Interaction between multiple tunnels in soft ground

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ABSTRACT: An important aspect of planning for shallow tunnelling under urban areas in soft ground is the determination of likely surface movements and interaction with existing structures. Considerable research has been undertaken of this problem for the case of a single tunnel but little analysis exists for the case of multiple tunnel construction. This paper describes numerical analyses of multiple tunnelling undertaken using the commercial finite element software Plaxis. The aim of the study is to improve prediction of minimum interaction distance between multiple driven tunnels at shallow depths in soft ground.

1 INTRODUCTION

The need for tunnel design and construction in urban areas, mainly for transportation purposes, has increased markedly in recent years, especially in Europe. New tunnels are often required in close proximity to existing tunnels and construction must be carried out without damage either to the buildings above the excavation or to subsurface infrastructure. During the design stage it is therefore necessary to predict possible interaction effects.

The surface settlements, S above a single tunnel constructed in soft ground are usually assumed to follow an inverted Gaussian curve, i.e.

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

where S_{max} is the maximum settlement (over the tunnel axis), x is the orthogonal distance from the tunnel axis and i is the width of the settlement trough (Attewell & Farmer, 1975).

The source of these settlements is the "volume loss" which occurs at the tunnel. It is defined as the additional volume of soil which is excavated over the volume required to house the final lining. As excavation proceeds, the soil ahead of the face is unloaded and tends to move inwards. Losses also occur behind the face due to the nature of the shield in which the excavation is being carried out. Many field studies have confirmed Equation 1 to be acceptable for greenfield sites (Mair et al., 1993; Atkinson & Mair, 1981) but where structures are present, as in urban situations, Equation 1 is no longer valid.

For multiple tunnels, settlements from each are calculated according to Equation 1 and summed. This however ignores the interaction between tunnels during their construction. It is clear that the disturbance associated with tunnel construction must change the properties of the surrounding soil, and hence alter the effect of a subsequent tunnelling operation through that zone of soil.

Consider a multiple tunnelling scheme, with two parallel tunnels. Due to construction of the second tunnel, the first tunnel and the surrounding soil may move as a rigid body. The redistribution of stress creates an effect which is known as "arching" around the second tunnel. This has as a consequence the load removal from the tunnel. In other words, a reduction in earth pressure (Hansmire 1984). Furthermore, if the second tunnel is in close proximity there will be lining distortion and displacements to the first tunnel. The minimum distance between the tunnels, so as not to avoid interaction effects, clearly varies according to position and soil properties.

In the past, researchers have used both physical and numerical models to study tunnel interaction. Ghaboussi & Ranken (1977) performed a series of two dimensional finite element (FE) analyses of multiple tunnels using a linear elastic soil model. They reported that interaction effects were small at a pillar width (i.e. the clear space between the outside of two tunnels) of one tunnel diameter (1D). However, at a pillar width greater than 2D there was no apparent interaction. Therefore the tunnels could in this case be considered as independent and settlements calculated accordingly. These authors also found that the surface settlements created by the excavation of the second tunnel were higher than those created by the first.

Kim et al. (1998) performed reduced-scale physical model testing of parallel tunnels. For pillar widths greater than 1.5D the interaction effects were found to be small. Addenbrooke & Potts (2001) performed two-dimensional FE analyses of multiple tunnels using a non-linear elastic-perfectly plastic soil model. They concluded that for tunnels side-by-side, interaction effects only became negligible for a pillar width greater than 7D. On the other hand, for the "piggy-back" situation (where the tunnels axes are vertically aligned) the pillar width at which interaction ceased was 1D. However, when the second tunnel was driven below an existing tunnel, interaction always occurred, regardless of the depth of the former. Recent experimental data from a three station tunnel construction close to existing tunnels on the Piccadilly line in London indicated no interaction for pillar widths beyond 6D and 7D (Cooper et al., 2002).

