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A Study of Interactions Between Perpendicularly Spaced Tunnels

상하교차터널의 상호거동에 대한 연구

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요 지

도심지 교통해소를 위한 수단으로 터널구조물의 건설은 날로 증가되는 추세에 있다. 그러나,새로운 터널구조물이 기존 터널구조물과 인접 또는 교차하여 건설됨에 따른 기존 터널구조물의 운영상의 안정성 확보는 상당히 중요하다. 이에 따라, 이 논문은 기존터널과 인접하여 건설되는 신설터널과의 상호거동에 대한 연구로써, 특히 상하교차터널에 대하여 중점을 두었다. 이 연구는 1g 모형시험을 실시하고 그들 결과의 분석을 통하여 교차터널의 거동에 대한 연구 고찰하였다. 또한 교차터널의 설계방향 제시함으로써 보다 실질적인 교차터널의 설계기술 발전을 위하여 활용되어 질 것으로 판단된다.

Abstract

This paper describes a study of the effect of shield tunnel construction on the liners of nearby existing perpendicular tunnels. The research programme investigated the influence of tunnel proximity and alignment, liner stiffness on the nature of the interactions between closely spaced tunnels in clay. A total of two sets of carefully controlled 1g physical model tests, including the same test for repeatability, were performed. A cylindrical test tank was developed and used to produce clay samples of Speswhite kaolin. In each of the tests, three model tunnels were installed in order to conduct two interaction experiments in one clay sample. The tunnel liners were installed using a model tunnelling machine that was designed and developed to simulate the construction of a full scale shield tunnel. The first tunnel liner was instrumented to investigate its behaviour due to the installation of each of the new tunnels. The interaction mechanisms observed from the physical model tests are discussed and interpreted.

Keywords : Liner stiffness, Model tunnelling machine, Perpendicular tunnels, Shield tunnel

1. Introduction

In recent years many new tunnels have been planned and designed or constructed near existing tunnels in order to develop and 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. Therefore, it is necessary to understand the interaction mechanisms between new and existing tunnels, to ensure that no damage occurs to the existing tunnel. In general, this interaction may be strongly dependent on the spacing between the tunnels. If the new tunnel is constructed at a large distance from the existing tunnel, the individual tunnels can be considered separately as single tunnel and analysed as such. However, if the influence zones of the two tunnels overlap, some degree

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of interaction between tunnels will take place. This interaction will affect the state of stress and the displacements in the ground around the tunnels, the ground surface displacements and the support loads.

The structural interactions that occur between closely spaced tunnels are associated with a variety of mechanisms. For tunnels that are located one above the other, additional mechanisms may come into play. For example, if a new tunnel is constructed beneath an existing tunnel, then this may lead to settlements in the upper tunnel. Further mechanisms have also been proposed; for example Hansmire *et al.* (1981) suggest that, for shield tunnelling, the stresses developed between the shield and the ground may lead to significant structural effects in any nearby tunnels. It is clear that the problem of interaction between adjacent tunnels is highly complex, and any structural interaction will depend on the geometry of the tunnels, the properties of the liner and the soil and the method of tunnel installation.

Interaction between closely spaced tunnels has been studied in the past using a variety of approaches. Much of the published work is based on measurements made during the construction of full scale tunnels. Some work is also reported in which finite element analysis was used to study the problem. Very little research, however, seems to have been carried out based on the use of reduced scale laboratory testing.

Hansmire *et al.* (1981) reported a set of measurements that were made during the construction of a new sewer in close proximity to a pair of subway tunnels. In this case, the sewer was perpendicular to, and above, the two existing tunnels. Excavation for the sewer led to an increase in the vertical diameter of the two existing tunnels in the zone immediately beneath it. This increase in diameter is, presumably, associated with a reduction in vertical stress acting on the tunnels. When the shield was advanced, however, the vertical diameter of the existing tunnels tended to decrease. Hansmire *et al.* suggested that this latter observation was associated with shear stresses developed between the shield and the soil.

