TWIN-TUNNELLING INDUCED CHANGES TO CLAY STIFFNESS

DIVALL S, GOODEY RJ, & STALLEBRASS SE

ABSTRACT

Tunnels used for transportation in the urban environment are often constructed in pairs. Projects where tunnels are constructed sequentially and within a close proximity are referred to as 'Twin-tunnelling'. Case studies and recent research indicate that prediction of settlements for such a scheme cannot be determined using existing simple methods derived from consideration of a single tunnel. To establish the reasons for the observed variation in settlements, a series of centrifuge tests was undertaken on various twin-tunnel arrangements in overconsolidated clay. The tests consisted of preformed cavities from which a specific quantity of supporting fluid could be drained, with precision, creating a predetermined magnitude of tunnelling volume loss. Data were obtained for surface and subsurface displacements, changes in pore-water pressure near the tunnels and the support pressure within the tunnels. Systematic use of cavity contraction models was found to be an informative method of explaining the observations. Using an elastic, perfectly-plastic cavity contraction model coupled with the observations from the experiments enabled the shear stiffness of the clay around the tunnel to be described. Further analysis demonstrated a reduction in shear stiffness of the soil prior to and during the second tunnel excavation explaining the increase in volume loss observed in that event.

Centrifuge modelling, Ground movements, Stiffness, Tunnels & tunnelling

NOTATION

a	tunnel radius
C	tunnel cover
d	centre-to-centre tunnel spacing
D	tunnel diameter
G	shear modulus
i	horizontal distance from the tunnel centre-line to the point of inflexion on the settlement curve
K	trough width parameter (equal to 0.5 for clays)
m	average gradient of the graph of δ_r/a plotted against a/r
N	stability ratio
r	radial distance from centre of tunnel
R_p	radius to the “plastic zone”
S_u	undrained shear strength
S_v	settlement at any point
Δu	change in pore-water pressure
V_L	volume loss (usually %)
V_{Lg}	volume loss in the greenfield or first tunnel construction case (usually %)
V_s	volume of surface settlement trough
x	horizontal distance from the tunnel centre-line
z	depth below ground surface

z_0 depth to tunnel axis level

δ_r radial movement at radius, r

γ_s bulk unit weight of soil

γ shear strain

σ_T tunnel-support pressure

τ shear stress

1. Introduction

Congested urban environments have a long history of utilising underground construction. In particular, tunnelling has been used to create efficient transport links, communication systems and water supplies. When considering transport links, tunnels are commonly constructed in pairs (with associated cross-passages) both for practical and safety reasons. The separate tunnel cavities are often constructed sequentially within a relatively short space of time and within reasonably close proximity. This type of construction is known as twin-tunnelling.

Independent of the construction method adopted, tunnelling inevitably causes ground movements which, in urban areas, have the potential to cause damage to existing infrastructure (Burland, 2001). Current practice for predicting tunnelling-induced settlements is based on observations from field studies and laboratory tests (Mair *et al.*, 1996). The values/parameters used in these predictions of settlements are often based on past-experience and influenced by the tunnelling method (e.g. Peck, 1969 or O'Reilly & New, 1982).

Many case studies (e.g. Cording & Hansmire, 1975; Standing *et al.*, 1996; Lo *et al.*, 1987; Cooper & Chapman, 1998; Nyren, 1998; and Shirlaw *et al.*, 1988) have reported a relative difference in the settlements associated with the first and second tunnel constructions. Previous research (e.g. Addenbrooke & Potts, 2001; Hunt, 2005; Fang *et al.*, 1994; and Divall & Goodey, 2015) have attempted to define the conditions that would ensure there would be no interaction between the tunnels. These studies related relative differences in the tunnelling-induced settlements with the centre-to-centre spacing or pillar width between cavities.

Barlett & Bubbers (1970) suggested that there would be a weakening of the strata between the two tunnels resulting in more pronounced settlement and distortions in the tunnel lining. Mair & Taylor (1997) had also stated that significant interaction effects should be evident when tunnels are very closely spaced. Standing & Burland (2006) commented that establishing the reasons for these unexpected settlements is essential for future tunnelling proposals.

The principal aim of the research presented in this paper is to develop a greater understanding of the behaviour of the ground 'shared' by closely spaced tunnels in clay, i.e. soil where there are significant changes in stress and strain during both tunnel excavation events. In particular, evaluating any 'weakening of the strata' or reduction in strength or stiffness and the effect of this on the resulting tunnelling-induced settlements. The scope of this work has been limited to movements in a plane perpendicular to tunnels driven in clay.

