4D) where D is the tunnel diameter.

The plane strain rig consisted of a rectangular tank of internal dimensions 1000 mm by 300 mm in plan and 600 mm in height. The two 25 mm thick perspex walls contained three holes (corresponding to the positions at which the model tunnels were installed).

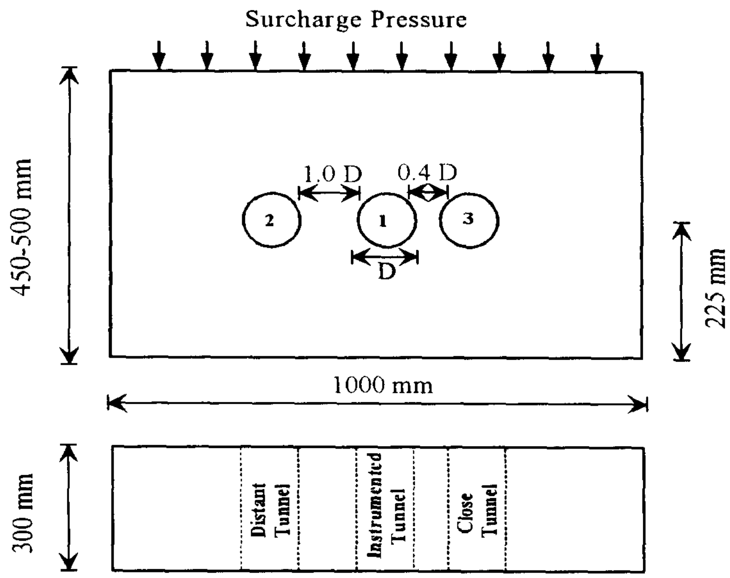


Figure 1. Layout of Plane Strain Tank

During this research project, a set of additional tests were carried out in which the tunnels were perpendicular rather than parallel. Discussion of these tests, however, is beyond the scope of the paper. Details of these tests are given by Kim (1994, 1996).

The geometry and soil properties of each of the plane strain tests are specified in Table 1.

Table 1. Specification of the tests

Tests	H/D	su (kPa)	OCR	t(mm)	Ps(kPa)
PS1	3.36	23.4	1.00	0.254	88.5
PS2	3.79	24.3	2.90	0.254	38.4
PS3	3.94	20.7	1.00	0.356	88.5
PS3R	3.79	21.6	1.00	0.356	88.5
PS4	3.65	24.3	2.81	0.356	41.3

where s_u = Undrained shear strength
Ps = Surcharge pressure
t = Liner thickness

The clay samples were obtained by consolidating a kaolin slurry within the test rig itself. The clay samples used in the tests were either normally consolidated or overconsolidated with a value of OCR of approximately 3. This value of OCR was chosen to be broadly representative of conditions in London Clay. All samples used in the tests had approximately the same shear strength (about 20 kPa). The upper boundary of the sample was stress controlled in order to simulate the behaviour of a tunnel that is distant from the ground surface.

The tunnel liners consisted of plain steel tubes of diameter 70 mm. The tube thicknesses were chosen to ensure correct scaling of the tunnel lining stiffness based on the flexibility ratio proposed by Peck *et al.* (1972). Two thicknesses of liner were used in the tests to model liners of different stiffness.

At the start of each test, a tunnel liner was installed at position 1 (see Figure 1). This liner was instrumented with eight strain gauges on the outside of the tube to allow estimates of the bending moments in the liner to be made. Total stresses and pore-pressures were also measured using four miniature pressure and two pore-water pressure transducers mounted on the tube.

After swelling and consolidation processes associated with the tunnel installation were complete, a second, non-instrumented, tunnel was installed in position 2 (see Figure 1). At this stage, the radial displacements of the instrumented tunnel were measured using a specially constructed device based on the use of LVDTs to measure radial movement. After sufficient time had passed for the pore-pressures to dissipate, a third, non-instrumented, tunnel was installed in position 3.

During these tests, data were collected from the various instruments during the installation of the tunnels in positions 2 and 3. This paper is concerned only with the undrained response, however, and so a discussion is only given of data obtained immediately after tunnel construction.