Typical numerical analyses of the interaction between tunnels are 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. For the configurations investigated, the interactions between two parallel tunnels were small when the centre-line spacing was greater than two tunnel diameters. Addenbrooke and Potts (1996) reported numerical analyses of the interaction between two tunnels constructed with an interval of one month, using a small strain non linear soil model. They concluded that the interaction between two tunnels passing at depth depends on relative tunnel position (to the side or vertically above) and on the spacing. Driving a new tunnel above an existing tunnel has significantly less influence on the existing tunnel lining than driving a new tunnel to the side.

Little work has been carried out to investigate the interactions between tunnels using laboratory scale model tests. In recent years, however, much research on single tunnel behaviour has been carried out using centrifugal modelling, for example, particularly in the Cambridge Soil Mechanics Group by Cairncross (1973), Atkinson *et al.* (1974), Atkinson and Orr (1976), Potts (1976), Seneviratne (1979), Mair (1979) and Taylor (1984). It is difficult to simulate tunnel construction in a centrifuge. However, Nomoto *et al.* (1994, 1996) and Imamura *et al.* (1996) describe the design and development of a miniature shield tunnelling machine for a centrifuge.

In the performance of a tunnel liner, the relative flexibility of the liner and the surrounding soil is of fundamental importance. Peck *et al.* (1972) suggested that the relative stiffness of an embedded tunnel liner may be characterised by two dimensionless numbers termed the 'flexibility ratio' and the 'compressibility ratio'. The analytical basis for these parameters was derived by Burns and Richards (1964) and Heg (1968) taking account of soil-structure interaction. The flexibility ratio (F) determines the relative bending stiffness of the lining and the compressibility ratio (C) defines the relative stiffness of the lining in hoop compression. These ratios are expressed by:-

$$C = \frac{ER(1-v_L^2)}{E_L A(1+v)(1-2v)} \tag{1}$$

$$F = \frac{ER^3(1-v_L^2)}{E_L I(1+v)} \tag{2}$$

where EL, vL, R, A and I are respectively the Young's modulus, Poisson's ratio, radius, cross-sectional area and second moment of area of the liner, and E and vare the Young's modulus and Poisson's ratio of the soil. Note that $E = 2(1+v)G$, where G is the shear modulus. The term (1 - $vL2$) assumes that the liner is continuous longitudinally and may be omitted for segmental liners with compressible joints between successive rings.

Peck *et al.* (1972) suggested that tunnel liners may be considered flexible if F is greater than 10. Then the stresses acting on the interface between the liner and the soil may lead to significant liner displacements, and the nature of the stress distribution will be determined, in part, by the liner stiffness. In general, tunnels in soil have a value of C less than 1, while for rock tunnels C is greater than 1 (Sinha, 1989). However, for short-term undrained behaviour of clay, with v equal to 0.5, C becomes infinite and the liner hoop stiffness is of no consequence. Typical values of F for real tunnels lie in the range of about 5 to 300; however, tunnels with a substantial concrete secondary lining may be stiffer than this, while segmental linings with articulated joints between segments may be extremely flexible.

The project described in this paper sought to study the highly complex soil/structure interaction problem of closely spaced perpendicular tunnels in soft ground by high quality 1g laboratory scale model tests, with direct measurement of the additional bending moments and deformations induced in an existing tunnel by the installation of new tunnels nearby.

The project was confined to the study of tunnels in normally consolidated clay. At the start of the project, the need to develop accurate and repeatable methods of tunnel installation was identified and so a model earth pressure balance tunnelling machine was developed. Radial movements of the tunnel liner were identified as representing important interaction data; new instruments for making these measurements were therefore also developed. The tunnel linings used in the study consisted of thin steel tubes that were instrumented with strain gauges, pressure cells and pore pressure transducers.

