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Twin-tunnelling-induced ground movements in clay

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Modern tunnelling methods aim to reduce ground movements arising from the construction process. In clay strata the usual method of construction is by tunnel boring machine, which allows close control of the tunnelling process; however, any movements have the potential to cause damage to existing structures at, and below, the ground surface. The construction of underground rail systems often comprises two tunnels running in opposite directions. Common practice for assessing construction-generated movements around these tunnels is to make predictions based upon individual tunnel construction and utilise superposition to generate a total deformation profile. This approach does not take into account the strain- or stress-dependent effects between tunnel constructions. A delay may result in unanticipated ground movements generated by the construction of the second tunnel. The effect of this delay on the ground movements arising between the first and the second tunnel construction process was investigated in a series of plane strain centrifuge tests. The ground movements at and below the surface were monitored and were assessed against superposition-based predictions for surface settlement with the outcomes highlighting some inconsistencies. A procedure for predicting both surface and subsurface vertical settlement profiles in the plane transverse to the advancing tunnels in clay is suggested.

Notation

D	tunnel diameter
d	distance between the tunnel centre-lines
i	horizontal distance from the tunnel centre-line to the point of inflection in the Gaussian distribution curve
K	dimensionless trough width parameter
S_V	settlement at a given horizontal distance from the tunnel centre-line
S_{Vmax}	maximum settlement
V_L	volume loss expressed as a ratio of the area of 'ground loss' to area of bored tunnel
x	horizontal distance from tunnel centre-line
x_A	horizontal distance from centre-line of first bored tunnel
z	vertical distance below the un-deformed ground surface
z_0	vertical distance from the un-deformed surface to the tunnel axis level

1. Introduction

London is a prime example of an urban environment that has restricted surface space. To meet the growing demand for mass rapid transit, tunnelling has been extensively utilised. The London Underground network services the majority of Greater London by way of a number of distinctly separate lines, which are generally contained within tunnels in the city centre. These tunnels are in pairs to facilitate travel in opposite directions and are usually at a relatively close spacing and within a relatively short space of time. Irrespective of the method used, any underground construction will have unsupported soil at some point and consequently ground movements will occur. These movements propagate throughout the soil mass and may have a detrimental effect on buildings and services.

Devriendt (2010) detailed the design requirements during potential damage assessment of a proposed tunnelling project. The author stated that stage 1 is the identification of the significant settlements. The criteria used to decide whether existing services or structures are within an area of potential damage are when the settlements are anticipated to be greater than 10 mm or gradients greater than 1:500. The latter measure addresses the strains transferred to buildings through differential settlements. Mair *et al.* (1996) suggested that building damage for sections of a building in hogging or sagging modes could be quantified by comparing the deflection of the building with the tunnelling-induced settlements. This suggests that not only should the practising engineer consider the magnitude of the settlements but also the extent and shape of the settlement trough. Therefore, any improvement in the prediction of ground movements would be beneficial in order to give greater confidence in the anticipated zones where structures may be influenced by tunnelling-induced ground deformations.

Accurate predictions can ensure efficiency and significant cost savings. Burland (2001) identified the costs for the Jubilee Line Extension (JLE) project and mitigation measures. The estimated project cost was £1·8 billion with the civil engineering works initially costing £650 million. Approximately one sixth (£108 million) would be spent on protective measures (i.e. underpinning and compensation grouting).

Tunnelling-induced ground movement predictions have been developed based, largely, on knowledge from single tunnel arrangements (e.g. Attewell and Yeates, 1984; Mair, 1979; Peck, 1969; Taylor, 1984). Twin-tunnelling surface settlement predictions are often made assuming the superposition of two single tunnel predictions. The assumption is that the ground movements arising from the construction of a second tunnel are unaffected by construction of the first. Previous research, particularly numerical studies, has indicated that superposition may not necessarily be sufficient (i.e. Addenbrooke and Potts, 2001; Hunt, 2005).

To investigate the influence of tunnel arrangement on the ground movements generated during construction, eight plane strain centrifuge tests were conducted. The work described in this paper explores the ground movements in overconsolidated clay when sequentially constructing parallel tunnels with a small separation distance. Relatively complex apparatus was used to accurately simulate volume loss in both tunnels (see Divall and Goodey, 2012). This allows the simulation of two identical volume loss events representing the construction of individual tunnels. A pause representing a construction delay is introduced between each simulated construction and the overall patterns of ground movement, both at and below the surface, are monitored.

2. Current practice for predicting tunnelling-induced ground movements

During construction using a tunnel boring machine (TBM) the bored size of a tunnel will always be larger than the external diameter of the segmental lining. Mair and Taylor (1997) described the primary sources of the resulting short-term ground movements by dissecting this construction process.

- Component 1, the deformation of the ground towards the face. Where using closed-face tunnelling, controlling the TBM face pressure is crucial for minimising any subsequent ground movements.
- Component 2, the passage of the shield, provides the overcutting at the TBM face.
- Component 3, the tail void, is often minimised by grouting to fill the gap.