All tunnels were installed using a model tunnelling machine that was intended to simulate the construction of a full scale shield tunnel. This tunnelling machine was designed to produce approximately 6 percent overcut. This relatively large amount of overcut was chosen to ensure that the experiments modelled the worst case that might be expected in practice. The model tunnelling machine used in the tests is illustrated in Figure 2.

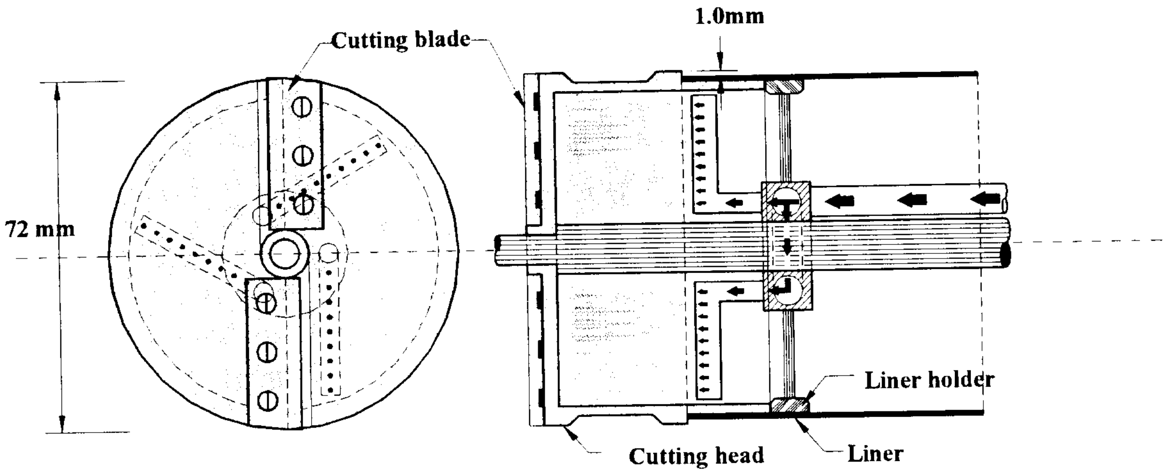


Figure 2. Model Tunnelling Machine

3 NUMERICAL ANALYSIS

The mechanics of the interaction between adjacent tunnels is complex and cannot be fully studied using physical model testing because of the large number of parameters involved. A parametric study could be carried out using finite element analysis but, in order to have confidence in the results, it is first necessary to develop realistic modelling procedures. A limited amount of finite element analysis has been carried out in order to develop some preliminary conclusions as to how this soil-structure interaction problem might best be modelled. The finite element analysis has also been used to investigate some of the modelling assumptions inherent in the physical tests. Further studies could be carried out to investigate possible scaling errors or boundary effects although this aspect is beyond the scope of the paper.

The analysis is limited to the case of the undrained condition to model interaction behaviour immediately after tunnel construction. The analyses were all performed using a linear elastic soil model. Six-noded continuum elements and three-noded beam elements were used to model the ground and the tunnel liners respectively. A typical finite element mesh used in these analyses is shown in Figure 3. (Note that the beam elements were based on a formulation that required them to be straight.)

In these calculations, the value of shear modulus (G) was estimated using the chart given by Duncan and Buchigani (1976) in which G is correlated with values of undrained shear strength (s_u), overconsolidation ratio (OCR) and plasticity index (PI). In each analysis, the appropriate values of s_u and OCR given in Table 1 were adopted in conjunction with a plasticity index of 31 %, which is a typical value for the kaolin clay used in the model tests (Smith, 1993).

The excavation of the tunnel was simulated by numerical removal of the soil elements inside the tunnel (Augarde *et al.*,1995). In these analyses, it was necessary to model the gap between the soil and liner associated with the overcutting. This was achieved by applying a suitable hydrostatic suction to the inside of the tunnel liner in order to reduce its circumference. This procedure would be expected to produce spurious values of hoop force, but this is not significant for the calculations reported here.