2. Development of Testing Equipment and Procedures

Two sets of model tests were carried out. The behaviour of perpendicular tunnels was studied in a cylindrical tank (Figure 1). The test tank was designed

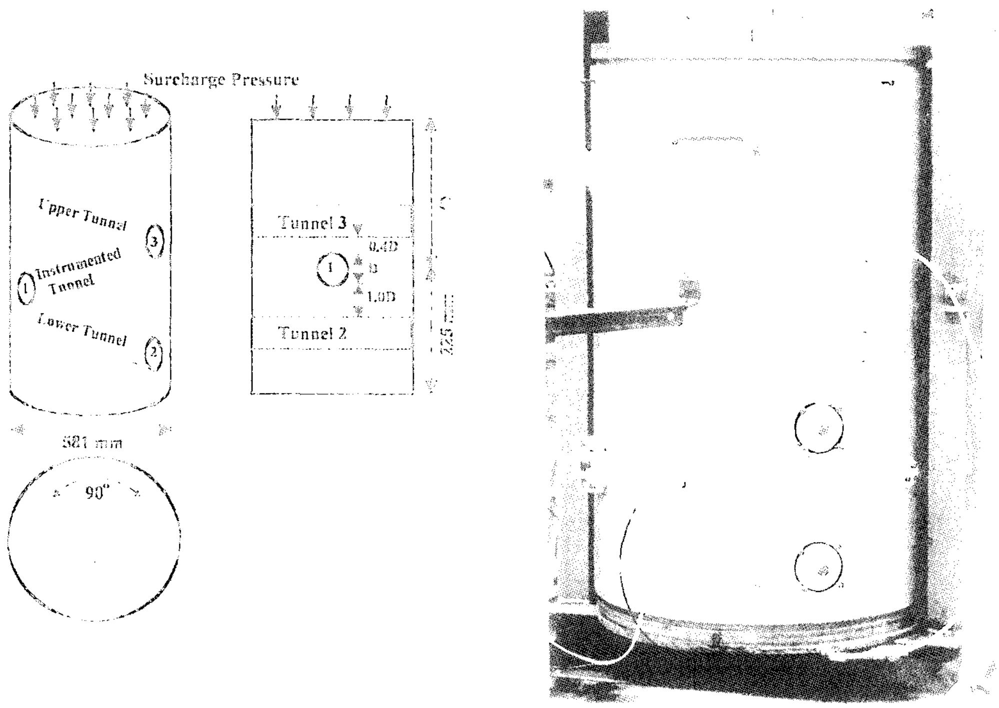

Fig. 1. Cylindrical test tank

such that each clay sample could be used to conduct two independent interaction tests. In order to achieve this, three positions were made available for tunnel installation.

The ports marked 1 in Figure 1 indicate the positions where the first tunnel of the test was installed. This tunnel was in all cases instrumented and was intended to represent an existing tunnel liner. Two further ports were available at axis separations of 1.4*D* and 2.0*D*, where *D* is the tunnel diameter. These values of separation were adopted on the basis of previous studies which suggested that interaction effects were important only for tunnel spacings of less than about 2*D*. Each test proceeded by installing tunnels in the 'distant' position (axis separation of 2.0*D*) followed by the 'close' position (axis separation of 1.4*D*). The assumption was made in the interpretation of the results that interaction effects measured during installation of the close tunnel were unaffected by the previous installation of the distant tunnel. This assumption was checked by finite element analysis (Kim 1996 and Kim *et al.* 1996). It was necessary to adopt a value of *D* that was sufficiently large to allow the liner to be fully instrumented and yet not so large that the tank boundaries might be expected to influence the results. A value of *D* of 70mm was adopted as a reasonable value.

All of the tests were carried out using Speswhite kaolin clay consolidated from a slurry in the test tanks themselves. The samples were all intended to have the same value of undrained shear strength, *su* of 20kPa.

2.1 Perpendicular Tunnel Tests

Two perpendicular tunnel tests were carried out, including a repeated test to check the consistency of the testing procedures. The tests were carried out with a surcharge of value *Psur* applied to a stress controlled boundary on the top surface of the clay in order to provide the required value of OCR. The tests are specified in Table 1.