Peck (1969) described tunnelling-induced ground movements as radial displacements towards the cavity and longitudinal displacements towards the advancing tunnel heading. This phenomenon has been described by the term 'volume loss' or 'ground loss' (Peck, 1969). Because of the complex three-dimensional nature of these movement patterns (Attewell and Woodman, 1982) analysis methods often separate them into two scenarios: the longitudinal settlements (in the plane of the advancing tunnel) and transverse (in the plane perpendicular to the advancing tunnel). In the research presented here only the transverse settlement trough is investigated.

It is important to note that in the undrained case the volume of 'ground loss' around a tunnel cavity should be equal to the volume of any subsequent surface settlement trough. Surface settlement troughs are formed due to the propagation of displacements towards the cavity. It is the prediction of these settlements that are of importance to practising geotechnical engineers.

2.1 Single tunnelling-induced ground movements

It is accepted that the displacements associated with single tunnelling-induced ground movements fit a form of Gaussian distribution. This was proposed by Peck (1969) and verified by many site measurements and centrifuge tests (e.g. Mair *et al.*, 1993). Semi-empirical approaches have been adopted based on this observation for calculating the surface settlements as follows

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

where

$$S_{V\max} = \sqrt{\frac{\pi}{32}} \frac{V_L D^2}{i} \tag{2}$$

S_V is the theoretical settlement at a given horizontal distance from the tunnel centre-line; $S_{V\max}$ is the theoretical maximum settlement at the tunnel centre-line; x is the horizontal distance from the tunnel centre-line; i is the horizontal distance from the tunnel centre-line to the point of inflection in the Gaussian distribution curve; and V_L is the volume loss expressed as a ratio of the area of 'ground loss' to area of bored tunnel.

When considering the surface settlement trough above a tunnel, the volume loss is essentially a measure of the magnitude and i is a measure of the distribution. This implies that i will control the settlement trough width with larger values indicating a wider trough. O'Reilly and New (1982) proposed that

$$i = Kz_0 \tag{3}$$

where z_0 is the vertical distance from the un-deformed surface to the tunnel axis level and K is a dimensionless trough width parameter. The average value of K was found to be 0·5 for tunnels in moderately stiff clay. This agreed in general with the findings of Peck (1969), although the data presented varied between 0·4 and 0·6.

Mair *et al.* (1993) indicated that although the surface settlement troughs above single tunnels were well predicted by assuming values obtained from Equation 3, the magnitude of i at depth was considerably underestimated. The authors considered the following distribution of i with depth

$$\frac{i}{z_0} = 0{\cdot}175 + 0{\cdot}325\left(1 - \frac{z}{z_0}\right) \tag{4}$$

which implies that K varies with depth as

$$K = \frac{0{\cdot}175 + 0{\cdot}325(1 - z/z_0)}{1 - z/z_0} \tag{5}$$

where z is the vertical distance below the ground surface.

2.2 Twin-tunnelling-induced ground movements

Superposition is the accepted method of predicting settlements above any twin-tunnel arrangement. Essentially, a Gaussian distribution of the settlements is assumed with the maximum settlement positioned over the centre-line of the first constructed tunnel. An identical distribution is positioned over the centre-line of the second constructed tunnel and ignores any influence of the first. The summation of these two overlapping distributions describes the twin-tunnel settlement.

O'Reilly and New (1982) proposed a formula for the prediction of surface settlements by superposition

$$S_{\mathrm{V}} = S_{\mathrm{Vmax}}\left[\exp\left(\frac{-x_{\mathrm{A}}^2}{2i^2}\right) + \exp\left(\frac{-(x_{\mathrm{A}} - d)^2}{2i^2}\right)\right] \tag{6}$$

where d is the horizontal distance between the two tunnels' centre-lines and x_{A} is the horizontal distance from the centre-line of the first bored tunnel. However, the expression assumes the tunnels have the same depth, diameter, magnitude of volume loss and settlement trough width. Moreover, the formula is unverified for predicting subsurface settlements.

Nyren (1998) conducted a detailed study of the settlements resulting from the JLE at St James' Park, London. This is an example of twin 4·85 m diameter running lines constructed sequentially in very stiff London Clay. The westbound tunnel was constructed first followed by the eastbound tunnel. These tunnels were 21·5 m apart in plan at depths of 31 m and 20·5 m, respectively. The settlement half trough, towards the first tunnel, of the eastbound tunnel was wider and deeper than expected. Nyren (1998) observed a volume loss in the settlement half trough away from the existing tunnel of 1·1%. However, the measured volume loss of the half trough towards the existing tunnel was a larger value of 1·8%. Nyren (1998) stated the eastbound tunnel was significantly affected by the previous construction of the westbound tunnel. Moreover, superposition was seen to under-predict the overall settlements due to the additional volume loss observed during construction of the second tunnel. These observations would appear to highlight the deficiencies inherent in using superposition of similar Gaussian settlement distributions as a prediction technique.