The range of results outlined above indicates considerable disagreement as to the minimum pillar width (for any arrangement of tunnels) beyond which interaction effects can be ignored. A further contribution to resolve this disagreement is made in this paper. The study presented below comprises a series of numerical simulations of multiple tunnelling using the FE software Plaxis. Instead of examining the surface settlement profiles exclusively, evidence of interaction is also sought from observing the predicted tunnel lining shapes.

2 ANALYSIS GEOMETRY

Three cases of parallel multiple tunnelling geometries are studied here (Fig. 1):

a. Tunnel axes are horizontally aligned
b. Tunnel axes are vertically aligned: second tunnel driven above the first
c. Tunnel axes are vertically aligned: second tunnel driven below the first

In all cases the tunnels are assumed straight and circular with a diameter, D = 4.174m, to match the work in Addenbrooke & Potts (2001). The soil stratigraphy modelled in this study was kept constant throughout and is shown in Figure 2. It is identical to that used by Addenbrooke and Potts (2001) and relates to a site in London. The only modification made in this study was to extend the distance to the lower boundary in the FE model.

In geometry (a) the axis depth was 34m and the pillar width distance was varied from 0.5D to 7D. The calculations were repeated for a tunnel depth of 14m. In geometry (b) pillar width (now a vertical

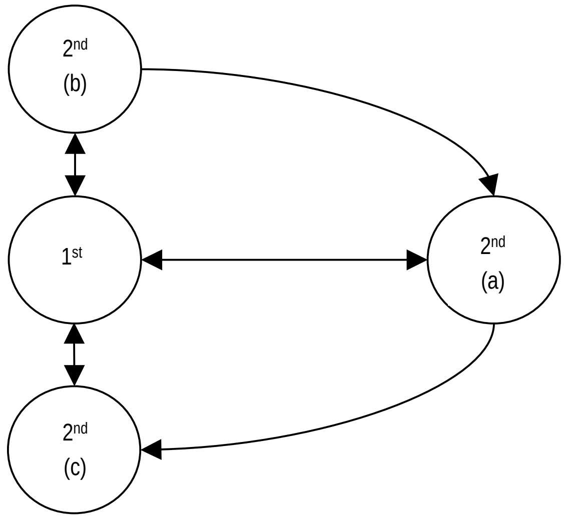


Figure 1. Three different positions of the second driven tunnel

distance) was varied from 0.5D to 4D. The upper tunnel depth was kept constant at 14m while the lower tunnel position was varied. In geometry (c) the pillar width was varied from 0.5D to 7D. Once again the upper tunnel depth was kept constant at 14m while the position of the lower tunnel was varied.

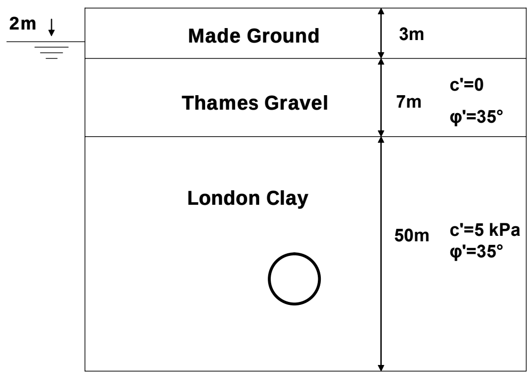


Figure 2. Soil stratigraphy assumed throughout this study

The layer of Made Ground (see Figure 2) was modelled as linear elastic with the water table at a depth of 2m. Both the Thames Gravel and London Clay were modelled as elastic-perfectly plastic (using the Mohr-Coulomb failure criterion) with increasing elastic stiffness with depth ($E = E_{ref} + z\ E_{inc}$). The interface between the London Clay and the tunnel lining was simulated as impermeable.

The tunnels were modelled using curved beam finite elements attached to the surrounding soil. Various methods are available for numerical modelling of volume loss as described elsewhere (e.g. Burd et al., 2000; Potts & Zdravkovic, 2001) and in this study the lining was shrunk using nodal forces applied to the lining elements. Throughout, a value of volume loss of 1.4% was specified, being a reasonable value found in practice in this stratigraphy (Addenbrooke & Potts, 2001). The material properties of the lining and the soil are presented in Tables 1 and 2.