The values of OCR listed in Table 1 were calculated using the actual values of overburden stress acting at the level of the tunnels in the model tests. The thickness (t) of the tubular steel liners was chosen to ensure reasonable values of the flexibility ratio *F*, of about 70 and 200. Although, based on Peck's criterion, both of these tunnels would be classified as flexible, in the discussion of the results given below the liner for which t = 0.254mm is termed 'flexible' and the liner for which t = 0.356mm is termed 'stiff'.

The values of soil shear modulus given in Table 1 were estimated from the measured values of undrained shear strength and the appropriate value of OCR, on the basis of charts presented by Duncan and Buchignani (1976) and further information from Wroth *et al.* (1979), assuming a plasticity index of 31%. The clay shear strength was measured while the surcharge loading was applied, by means of a shear vane inserted into the sample through ports installed in the front panel of the test rig. Shear vane measurements were also made after removal of the surcharge and after swelling had taken place; these latter results, of course, do not correspond directly to the soil strengths that were relevant to the tests although the two sets of data could be correlated.

2.2 Instrumentation

A variety of instrumentation systems were adopted as summarised below. A study on the accuracy of the measurements is given by Kim (1996).

1) Monitoring of Sample Preparation

Two transducers were installed in the side of each test tank in order to measure total stresses and pore pressures during consolidation. Sample settlement and consolidation pressure were also recorded.

Table 1. Specification of tests

Test No.	OCR	*Psur*(kPa)	*t*(mm)	*F*	*su*(kPa)	*G*(kPa)
CY1	1	89.04	0.254	214	23.6	4712
CY2	1	88.2	0.356	75	22.7	4530

2) Instrumented Tunnel

Instrumentation of the first tunnel to be installed is illustrated in Figure 2 and included:

(a) Eight strain gauges spaced equally around the circumference to measure hoop strains on the liner exterior. Strains recorded using these gauges were used to estimate liner bending moments on the assumption that hoop forces were small.

(b) Four miniature total pressure cells.

(c) Two pore pressure transducers.

3) Measurement of Radial Movements

Two different types of radial displacement measuring devices, RDMD1 and RDMD2, were developed and used to monitor the radial displacements of the liner.

The RDMD1 consisted of four strain gauged cantilevers made of stainless steel, mounted on a specially machined aluminium block, to measure radial displacements of the crown, invert and both springlines of the tunnel liner. The maximum range of each cantilever was 20 mm corresponding to the maximum elastic deflection. Each cantilever was 70 mm long, 5 mm wide, 0.25 mm thick and inclined to the horizontal at 40°. A dome-shaped tip was attached to the free end of each cantilever to ensure good contact with the measuring points.

This device was found to be sensitive to temperature variations, and the RDMD2 was developed to overcome this problem. It measured the liner deformations by converting the displacement direction from radial to axial through four arms rotating about pin supports on a mounting block and connected by pin-jointed bars to corresponding LVDTs. A small roller was mounted at the end of each arm to move smoothly on the liner. The LVDTs had a maximum working range of 10 mm.

2.3 Model Tunnelling Machine

A model tunnelling machine, based on an earth pressure balance shield, was therefore developed as shown in Figure 3. The rotary cutting head consisted of a flat plate with two cutting blades. The head was connected to a shaft which was rotated by an electric motor in order to excavate the soil. Soil removed by the cutting head was washed back through the tunnel using a water jet system. The lining was jacked into the soil immediately behind the cutting head which was oversized slightly in order to produce a ground loss of about 6%.