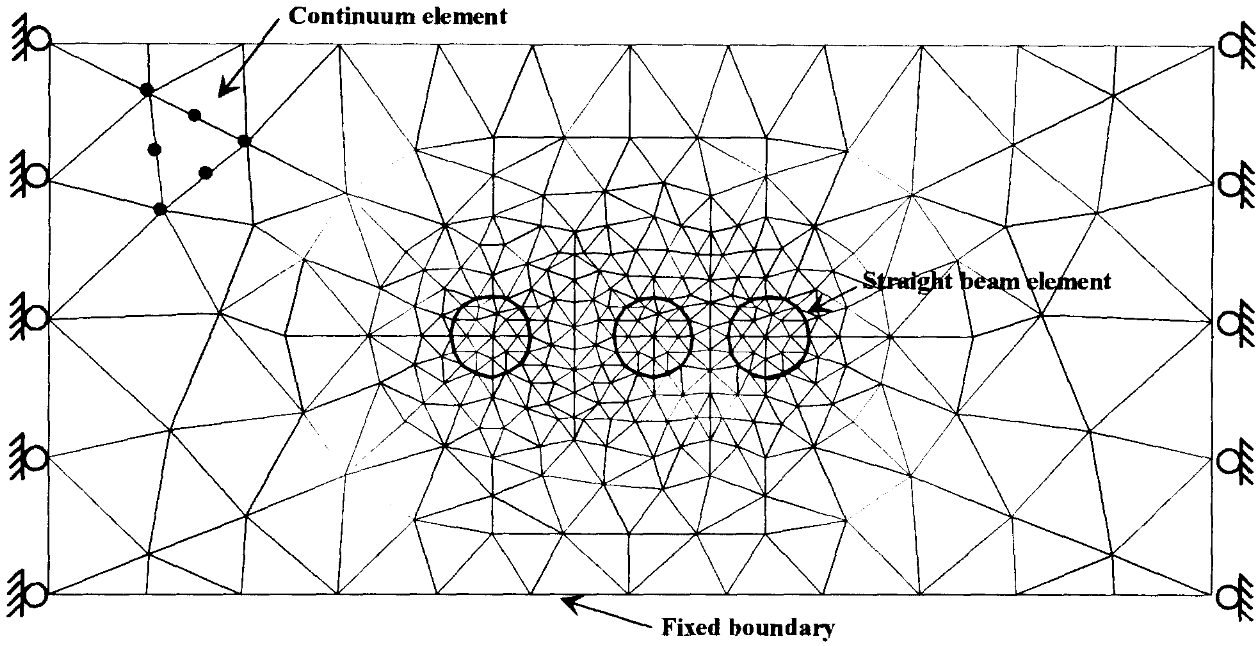


Figure3. Finite Element Mesh

The following modelling procedures were used for the construction of each tunnel in the analyses.

General = Apply initial stresses to soil with beam elements switched off. The value of Ko was assumed to be given by the expression $0.64(OCR)^{0.5}$.
Step 1 = Remove excavated elements within tunnel.
Step 2 = Switch on beam elements to model tunnel liner in tunnel position to be installed.
Step 3 = Apply internal pressure to simulate ground loss.

Four separate calculation procedures were adopted as given below.

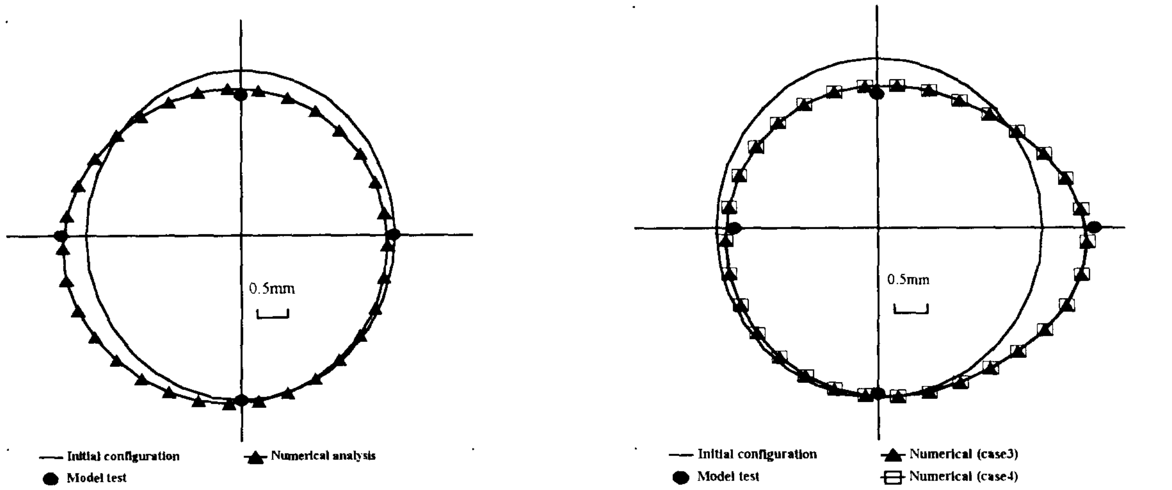
Case 1 = Single tunnel construction in position 1.
Case 2 = Tunnel construction in position 2 after installation of tunnel in position 1 (Two parallel tunnels).
Case 3 = Tunnel construction in position 3 after tunnels installation in positions 1 and 2 (Three parallel tunnels).
Case 4 = Tunnel construction in position 3 after tunnel installation in position 1.