2.3 Recent twin tunnelling research

Recent research undertaken to better understand the ground's response to these construction processes has consisted of complex non-linear finite-element analysis and 1$\boldsymbol{g}$ laboratory tests. Addenbrooke and Potts (2001) performed site-specific finite-element analysis on a variety of twin-tunnel arrangements using an elastic–perfectly plastic constitutive model. The analysis produced design charts which intended to modify the key parameters V_{L} and the position of S_{Vmax} for the second tunnel. Chapman *et al.* (2006) performed a series of 1$\boldsymbol{g}$ laboratory tests in which an auger type cutter within a shield was utilised to investigate the subsurface movements in Speswhite kaolin clay. The average shear strength values were reported to be 20 kPa (with surcharge) and 5 kPa (without surcharge). The subsurface movements were recorded using digital photography (first successfully used by Mair (1979)) through a Perspex window, which had replaced one of the walls of the soil container. The results from this study are reported to match with the modification factor method proposed by Hunt (2005) and field monitoring. This method aims to modify settlements of the second tunnel in an 'overlapping zone' dependent on its distance from the first tunnel construction.

3. Centrifuge model tests

3.1 Introduction

In order to provide some insight into the observations by Nyren (1998), a series of physical model tests was conducted into this complex construction scenario. Centrifuge modelling has been shown to provide a means of conducting well-controlled effective-stress-path scale model tests using real soil (Taylor, 1995). The tests were carried out using City University London's Acutronic 661 geotechnical centrifuge, which has a radius of 1·8 m and is capable of achieving 200 times earth's gravity.

Mair (1979) showed that while tunnelling-induced ground movements are three-dimensional, many useful insights can be gained from two-dimensional idealisations of tunnels. Taking this assumption, apparatus was developed (described by Divall and Goodey (2012)) to simulate sequential tunnel constructions in moderately stiff overconsolidated clay. Essentially, the apparatus provided support to pre-formed tunnel cavities using water and then generated the movements associated with volume loss by removing a precise volume of that water.

3.2 Test series

The details of the test series are given in Table 1 and illustrated schematically in Figure 1. Figure 2 shows the general arrangement and dimensions of a typical test within the model container. A range of conventional instrumentation was used to monitor each test including Druck pore pressure transducers (PPTs) to

Test ID	V_L: %	Cover (D)	Spacing (D)
SD10	3	2	1·5
SD11	3	2	3
SD12	3	2	4·5
SD13	5	2	1·5
SD14	5	2	3
SD15	5	2	4·5
SD17	3	2 and 3·5	2·7
SD18	3	3 and 3·5	2·12

Table 1. Details of the various twin-tunnel arrangements tested

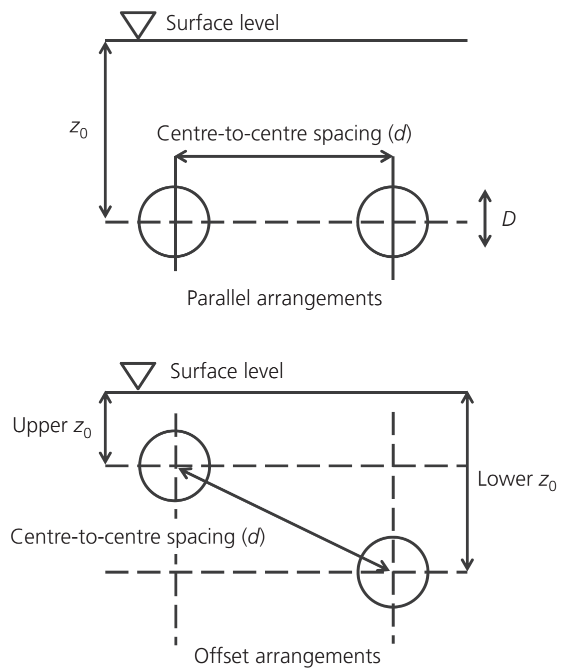

Figure 1. Illustration of the various twin-tunnel arrangements tested

measure groundwater pressures and linear variable differential transformers (LVDTs). A rack containing 12 LVDTs was bolted to the top of the soil container for measurement of the vertical settlement at the surface. A row of nine LVDTs was placed along the transverse centre-line of the model at distances of 0, ±45, ±90, ±135, ±180 mm from the plane centre-line of the soil container. A second row of three LVDTs was offset 45 mm from the transverse centre-line of the model at distances of 0, −90, −180 mm from the plane centre-line of the soil container. These measurements were used as a check to ensure that the response of the model was plane strain as expected. Ports in the back wall of the soil container allowed the installation of the PPTs within the model and the fluid feed for the tunnels. The PPTs were installed