To operate the machine, a small guide shaft was first placed in the sample along the tunnel axis. This shaft was

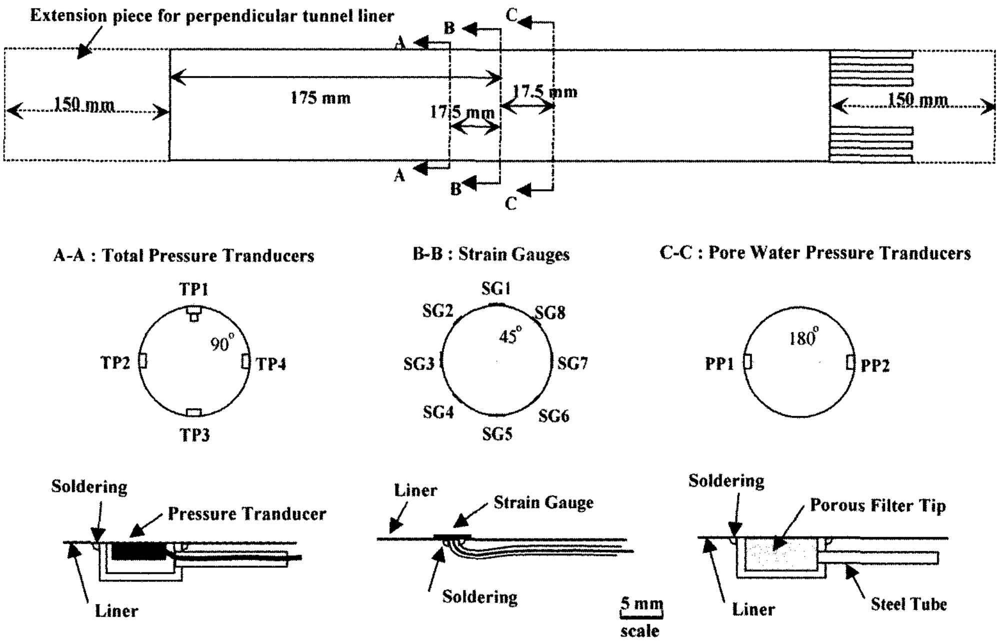

Fig. 2. Tunnel liner instrumentation

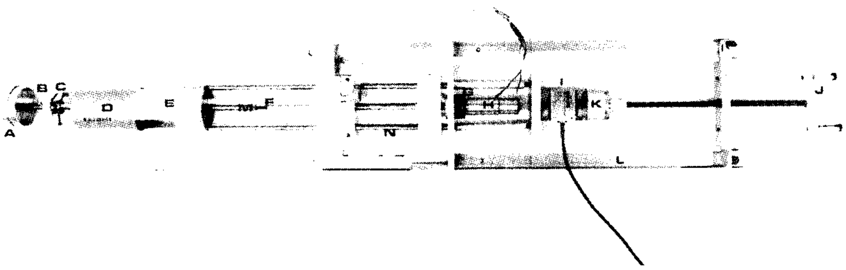

Fig. 3. Model tunnelling machine

installed to a high accuracy using a reference frame bolted to the side of the tank, and was then used to locate the cutting head during tunnel installation.

3. Model Test Results

The normal stress interaction effects were strongly dependent on the new tunnel positions. As would be expected, the largest stress interaction effects occurred at the crown for the upper tunnel case and at the invert for the lower tunnel case.

Figure 4 shows the variation of the normalized incremental bending moments (*M*/*HR*2) plotted against flexibility ratio (*F*) for installation of the new perpendicular tunnels. Here *M* is the maximum incremental bending moment developed in tunnel 1 after installation of each of tunnels 2 and 3, at the crown for the upper tunnel (close spacing, S = 0.4D), at the invert for the lower tunnel (distant spacing, S = 1.0D). The moment interaction effects are strongly dependent on new tunnel position and liner stiffness rather than tunnel spacing. It is clear that in the upper tunnel case (S = 0.4D), the interaction effects were much greater for the flexible liner than the stiff liner, whereas in the lower tunnel case (S = 1.0D), the influence of liner stiffness seems to be small. For the lower tunnel case, the moment interaction effects are relatively similar in magnitude to those measured in the corresponding parallel tunnel tests, while for the upper tunnel case the interaction effects are much smaller than for the corresponding parallel tunnel tests, in spite of the fact that the upper tunnel was closer than the lower tunnel. The differences between perpendicular and parallel tunnels tend to increase with increasing stiffness of the liner.