Cases 1,2 and 3 were intended to represent the procedures adopted in the physical model tests. Case 4 was studied in order to investigate the possibility of interactions between tunnels installed in positions 2 and 3 by comparing the results of Cases 3 and 4.

4 RESULTS AND DISCUSSION

The results presented here are limited to the comparison between the results of test PS3 and the corresponding finite element back-analysis.

The results of the physical model tests generally indicate that interaction effects are greatest on the pillar springline and crown of the instrumented tunnel. It was also generally observed that the total stresses acting on the instrumented tunnel decreased immediately after a non-instrumented tunnel had been installed. These stresses tended to increase in the long term as pore-pressure dissipation occurred.



[a] Tunnel installation at position 2 (W =2.0 D) **[b] Tunnel installation at position 3 (W =1.4 D)**

Figure 4. Additional Displacement of the Existing Tunnel Liner for PS3

Figure 4 [a] and [b] show typical results of displacements of the instrumented liner due to the installation of two new tunnels in positions 2 and 3 for the physical model test and the finite element analysis. These results show that the data obtained from physical model tests are in good agreement with the results obtained from numerical analysis. Both sets of results show that large outward displacements of the pillar springline occurred when a new tunnel was installed. In each case the crown of the tunnel displaced downwards; movement of the invert, however, was negligible. The general pattern of displacement for tunnels spaced at separation of 2.0D and 1.4D is similar although the magnitudes are greater for the closer tunnel. The results obtained from the finite element analysis for cases 3 and 4 are similar which suggests that the tunnels placed in positions 2 and 3 do not interact significantly. This result suggests that the approach adopted in the model tests of using one clay sample to perform two interaction experiments is a reasonable one.

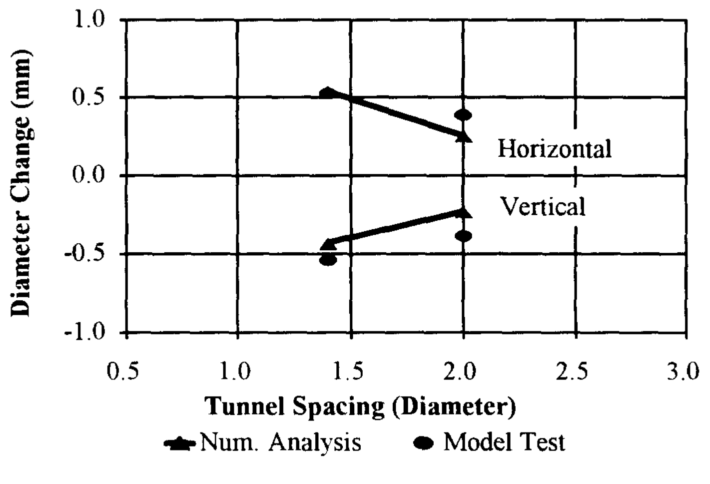
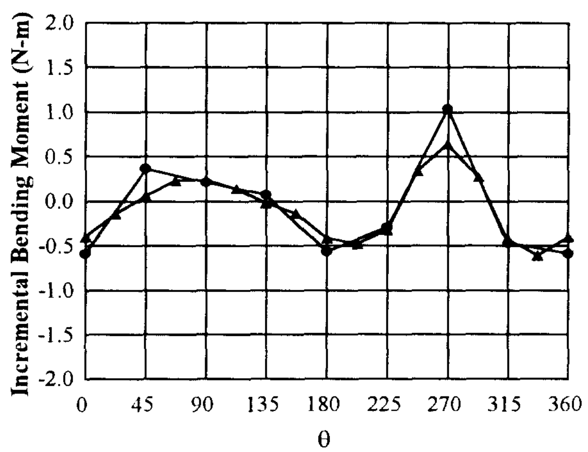