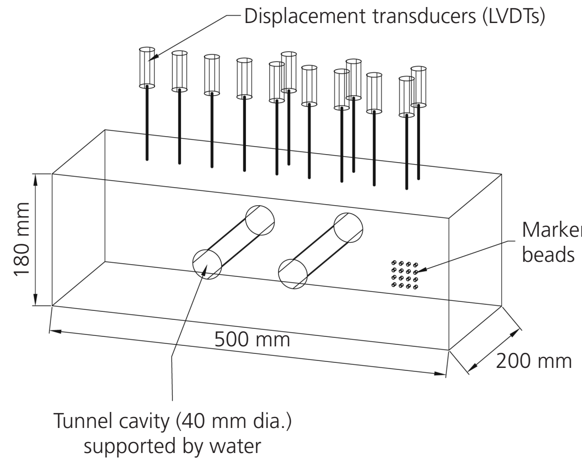

Figure 2. Schematic diagram of an example test arrangement (after Divall and Goodey, 2012)

between the tunnels (space permitting) and close to the model boundaries. In addition to this conventional instrumentation the movement within the soil mass was recorded using a digital image-processing system (described in Taylor *et al.* (1998)). Subsurface movements were tracked by monitoring the movements of markers pressed into the front surface of the clay, observed through a Perspex window bolted to the front of the soil container.

3.3 Test procedure

The test procedure began with the preparation of the model. A slurry was mixed from Speswhite kaolin clay powder and distilled water to a water content of 120%. This clay slurry was placed inside a rectangular soil container and consolidated one-dimensionally in a hydraulic press. The desired stress history was achieved by consolidation to a vertical effective stress of 500 kPa followed by a period of swelling to 250 kPa vertical effective stress. A further period of consolidation occurred during centrifuge spin-up, resulting in a clay sample with an overconsolidation ratio of approximately 6 at the level of tunnels.

The soil container was removed from the press to begin the model-making stage. The front wall of the soil container was removed to gain access to the clay. The clay was trimmed to the required height and the tunnel cavities bored using a thin-walled, seamless, circular, stainless steel cutter. The tunnel supporting apparatus was placed inside the cavities and the necessary fluid pipes connected. The system was bled to ensure no air remained, as this was found to be highly detrimental to the performance of the apparatus. The aforementioned Perspex window replaced the front wall of the soil container to allow subsurface observations. The models were sealed with silicone oil to prevent the model from drying out and the final pieces of instrumentation were secured. At this stage the model preparation was complete and the soil container was loaded onto the centrifuge swing.

During the increase in acceleration to 100***g***, support pressure equal to the overburden within the two cavities was automatically maintained by a standpipe. On reaching the required acceleration these pressures were checked and, if satisfactory, the tunnelling system was isolated from the standpipe so that the amount of fluid within the tunnels was fixed. Water was fed to the soil model by way of a second standpipe to maintain a water table set just below the ground surface. The model was left at 100***g*** for a period of approximately 24 h to ensure that the pore pressures within the soil were in equilibrium. At this stage the model had an overconsolidated stress history which also varied with depth.

A series of solenoid operated valves were able to isolate each tunnel. In essence, this meant only one tunnel at a time would undergo a simulated construction. By means of a Bishop ram driven by a servo motor 3% or 5% of the total volume of the cavity was removed from the first tunnel (tunnel A, which in the case of the offset arrangements is the lower tunnel). This was followed by a pause to represent a construction delay of 3 weeks at prototype scale, during which time very little consolidation settlement occurred (verified by monitoring the surface profile between the end of construction of the first tunnel and the beginning of the second). Once this time had elapsed the second tunnel construction was simulated (tunnel B) by removing the same volume of water as from the first tunnel. The deformations during the entire construction process were recorded by the LVDTs and the image-processing system at a rate of approximately one set of readings per second.

4. Results

4.1 Surface settlements

A number of centre-to-centre spacings were investigated and, for clarity, selected results are presented here. A full description of the surface measurements for all centre-to-centre spacings is given by Divall (2013) and any potential influence of the chosen time delay is discussed by Divall *et al.* (2014).

Figure 3 shows the final settlement trough data as measured by the LVDTs from tests where the centre-to-centre spacing of the tunnels was 3*D*. The settlement trough was obtained with reference to a zero reading of the surface measurements immediately before the start of the tunnel construction simulation. The final settlements were taken at a point immediately upon completion of the second tunnel. These settlements have been normalised against the maximum settlement observed after the first tunnel construction and it should be noted that after this normalisation the data for both tests are highly comparable. The centreline and data relating to the first tunnel have been denoted by T_A and for the second, T_B. A surface settlement predication using Equation 6 is generated from the combined normalised experimental data. Values of S_{Vmax} and K are determined by a least-squares fit to the tunnel A data. It is clear that, particularly in the areas immediately above the second tunnel, the data are a relatively poor fit to the superposition-based prediction.