2. CURRENT PRACTICE FOR UNDERSTANDING TWIN-TUNNELLING-INDUCED GROUND BEHAVIOUR

2.1 Tunnelling-induced ground movements

The ground movements resulting from the construction of bored tunnels, in the case of a single tunnel in 'greenfield' conditions, are well-defined (Peck, 1969; Cording & Hansmire, 1975; Clough & Schmidt, 1981; O'Reilly & New, 1982 and Cording, 1991). These ground movements have been idealised to a Gaussian distribution curve in shape. This shape is usually described by the expression given below;

$$\mathrm{S_v} = \left(\sqrt{\frac{\pi}{32}}\frac{V_L D^2}{i}\right)\cdot \exp\left(\frac{-x^2}{2i^2}\right) \qquad \text{Equation (1)}$$

Where it is often convenient to express the Volume Loss as a percentage of the excavated area of tunnel.

$$\mathrm{V_L} = \frac{V_s}{(\pi D^2)/4} \qquad \text{Equation (2)}$$

Many case studies (i.e. Cording & Hansmire, 1975; Standing *et al.*, 1996; Lo *et al.*, 1987; Cooper & Chapman, 1998; Nyren, 1998 and Shirlaw *et al.*, 1988) have reported a relative increase in the Volume Loss associated with the second tunnel construction compared with the first (or 'green field' situation). This would imply that different values for the relative Volume Losses would need to be incorporated into an expression similar to Equation (3) to give an accurate reflection of the twin-tunnelling response;

$$S_v = \sqrt{\frac{\pi}{32}}\cdot\left[\frac{V_{L\,A}D_A^2}{K_A z_{0A}}\cdot exp\left(-\frac{{x_A}^2}{2K_A z_{0A}}\right)+\frac{V_{L\,B}D_B^2}{K_B z_{0B}}\cdot exp\left(-\frac{(x_A-d)^2}{2K_B z_{0B}}\right)\right] \qquad \text{Equation (3)}$$

where the subscripts *A* and *B* refer to the first and second bored tunnel respectively.

An engineer could estimate the settlements above a twin-tunnel arrangement using Equation (3) given details of the geometry, geology and values for the Volume Losses. Therefore, guidance on what values would be appropriate to use for the Volume Losses, which are the key variables in Equation (3) is required. The relative differences in the Volume Losses have only recently been studied (e.g. Addenbrooke & Potts, 2001; Hunt, 2005 and Divall & Goodey, 2015). These studies offer a means by which the magnitude of the increase in Volume Loss can be ascertained based on the tunnel centre-to-centre spacing. Figure 1 shows the overall trend presented by Divall & Goodey (2015) along with three case studies with some selected field measurements. These case studies (Cooper & Chapman, 1998; Cording & Hansmire, 1975; Nyren, 1998) are consistent with the experimental data supporting the relationship proposed. Analysis of the detailed subsurface ground movements measured during the centrifuge model tests can explain these observations. Therefore, it is important to understand the reasons for these increases in Volume Loss to improve settlement estimations.

2.2 Relationship between movements and changes in ground stress

Any observed ground movements are caused by changes to the stress-state in the ground. In tunnelling projects, this stress change is due to removal of soil from within the soil mass. Tunnels constructed by closed-face tunnel boring machines minimise movements by supporting the soil mass via a pressurised face before the permanent lining is installed. The tunnel support pressure has previously been related to the observed settlements by case studies (Macklin, 1999) or via plasticity solutions (Mair & Taylor, 1993). Both studies used the concept of a Stability Number, N (after Broms & Bennermark, 1967).

$$\mathrm{N} = \frac{\gamma_s z_0 - \sigma_\mathrm{T}}{\mathrm{S_u}} \qquad \text{Equation (4)}$$

Whilst it should be noted that the concept of Stability Number was primarily developed to investigate the undrained stability of headings, it has been incorporated subsequently into studies concerned with pre-collapse movements. The relationship between tunnelling-induced changes in stress and subsurface settlements were investigated extensively by Mair & Taylor (1993). Their study produced closed-form plasticity solutions for an unloading cylindrical cavity representative of a tunnel.