Table 1. Material properties of the tunnel lining

Type of behaviour: Linear Elastic			
Parameters	name	value	unit
Normal Stiffness	EA	4704000	kN/m
Flexural rigidity	EI	11064	kNm^2/m
Thickness	d	0.168	m
Unit weight	γ_s	24	kN/m^3
Poisson's ratio	v_s	0.3	-
Volume Loss		1.4%	-

Table 2. Material properties of the soil

Soil:		Made Ground	Thames Gravel	London Clay	
Type of behaviour:		Linear Elastic	Mohr Coulomb	Mohr Coulomb	
Parameters					
Dry soil weight	γ_{dry}	18	17	18	kN/m^3
Wet soil weight	γ_{wet}	20	20	20	kN/m^3
Permeability	k	10^{-5}	10^{-5}	10^{-10}	m/s
Young's modulus	E_{ref}	5000	5280	6207	kPa
Poisson's ratio	v	0.3	0.32	0.33	-
Cohesion	c	-	0	5	kPa
Friction angle	φ	-	35^0	25^0	-
Dilatancy angle	ψ	-	17.5^0	12.5^0	-
Effect. stress ratio	Ko	0.5	0.5	1.5	-

In all calculations drained analysis was performed meaning that no excess pore water pressures were generated. Consequently, strength parameters based on effective stress were used as indicated in Figure 2. The results therefore represent the settlements likely to occur over a long period of time. For the boundary conditions no horizontal or vertical movements were permitted at the horizontal boundary at the base of the mesh. On the vertical mesh boundaries, only vertical movements were permitted.

In all calculations, the analysis procedure began with the definition of initial effective stresses prior to tunnel construction, using a value of the ratio of effective horizontal to vertical stress, K_0 as specified in Table 2. Subsequently, four load stages were defined.

1. Construction of the first tunnel, simulated by activating the tunnel lining and deactivating the soil elements inside the first tunnel.
2. Imposition of volume loss and activation of the tunnel lining for the first tunnel.
3. & 4 repetition of steps 1 & 2 for the second tunnel.

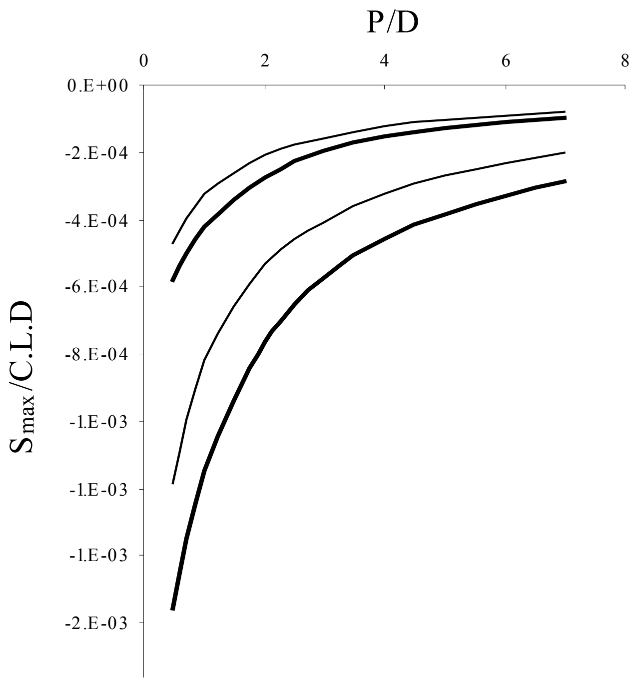
Each load stage was solved using standard non-linear solution techniques available in Plaxis, based on the Modified Newton-Raphson method.

3 RESULTS

3.1 *Side-by-side geometry (a)*

In the following, only the settlements of the second driven tunnel are examined. The settlements due to the first excavation are considered as the zero datum.