To further investigate the settlements caused by each construction

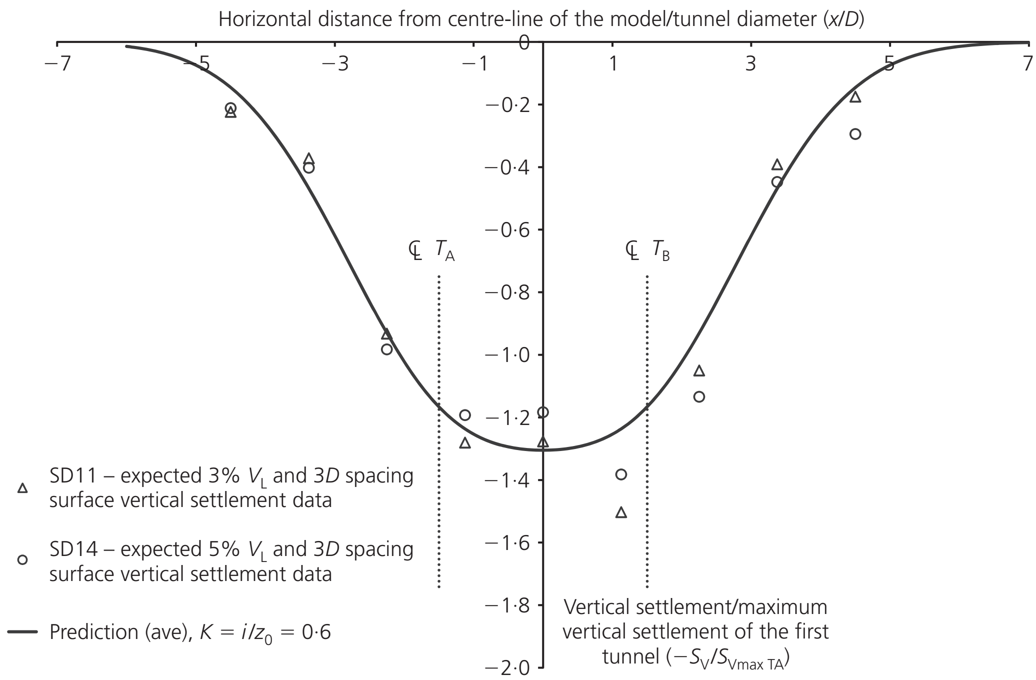

Figure 3. Surface settlement trough from two sequential tunnel constructions at a centre-to-centre spacing of 3*D*

event, the total settlements obtained were separated into those generated by tunnels A and B, respectively. Figure 4 shows this breakdown of the data. The second tunnel settlements were derived from the total end-of-test settlements minus the first tunnel settlements. A Gaussian distribution curve as described earlier was fit to the combined, normalised, individual tunnel data, again using a least-squares method (although it should be noted that there are many possible methods that could be used to interpret the data, such as those described by Jones and Clayton (2012)). In this exercise the values of S_{Vmax}, K and the position of S_{Vmax} for each tunnel were varied. The settlements solely from the first tunnel construction are shown to have good agreement with O'Reilly and New (1982). This is expected because the first tunnel is excavated in effectively a greenfield site and this behaviour was reflected in the first tunnel settlements for all tests.

Three observations can be made from this procedure. First, the observed volume loss associated with the second tunnel is larger than that of the first. For the 3*D* tests presented here the additional volume loss was on average 14%. Second, the surface settlements generated by either tunnel construction on the side of the centre-line away from the other tunnel are fairly typical in shape (i.e. could be well represented as a Gaussian curve with values of i and K comparable with previous research and field data). The third observation is that the surface settlements generated by either tunnel construction on the side of the centre-line towards the other tunnel do not appear to be as well represented by the Gaussian curve. This could potentially be a feature of the modelling technique, as both tunnels are pre-bored in the clay prior to the experiment. These observations described in relation to the 3*D* spacing tests were also observed in the other arrangements tested. In essence, the closer the construction is of a second tunnel to the existing tunnel, the greater the effects on the associated settlements.

4.2 Subsurface

Subsurface movement data obtained from the image analysis system were analysed in a similar method to that described for the surface settlement data. Grant (1998) stated that in the vertical direction the error in measurements could have been in the order of 10–20 μm. As before, a number of centre-to-centre spacings were investigated and the results obtained from the 1·5*D* spacing tests will be presented in this section for clarity. Figure 5 shows the total subsurface settlement data measured by the image-processing system at a depth of 39 mm (approximately 1*D*) below the surface. The settlement trough was determined from analysis of images taken before and after both tunnel construction events. These settlements have again been normalised against the maximum observed subsurface settlement arising from the completion of tunnel A. The plotted prediction is determined from measurements of volume loss and maximum settlement produced by tunnel A, which are then used as the basis of a simple superposition calculation in the form of Equation 6.

It is clear that the subsurface settlement trough is not well represented by a superposition-based prediction. Therefore, a similar analysis method to that described for the surface settlement data was carried out for the settlements at this depth (Figure 6).