Figure 5 shows the diameter changes (*D*) of the existing tunnel after construction of the new tunnels plotted against flexibility ratio (*F*). The distortion interaction effects for perpendicular tunnels are strongly dependent on the liner stiffness. The variations in diameter changes are much larger for the flexible liner than for the stiff liner. It is worth noting that the diameter changes seem to be very similar in both upper and lower tunnel cases.

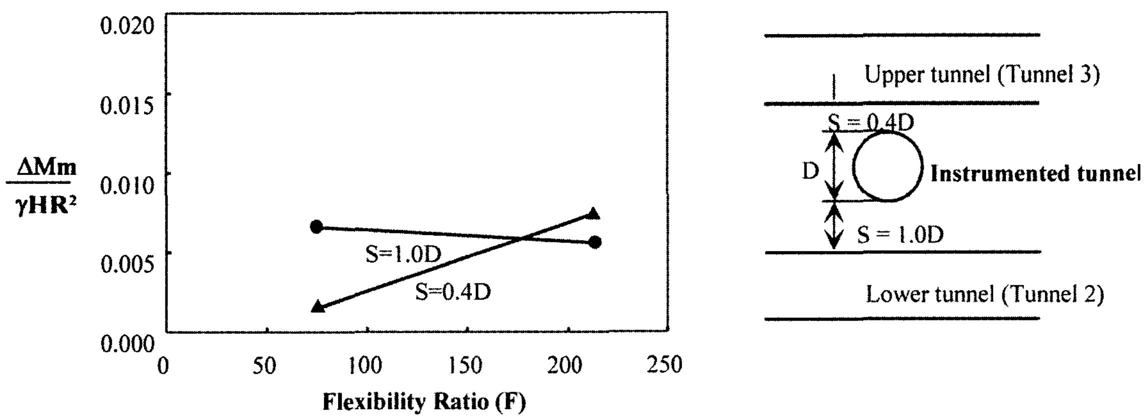

Fig. 4. Variation of incremental bending moments

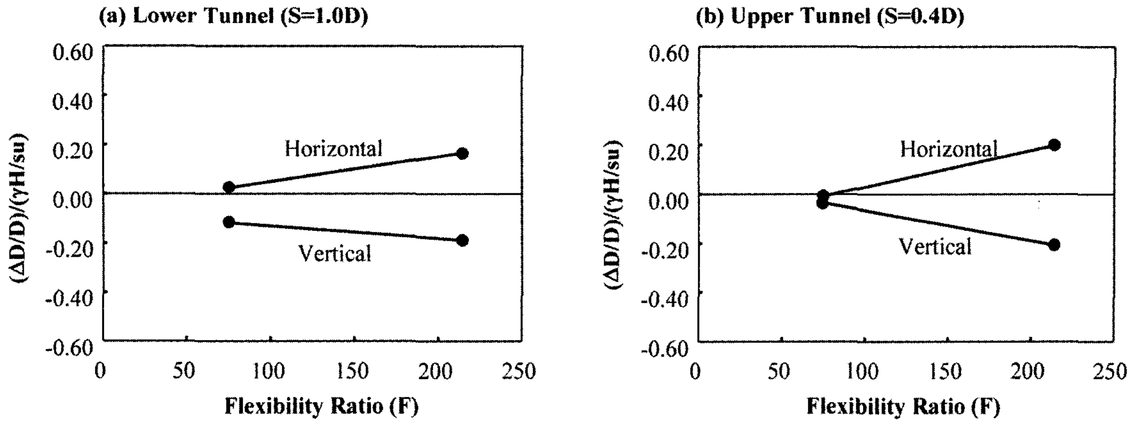

Fig. 5. Variation of incremental diameter change with W/D

However, following construction of the lower tunnel, the instrumented tunnel also displaced downwards by a significant amount, presumably as a result of settlement generated by the lower tunnel. The deformation interaction effects in perpendicular tunnels are relatively smaller than those observed in the corresponding parallel tunnel tests, the differences being much greater in the upper tunnel case than in the lower tunnel case.