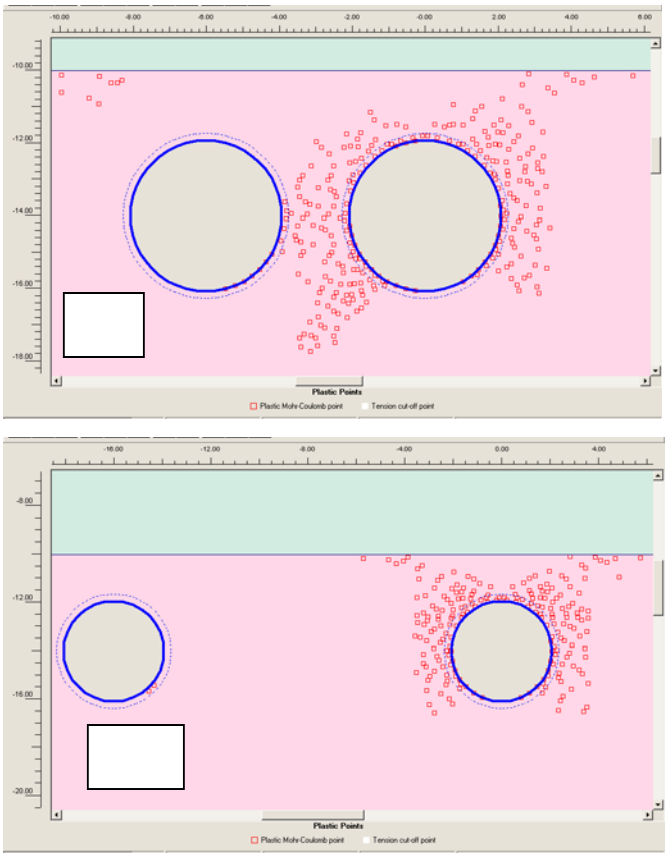
Results from a first parametric study are shown in Figure 3. This study examined the effect of excavation depth and elastic soil stiffness on interaction. In Figure 3**,** the maximum surface settlement over the second tunnel is plotted normalized by centre-line to centre-line distance (S_{max}/L) against pillar width normalized by tunnel diameter (P/D). The two curves for each depth (14m and 34m) are similar. As pillar width increases, the effect of differing stiffness decreases markedly. For a deep tunnel, the predicted settlement ratios are almost the same. The



slope of these curves also diminishes markedly beyond a pillar width ratio of 3 to 4, indicating, perhaps, a loss of interaction at this distance.

Figure 4 shows the "plastic points" (i.e. integration points in elements where stresses are on the yield surface) for sample calculations. As pillar width increases the zones of yield from the second tunnel cease to overlap with the first tunnel. The effect is much more marked for the less stiff soil. The size of these yield zones is discussed further below.

Displacement vectors for points on the tunnels' springlines are shown for these analyses in Figure 5. (In each case the second tunnel is shown to the right). While the vertical components of these vectors are similar, the horizontal components are in different directions. At low soil stiffness, the first