The analysis assumed undrained behaviour, an initial isotropic stress state and geometric axisymmetry. Using these assumptions, Mair & Taylor (1993) formulated a relationship between subsurface displacement and the Stability Number for a tunnel constructed in a linear elastic-perfectly plastic continuum.

$$\frac{\delta_r}{a} = \frac{S_u}{2G}\left(\frac{a}{r}\right)\exp(N-1) \qquad \text{Equation (5)}$$

Mair & Taylor (1993) concluded by stating that the predictions of ground movements obtained from a linear elastic-perfectly plastic soil model provided reasonable approximations to observed field and experimental data. It was acknowledged that the representation of the soil as linear elastic-perfectly plastic was a significant assumption and that further assumptions would need to be made regarding the soil stiffness. It follows that any second tunnel construction would need to make similar assumptions which, in reality, would be influenced by the stress changes arising from the construction of the first tunnel.

2.3 Tunnelling-induced soil stiffness response

It is generally accepted that there is significant interaction between adjacent tunnel constructions. This is especially true for tunnels in clay where recent stress history has been shown to have a significant effect on stress-strain behaviour (Atkinson *et al*, 1990). During tunnel construction, the ground in the surrounding region will have been subject to ‘appreciable’ shear strains (Mair & Taylor, 1997). The development of these shear strains leads to the degradation of soil stiffness observed by Atkinson & Sallfors (1991) and presented with respect to tunnelling problems by Mair (1993). It is reasonable to assume that a second tunnel constructed in ground close to the first tunnel may be in an area of soil with a changed stiffness. Changes in stiffness that may have occurred during the first tunnel construction have the potential to influence the volume loss, maximum settlement and distribution of settlements when the second tunnel is constructed (Mair & Taylor, 1997). However, the magnitude of any changes in the stiffness are not generally known (Wright, 2013).

3. OBSERVATIONS OF ADDITIONAL MOVEMENTS AND VOLUME LOSS RELATING TO TWIN TUNNELS

Divall (2013) investigated the relationship between tunnelling-induced settlements and the spacing between sequentially bored twin-tunnels. The results from eight plane strain centrifuge tests were described by Divall & Goodey (2015). The tests consisted of an overconsolidated clay sample with two preformed circular tunnels bored equidistant from the model centreline. The typical centre-to-centre spacing was 1.5, 3 and 4.5 times the tunnel diameter. All models had a cover to diameter (C/D) ratio equal to 2 and the tunnels' axis-levels were approximately 80mm above the base of the model. The tests were performed at 100g which made the tunnels comparable, in terms of stress similarity, with 4m diameter tunnels at prototype scale. Figure 2 shows a schematic of the typical model geometry tested and Table 1 gives some basic details of the tests performed. Divall & Goodey (2012) describe the equipment which facilitates the simulation of sequential tunnel constructions. Essentially, the cavities were supported with water and a precise amount of this fluid was removed equal to the magnitude of volume loss desired. Equal amounts of water were removed from each tunnel after a 'construction delay' of 3 minutes. Divall & Goodey (2015) concluded that the relative increases in settlements associated with the excavation of the second tunnel were best described as increases in volume loss (given as a percentage) when compared with the greenfield or first constructed tunnel case, V_{Lg}. Furthermore, this relative increase in settlement lessened with greater centre-to-centre spacings (Figure 1).

4. DISCUSSION OF THE FACTORS GIVING RISE TO ADDITIONAL VOLUME LOSSES.

4.1 Introduction

Essentially, tunnelling is an unloading event within a soil mass and as the stresses increase or decrease the soil will change in shape. Stiffness can be characterised in a number of ways for soils, such as in terms of shear stress and shear strain.

$$G = \frac{d\tau}{d\gamma} \qquad \text{Equation (6)}$$

The shear modulus for soils generally reduces with increasing levels of shear stress and strain (Atkinson & Sallfors, 1991). Even at small strains the clay stress-strain response is elastoplastic and consequently its stiffness when loaded is influenced by the previous stress history (Atkinson *et al.*, 1990; Gasparre *et al.*, 2007). For an element of soil located between the tunnels, the stiffness will be influenced by stress changes arising from the excavation of the first tunnel. It would then be expected that this same element would respond differently to the stress changes arising from construction of the second tunnel. This is different to the stress history (and therefore response) of soil located away from the area surrounding the first tunnel.

4.2 Stress changes

As previously stated, the laboratory tests described by Divall & Goodey (2015) remove a precise amount of the supporting fluid which simulated tunnelling-induced ground movements. This action resulted in a change in the supporting fluid pressure because the volume of fluid within the tunnel has changed. The construction was deemed complete (the generated ground movements had stopped) when the pressure reached a minimum value. As well as large amounts of settlement data (Divall, 2013) the centrifuge tests provided measurements of tunnel support and pore-water pressures. Druck miniature pore-pressure transducers (PPTs) and Omega sub-miniature flush diaphragm pressure transducers (TPTs) were used to measure the changes in pore-water pressure and tunnel support pressure, respectively.