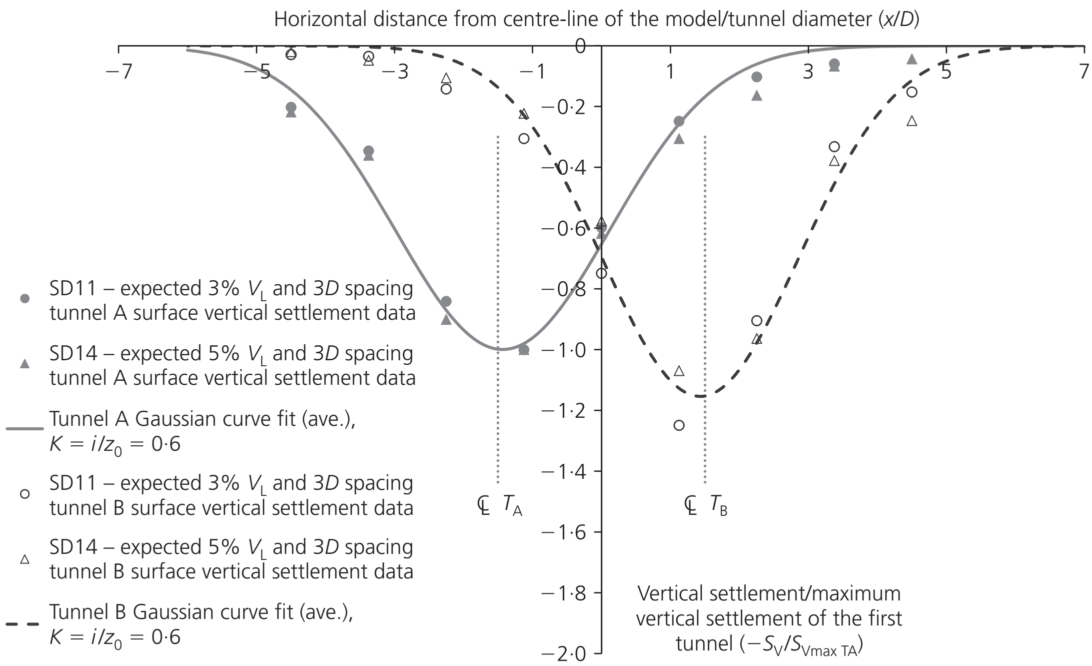

Figure 4. Individual surface settlement troughs for the first and second tunnel constructions at a centre-to-centre spacing of 3*D* with the individual superimposed Gaussian curves shown

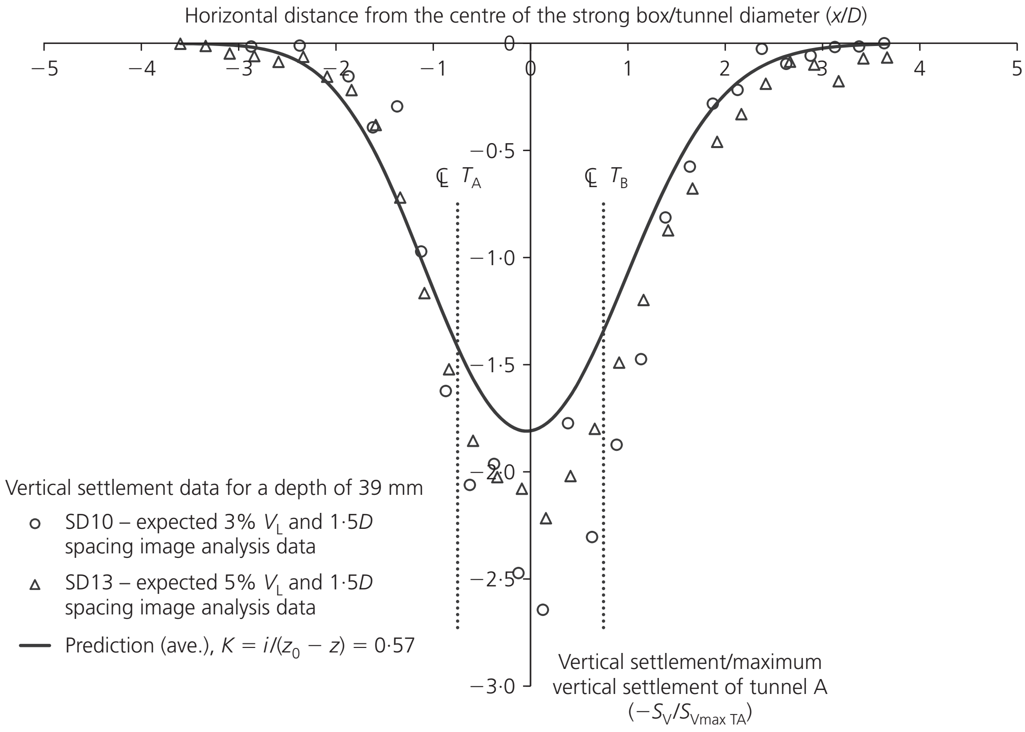

Figure 5. Subsurface settlement trough from two sequential tunnel constructions at a centre-to-centre spacing of 1·5*D*