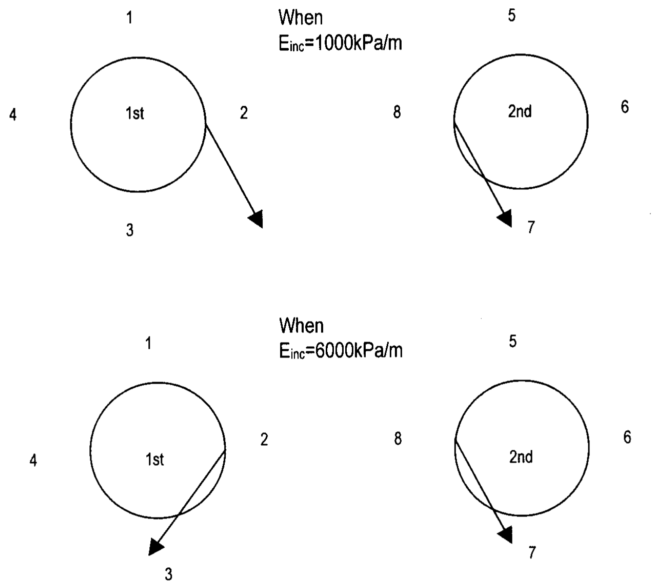
tunnel moves towards the second, while for high stiffness the first tunnel moves away. Similar results were found when the two tunnels were excavated at the shallower depth (14m). The reason for this change of direction of displacement vector at the springing of the first tunnel can be explained by imagining that in the lower soil stiffness analysis, the tunnel lining stiffness is relatively much higher (as compared to soil stiffness) than in the latter case. It therefore acts as a rigid body floating in the relatively flexible soil. Plots of the tunnel lining shape following excavation of the second tunnel bear this out. For the stiffer soil, the nodal forces required to produce the same volume loss must be greater. Horizontal stresses must therefore also be greater and the zone of yielding around the tunnel spreads wider. With yielding comes unloading, allowing the first tunnel to move away instead of towards the second tunnel. The experimental programme carried out by Kim et al. (1998) found similar behaviour.

2.1 *Piggy-back geometries (b) & (c)*

Figure 6 shows results from modelling of geometry cases (b) and (c), using the same axes as for Figure 3. Once again, the stiffer the soil, the higher the surface settlement ratio. In the case of geometry (c) these settlements are seen to reduce with increasing pillar width ratio, as in the results for geometry (a) above.

Another interesting feature of Figure 6 is that settlements appear to be greater for closely spaced tunnels when the upper tunnel is excavated first (geometry (c)). However as pillar width increases (geometry (b); lower tunnel first) settlements are more significant than for equivalent geometry (c).

Figure 7 shows the effect of soil stiffness on the nature of distortion of the first tunnel lining due to construction of the second tunnel above. For a stiff soil the lining is subjected to severe bending defor-

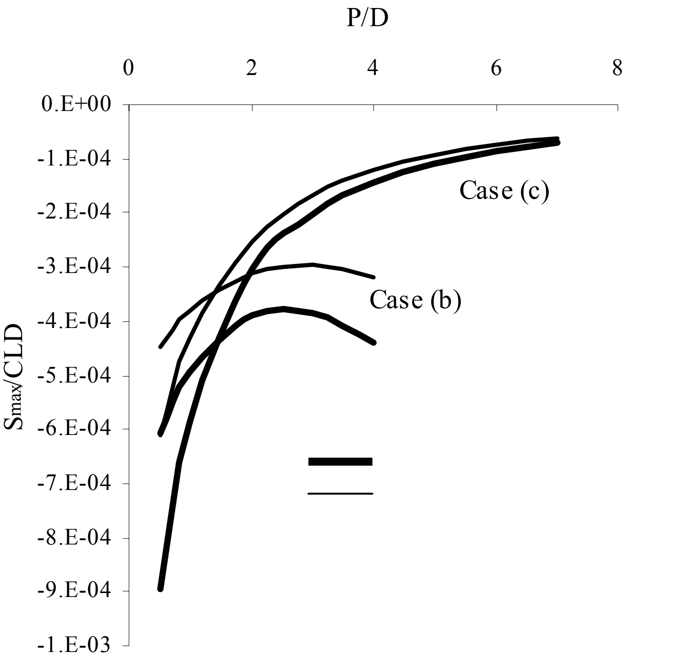


mations as compared to the tunnel in the less stiff soil. Once again, the ratio of soil/lining stiffness is important. In a flexible soil, the tunnel lining is less constrained and reacts in a general form to the uplift created by the tunnel excavation above (from unloading of the soil and from the modelling of volume loss).

The general behaviour in geometry (c) layouts (i.e. second tunnel constructed beneath the first) is for the first tunnel to be dragged downwards as a rigid body (maintaining its circular shape). Minor distortions to the first tunnel shape are evident only at very low pillar widths. The second tunnel lining shape is, however, significantly distorted from circular (which it would be for an isolated tunnel) in this geometry. The lining shape is elliptical with the major axis aligned vertically (as shown in Figure 8). This indicates continuing interaction at high pillar widths.