Table 2 and Figure 3 present the relative differences in tunnel support pressure from the centrifuge twin-tunnel tests. There are two notable features of these data; the largest difference between the change in support pressure of the first and second tunnel was generally under 10kPa and the rate of tunnel construction was almost identical for every simulated tunnel event. The stress change occurring during each tunnel construction simulation was, therefore, essentially the same because the driving force (weight of soil) above the tunnels did not change and the amount of support fluid removed was carefully controlled to a high level of precision.

Figure 4 shows the pore-water pressure response from two tests. The data presented are from a PPT set at the tunnel axis-level, mid-way between the two tunnels. The origin of the x-axis (time) is taken as the start of the first tunnelling event. The completion of the first tunnel and the start and end of the second tunnel construction event are shown. During each tunnelling event excess pore-water pressures were generated and some further changes occurred after tunnelling was complete due to local drainage within the model. By treating the construction of the tunnel as an undrained event values for excess pore-water pressure generated can be obtained; these are highlighted on Figure 4. In addition, Figure 4 also shows data from a 'far-field' PPT, the response of which during tunnelling is minimal.

4.3 Prediction of pore-water pressure changes using closed form solutions

As previously stated, Mair & Taylor (1993) presented closed-form plasticity solutions for the unloading of a cylindrical cavity and related the Stability Number to ground movements. This type of model is a useful idealisation that allows observed movements to be related to stress changes in the ground. These models idealised the soil behaviour as elastic – perfectly plastic where either linear or non-linear elasticity could be assumed. As well as ground movements, changes in pore-water pressure can be predicted and these predictions are compared with the data from the current tests to determine the validity of a closed-form solution as an interpretive tool.

Assuming linear elastic behaviour, the change in pore-water pressure can be calculated using the equations proposed by Mair & Taylor (1993), given as

$$\frac{\Delta \mathrm{u}}{\mathrm{S_u}} = 1 - \mathrm{N} + 2\log_e\left(\frac{\mathrm{r}}{a}\right) \text{ for } \mathrm{N} \geq 1 \text{ and } a \leq \mathrm{r} \leq \mathrm{R_p} \text{ where;} \qquad \text{Equation (7)}$$

$$\frac{\mathrm{R_p}}{a} = \exp\left(\frac{\mathrm{N}-1}{2}\right) \qquad \text{Equation (8)}$$

This model predicts that pore-water pressures will change inside a "plastic zone" around the tunnel and will be zero outside of that zone. The measured changes in pore-water pressure from all tunnel construction events in all tests (including some additional single tunnel tests used to verify the functionality of the equipment) are shown on Figure 5. A line representing the Mair & Taylor (1993) model is also shown but it should be noted that this varies depending on Stability Number, N, and S_u.

The majority of tests presented here had a value of N of approximately 2.5 and S_u of approximately 45kPa and these values have been used to generate the line shown. These values also indicate that the plastic zone extends to a distance of slightly more than one tunnel radius beyond the tunnel cavity.

As predicted by the model, the pore-water pressure changes measured in the tests outside of the plastic zone are minimal and those inside the plastic zone are well predicted when assuming a linear model for G. Based upon this observation it is reasonable to suggest that values for G in the tests might be determined by fitting experimental data to a model of this type.

In summary, each individual tunnelling event results in very similar measurements of changes in tunnel support pressure and generation of excess pore-pressures but a different response in terms of the displacements observed (both surface and subsurface). A change must have occurred in the soil state (i.e. the assumed stress history) for this to be possible. Specifically, the change must have occurred in the area previously unloaded by the first tunnel construction which is also within the region of the second tunnel construction.

4.4 Stiffness changes

In order to estimate soil stiffness (and any possible changes in stiffness) the tunnel is idealised as an axisymmetric contracting cavity and the soil is represented as a linear elastic – perfectly plastic material. Whilst it is accepted that this is a simplification of the behaviour of both the cavity and the soil, a model of this type allows investigation of the soil stiffness based upon the experimental data. Digital image analysis performed during the tests enables measurement of movements at any stage of the test. Figure 6 shows the radial movements towards the tunnel cavity during excavation simulation and are presented at model scale. The data shown are for the first tunnel excavated in a test where the volume loss was 5% and the tunnel spacing was 3D. These data clearly show that the assumption of axisymmetry is invalid due to the differing magnitudes of displacement but the form of the displacements is similar. Therefore, using these data to infer stiffness would allow the study of changes in stiffness if not the precise magnitude of stiffness.