Figure 6 shows the deformed cross sections of instrumented tunnel after adjacent perpendicular tunnel construction. These data indicate that, in all cases, installation of a new perpendicular tunnel caused increases in the horizontal diameter, and corresponding decreases in the vertical diameter, of the adjacent tunnels. It is thought that this tendency is associated with additional ground deformations caused by the jacking forces used to advance the shield and the tunnel liner. These additional deformations are thought to be sufficiently large to counteract the expected tendency of the vertical diameter of the adjacent tunnels to increase as a result of stress relief. A similar trend is apparent in the field data reported by Hansmire *et al.* (1981) relating to the construction of perpendicular staked tunnels in New York. These field data show that the interaction effects developed in adjacent tunnels are strongly influenced by the magnitude of the jacking force used to advance the shield. Large jacking forces tended to cause the horizontal diameter of adjacent tunnels to increase; when smaller jacking forces were adopted then the horizontal diameter of adjacent tunnels trended to decrease.

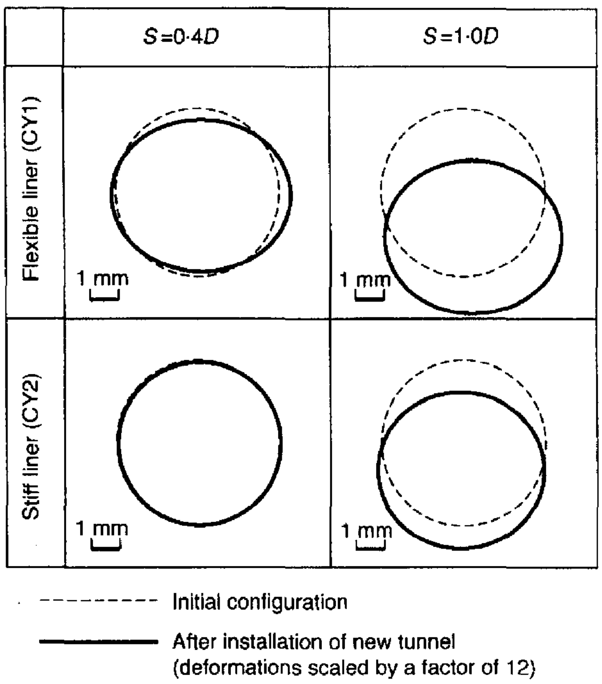

Fig. 6. Deformed cross-sections for perpendicular tunnels

The observed interaction behaviour of the perpendicular tunnels appears to be dominated by liner installation effects.

4. Conclusions

(1) High quality tests were carried out at laboratory scale for closely-spaced perpendicular tunnels.

(2) A model tunnelling machine was developed to allow accurate and repeatable tunnel installation procedures to be adopted.

(3) For the cases studied, tunnel installation was shown

to modify the stresses acting on the liner of an existing adjacent tunnel. These stress increments led, in turn, to liner deformations and induced bending moments. In extreme cases, the induced bending moments were found to be a factor of two or three greater than the maximum liner moment before adjacent tunnelling took place.

(4) The results of the perpendicular tunnel tests suggest that, in these cases, interaction effects are predominately caused by jacking forces applied to the liner and the tunnelling machine during tunnel installation.

(5) The perpendicular tunnel tests indicated interaction effects that were generally more severe than those expected in the field. It was observed that a perpendicular tunnel constructed beneath the instrumented tunnel tank led to relatively large movements.

(6) The experience gained from these tests indicates that the interaction mechanisms that operate between adjacent tunnels are highly complex.

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