For geometry (c) it appears that the first tunnel acts as a near rigid inclusion in the soil above the second tunnel, leading to reinforcement in the vertical direction and forcing the second tunnel lining to distort more along its horizontal axis.

A conclusion to be drawn from these results is that for the case of a second piggy-back tunnel excavated above the first, interaction soon disappears as pillar width increases. This is understandable as the disturbance to the insitu stress field associated with the first tunnel is now confined to an area below the location of the second tunnel. After P=3D the insitu stresses at the location for the second tunnel are little different to the case of a single tunnel at that depth, and hence the surface settlements arising from this second tunnel are similar to those from a single isolated tunnel. Similar conclusions were reached by Addenbrooke & Potts (2001).

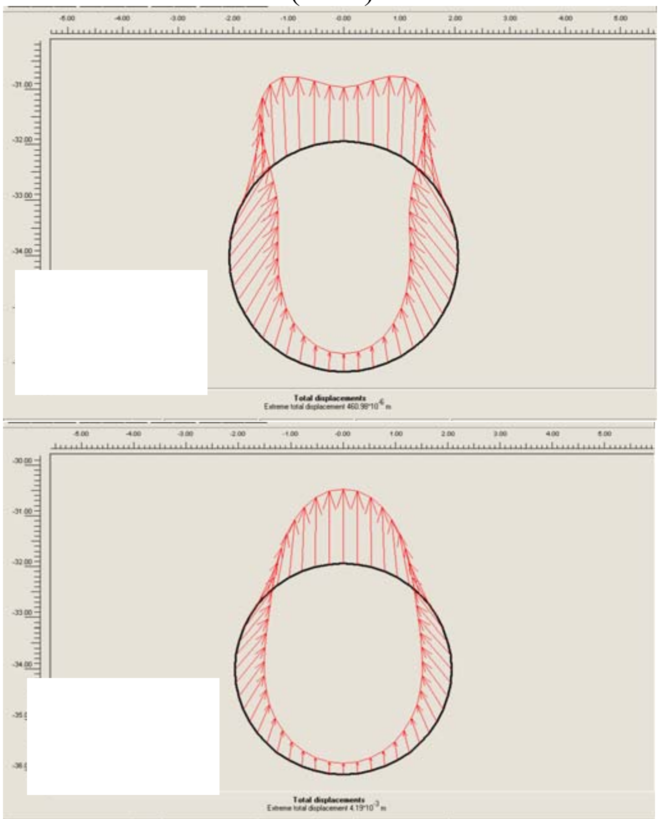


In the case where the upper tunnel is constructed first (geometry (c)), there appears to be no 'safe' minimum distance where interaction effects cease to occur. That is, whatever the depth of the second tunnel, its behaviour is affected by the presence of the first tunnel above.

4 CONCLUSIONS

This study provides some new guidance on possible interaction effects between twin tunnels for parallel and piggy-back geometries. In the former case interaction effects appear to be present up to a pillar width of three to four diameters. These values agree with those reported by Ghaboussi & Ranken (1977) and Kim et al. (1998) but appear conservative in comparison to the values reported by Addenbrooke & Potts (2001) and Cooper et al. (2002).

For the piggy-back geometry where a second tunnel is driven above an already existing one, small interaction effects seem to be present at a pillar depth distance of one diameter but beyond three diameters there seems to be negligible interaction. For the piggy-back geometry where the lower tunnel is constructed second interaction effects seem to appear no matter how deep the second tunnel was driven. These values agree well with what was reported by Addenbrooke & Potts (2001).



Clearly, this study is based on a particular soil stratigraphy so that conclusions for other soil profiles may differ. Very simple elasto-plastic soil models have also been used, further affecting the results. However, the results obtained fit in with previous studies (both numerical and physical) and demonstrate that economical numerical modelling of this nature, for parametric studies, is valid.

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