Using Equations (4) and (5) (Mair & Taylor, 1993) it is possible to obtain values for G/S_u by fitting the tunnel pressure data and the subsurface displacement data from the centrifuge model tests. Given that values of S_u were measured after the tests, it is then possible to determine values for the shear modulus.

The first step was to interpret the subsurface movements around each tunnel from each test (of the form shown in Figure 6) on a graph with axes of (δ_r/r) against (a/r). Measurements are plotted of movements towards the tunnel along lines extending vertically above the crown, horizontally at the tunnel axis and at an angle of 45° between these. An example plot showing these data at the completion of tunnel construction for test SD14 is shown in Figure 7. Each of these data sets has a line of best fit and the gradient of this line can be related to S_u, G and N.

The Stability Number, N, can be calculated at any point from the measurements of tunnel pressure. Additionally, a value for the bulk unit weight of clay, γ_s, is required. Grant (1998) used 17.5kN/m^3, based upon many laboratory tests performed at City, University of London and this value is adopted here. The undrained shear strength, S_u, was obtained by post-test shear vane readings and was found to be in the region of 45kPa for all tests.

The average gradient from all data sets (of the type shown in Figure 7) was determined which, along with the stability ratio and undrained shear strength, was used to derive a value of G/S_u;

$$\mathrm{S_u} = \frac{\gamma_s z_0 - \sigma_\mathrm{T}}{1 + \ln[2\mathrm{m}(\mathrm{G}/\mathrm{S_u})]} \qquad \text{Equation (9)}$$

Where; m is the average gradient from a graph with axes of δ_r/a against a/r.

The same procedure can then be applied to the measurements of subsurface movements around the second tunnel. Figure 8 shows the values of G (plotted against the reduction in tunnel pressure) obtained from the analysis for centrifuge tests where the volume loss was 5% and the spacing between the tunnels was varied.

As might be expected, the values and trends obtained from construction of the first tunnel in all three tests is very similar. They show a reduction in stiffness as the test proceeds (i.e. as the levels of strain accumulate). In each test, during construction of the second tunnel, the soil stiffness inferred from the model is initially significantly higher (possibly associated with changes in the stress path direction) but rapidly degrades to a value that is lower than that obtained at the end of Tunnel A construction. These changes are relatively consistent in that this initial increase in stiffness is approximately 160% of the value obtained for Tunnel A and the value at the end of the tunnelling event has reduced to approximately 70% of the Tunnel A value. In the tests presented the degradation of stiffness is similar irrespective of tunnel spacing. This is due to the close proximity of the tunnels in all tests and stiffness only being determined in areas close to the cavity. It is expected that at very large spacings there would be no interaction, this being supported by the observations of Addenbrooke & Potts (2001) who investigated numerically spacings of up to 7D. It should be noted that results for Tunnel B at the spacing of 4.5D are not presented due to a failure in the tunnel pressure transducer.

5. CONCLUSIONS

Case studies and recent research projects have shown there to be a relative difference between the tunnelling-induced ground settlements of closely spaced tunnels. It is clear that there is a requirement for establishing the reasons for these unexpected settlements for future tunnelling projects.

Data from a series of well controlled centrifuge tests indicate that, whilst movements generated during individual tunnelling events are observed to be different, the total stress and pore-water pressure changes are not. Use of a linear-elastic perfectly plastic model, introduced by Mair & Taylor (1993), successfully predicts the excess pore-water pressure response observed in the experiments and gives confidence that these models provide a means by which the stiffness degradation during a simulated tunnel construction can be investigated. Fitting the experimental data to this model shows that there is a significant reduction in shear stiffness of the soil during the second tunnel construction event. It is the conclusion of this paper that the prediction of tunnelling-induced ground movements could be improved by accounting for the reduction in stiffness in the area ‘shared’ by both tunnel constructions.