Figure 5. Diameter Change against Tunnel Spacing for PS3

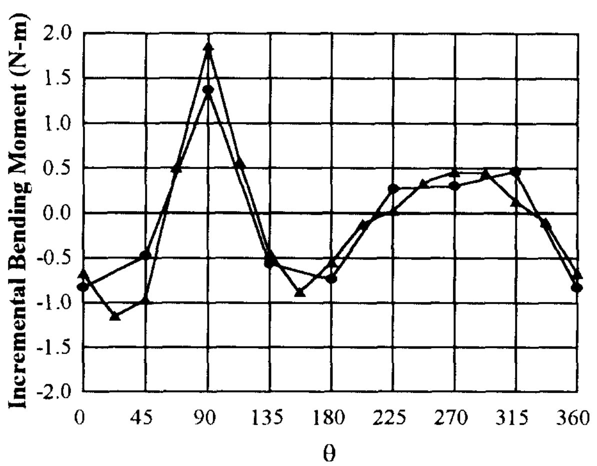
The tunnel distortions can be specified conveniently by two values of diameter change; horizontal diameter change at springline level and vertical diameter change. The diameter changes of the existing tunnel immediately after the construction of the new tunnels plotting against the tunnel spacing are shown in Figure 5. This figure also includes the results of the numerical analysis.

Figure 5 indicates good agreement between the physical and numerical results. These data would suggest, by extrapolation, that interaction effects would be expected to be small for values of tunnel spacing in excess of about 2.5 D.

Figure 6 [a] and [b] show the incremental bending moments induced in the instrumented liner due to the installation of the second and third tunnels. In these plots M is the incremental moment developed in the instrumented tunnel after the additional tunnels were constructed, calculated and measured from the finite element analysis and the physical tests.



[a] Tunnel installation at position 2 (W =2.0 D)



[b] Tunnel Installation at position 3 (W =1.4 D)

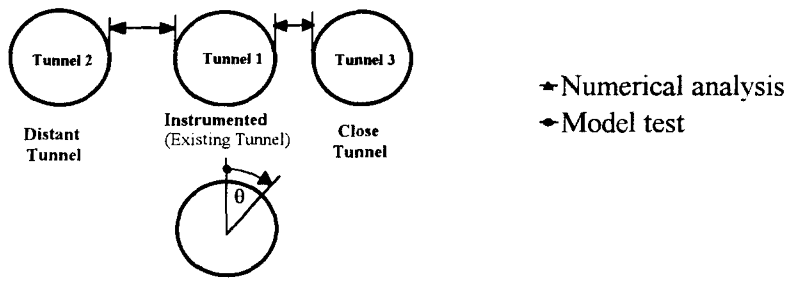


Figure 6. Additional Bending Moments(PS3)

These results indicate that significant bending moments were induced in the instrumented tunnel by installation of the new tunnels. In both cases, the numerical and model test data agree well. It is also found that the closer new tunnels are constructed to an existing tunnel, the larger additional moment is developed at the pillar springline of the existing tunnel liner.

5 CONCLUSIONS

The physical model tests show that for closely spaced tunnels the distortions and moments of the liner may be important. The values of the bending moments induced in the liners appear to be particularly significant.

It is shown that interaction between tunnels is unlikely to be significant unless the spacing between the tunnel centres is less than about two tunnel diameters.

Elastic finite element analysis with a hydrostatic gap modelling procedure gives results that appear to be in good agreement with the results of the physical model tests.

Based on the results of finite element analyses performed to verify experimental procedure, it is suggested that the procedures adopted in the model tests of using one clay sample to carry out two interaction experiments is reasonable.

6 ACKNOWLEDGEMENTS

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