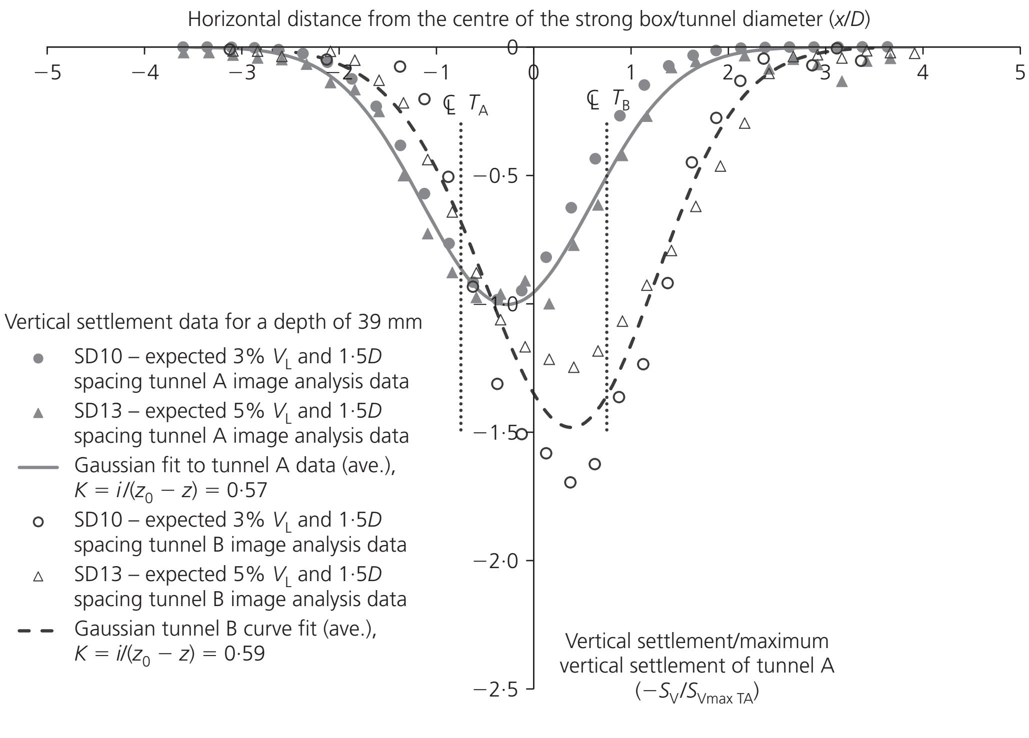

Figure 6. Individual subsurface settlement troughs for the first and second tunnel constructions at a centre-to-centre spacing of 1·5*D* with the individual superimposed Gaussian curves shown

The analysis procedure was also carried out for other spacings and depths below the surface (depth of 1·5*D* as well as 1*D*). Similar conclusions were drawn from the subsurface data as those for the surface.

4.3 Combined data

The data presented in the previous sections for all the tests have been re-plotted to provide further insight into overall settlement patterns. The apparent volume loss solely due to the second tunnel construction has been calculated from the area underneath the curves generated by the best fit process. This was conducted for all depths and all centre-to-centre spacings. The increase in volume loss could then be calculated by subtracting the greenfield value of volume loss (i.e. the tunnel A value) and then dividing by the same value to give a percentage increase. Figure 7 plots these increases in volume loss, for all observed depths, against centre-to-centre spacing in terms of tunnel diameter. There is clearly a trend showing that, as the separation between the tunnels increases, the effect on the additional volume loss reduces.

To examine the observed asymmetry of settlement troughs produced by tunnel B construction, separate Gaussian curves were fit to the data taken from the left- and right-hand sides of the second tunnel centre-line. This analysis gives separate values of K for settlements towards or away from the existing tunnel A. The asymmetry (shown by large differences in the values of K obtained) was more pronounced at centre-to-centre spacings less than 3*D* and in the offset arrangements. Figure 8 shows the values of K obtained from this analysis. K values on the side of the settlement trough near tunnel A are systematically higher for lower centre-to-centre spacings. The trend described by Equation 5 (Mair *et al.*, 1993) is also plotted and could safely be assumed to be valid for the settlement on the side away from tunnel A. A second, similar, relationship based on the data towards tunnel A is also shown. At spacings above 3*D* the settlement trough produced by construction of tunnel B was fairly symmetrical.

Given data on a particular twin-tunnel arrangement, Figures 7 and 8 could be utilised to predict the magnitude of additional volume loss and asymmetry that might be expected. These values could then be inserted into the relationships described by Peck (1969) and Mair *et al.* (1993) to predict settlements solely attributable to the second tunnel construction. These modified settlements could be summed with the greenfield first tunnel settlements (as proposed by O'Reilly and New (1982)) to give the total twin-tunnel settlements. This method would account for the effect of spacing and construction delay in the settlement prediction data.

5. Conclusion

The results from the eight centrifuge tests described in this paper have shown some shortcomings in the accepted practice of superposition for the prediction of twin-tunnelling-induced ground movements. Settlements arising from tunnel A construction were well represented by Gaussian distributions, as might be expected for a greenfield construction. However, tunnel B settlements were not. The main features of the results are listed below.