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FIGURES

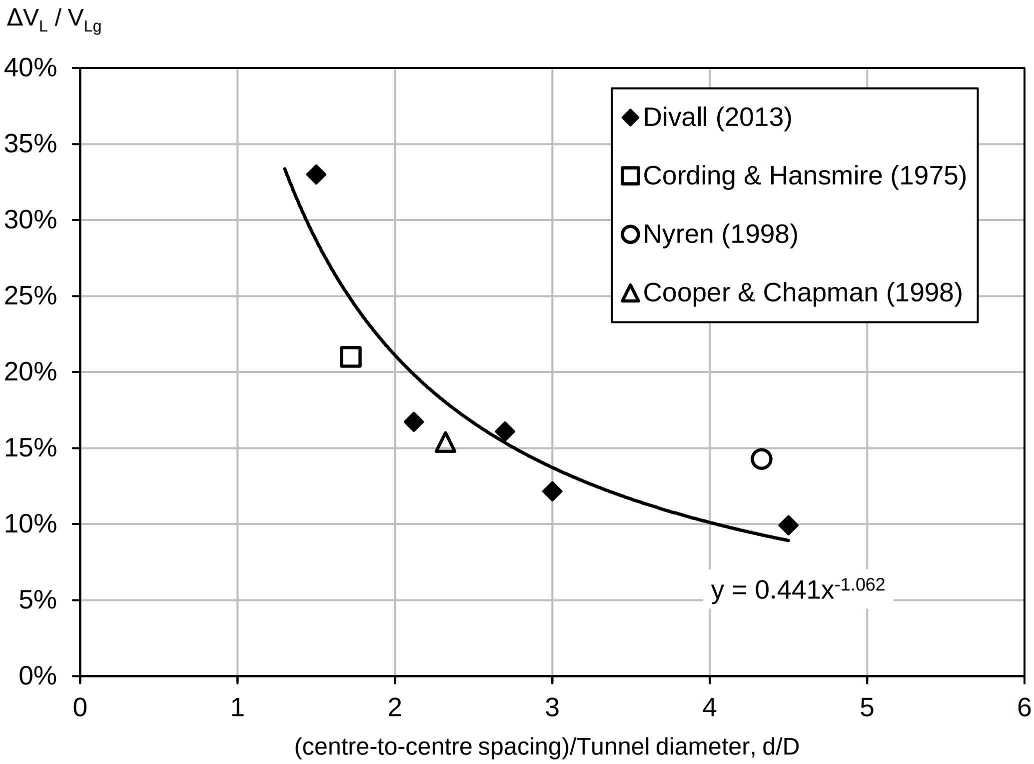

Figure 1: Trend of increased Volume Loss with tunnel spacing (after Divall & Goodey, 2015)

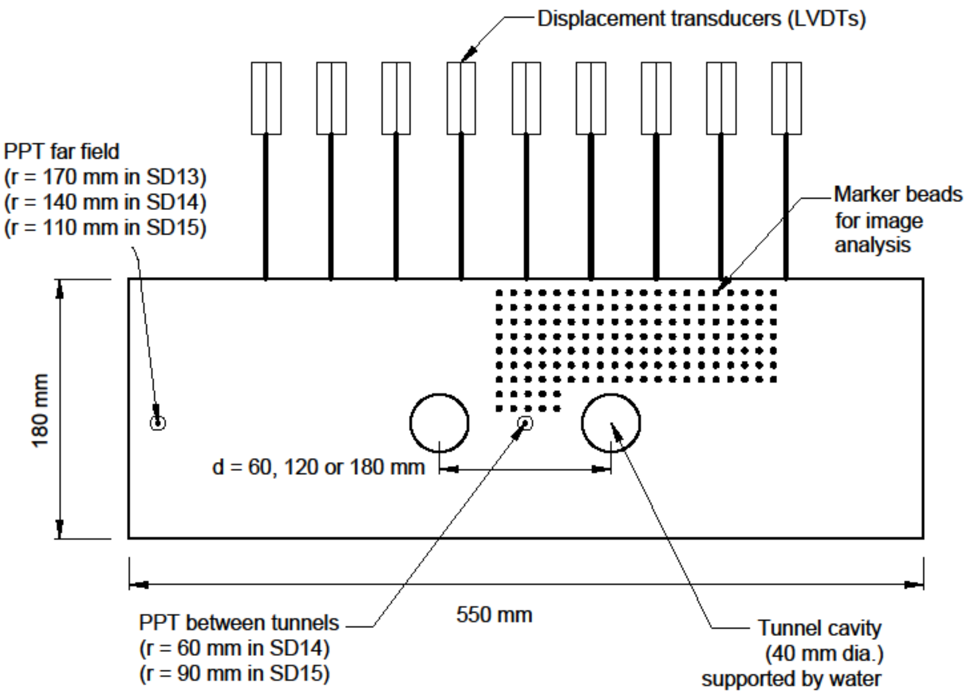

Figure 2: Typical model test (after Divall & Goodey, 2012)

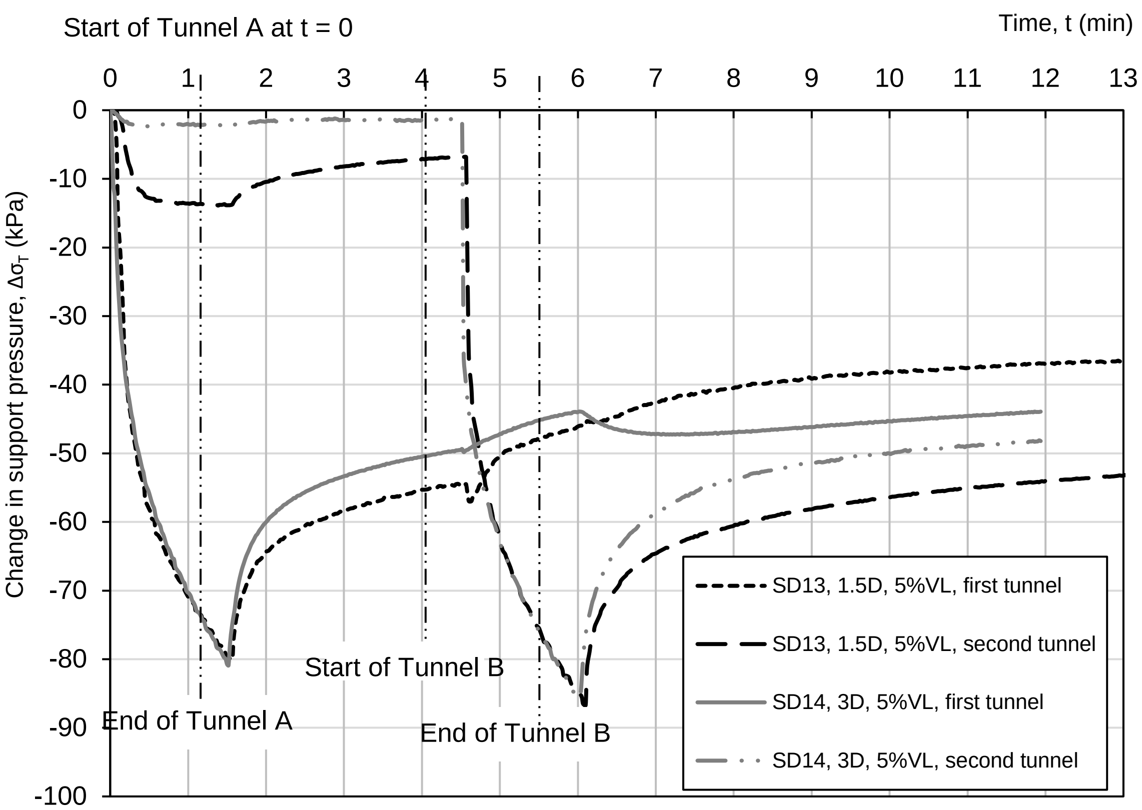

Figure 3: Tunnel support pressures against time for 5% Volume Loss tests

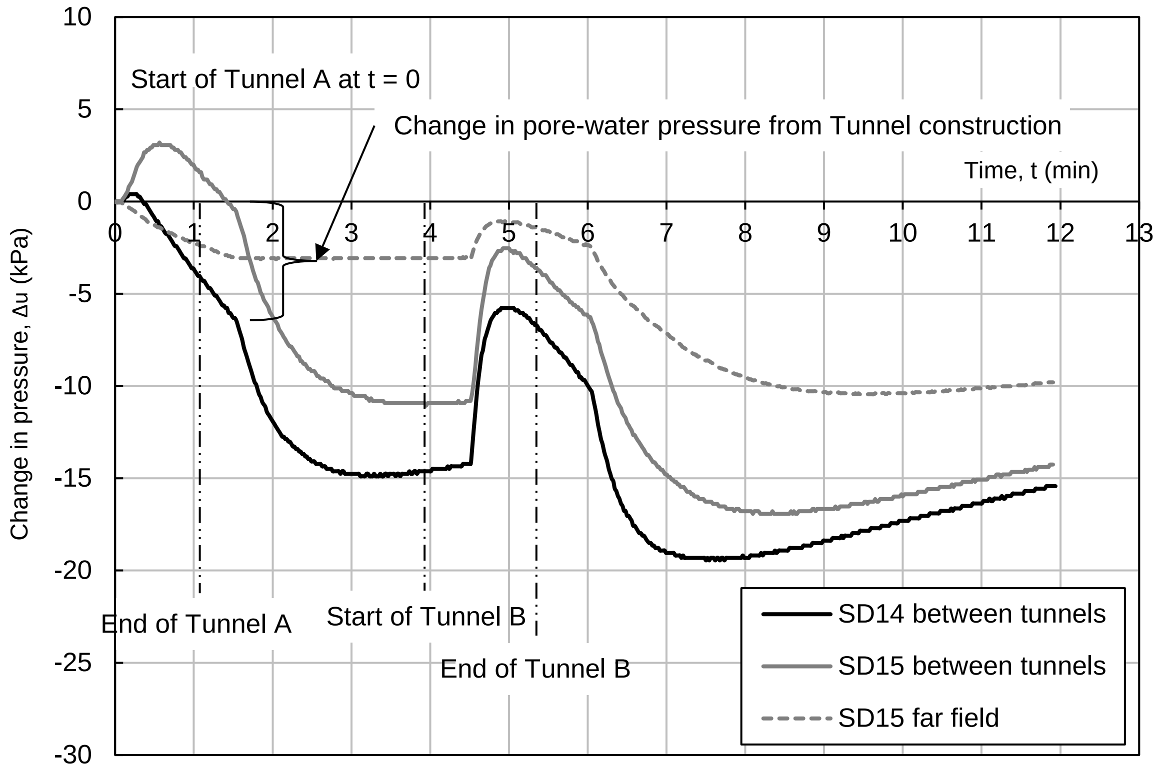