(*a*) The relative increases in settlements due to tunnel B compared with tunnel A were best described by an increase in the volume loss (given as a percentage). This effect was lessened by larger spacings between the tunnels.
(*b*) The increase in volume loss could be observed at the surface and at depths within the models.
(*c*) At the surface, the trough width parameter towards tunnel A was observed to be wider than a single tunnel (i.e. K was

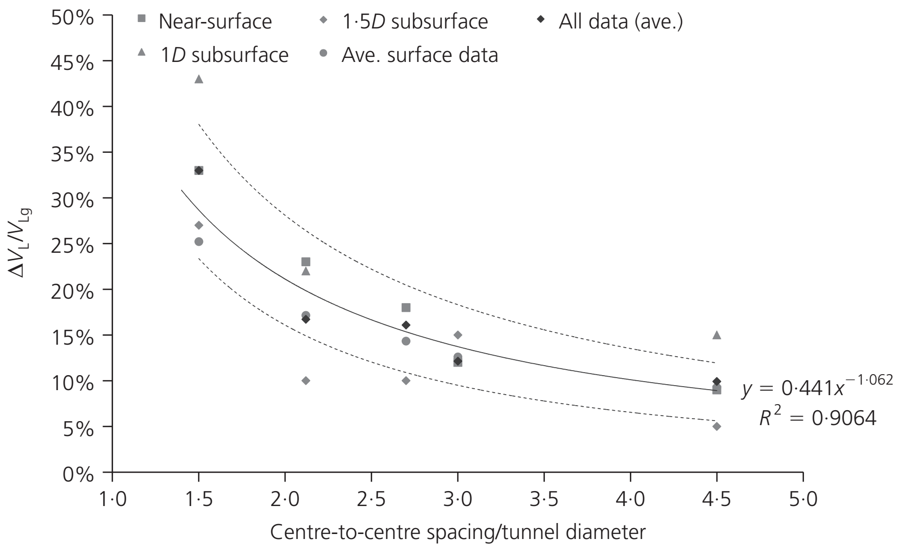

Figure 7. Increases in tunnel B volume loss in comparison with tunnel A plotted against the separation distance

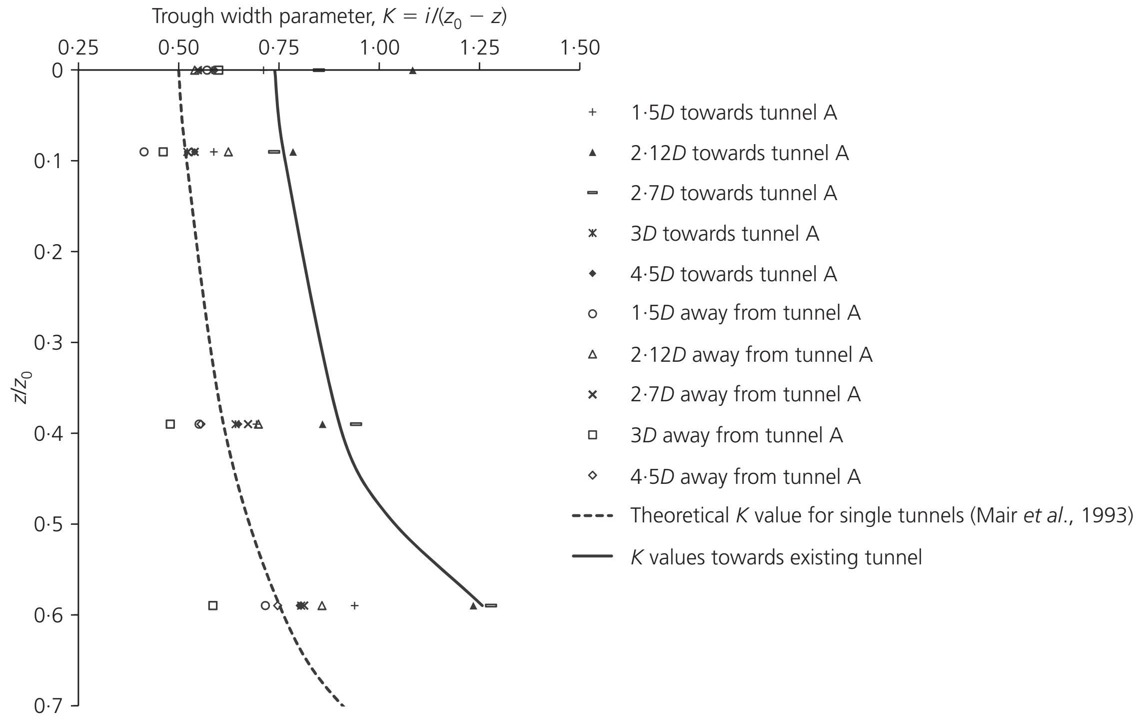

Figure 8. Variation of K values plotted with depth

found to increase) for spacings less than 3*D* or in offset arrangements.

(*d*) Twin-tunnelling settlement predictions can be improved by modifying the settlements solely attributable to the second tunnel construction. The second tunnel settlements can be predicted using equations by Peck (1969), O'Reilly and New (1982) and Mair *et al.* (1993), but with the modifications detailed in this paper.

The rationale behind these observations could be a reduced stiffness within certain areas of the soil mass. The volume of fluid removed from each tunnel was the same and therefore the system should be displacement controlled. The amount of soil being mobilised by the removal of the water is the same and therefore the stress change should be the same. Hence, a change in stiffness would allow for a greater magnitude of displacement in the soil associated with the second tunnel construction.

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