Figure 4: Pore-water pressure responses against time for 5% Volume Loss tests

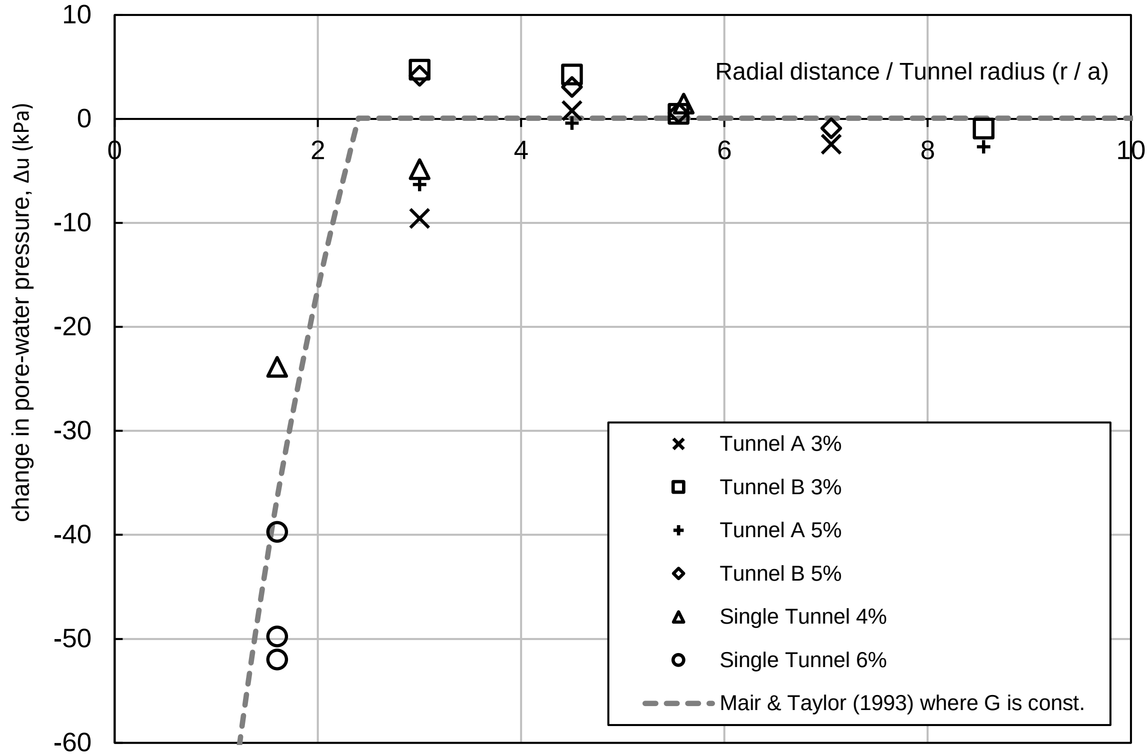

Figure 5: Pore-water pressure changes against Mair & Taylor (1993) prediction

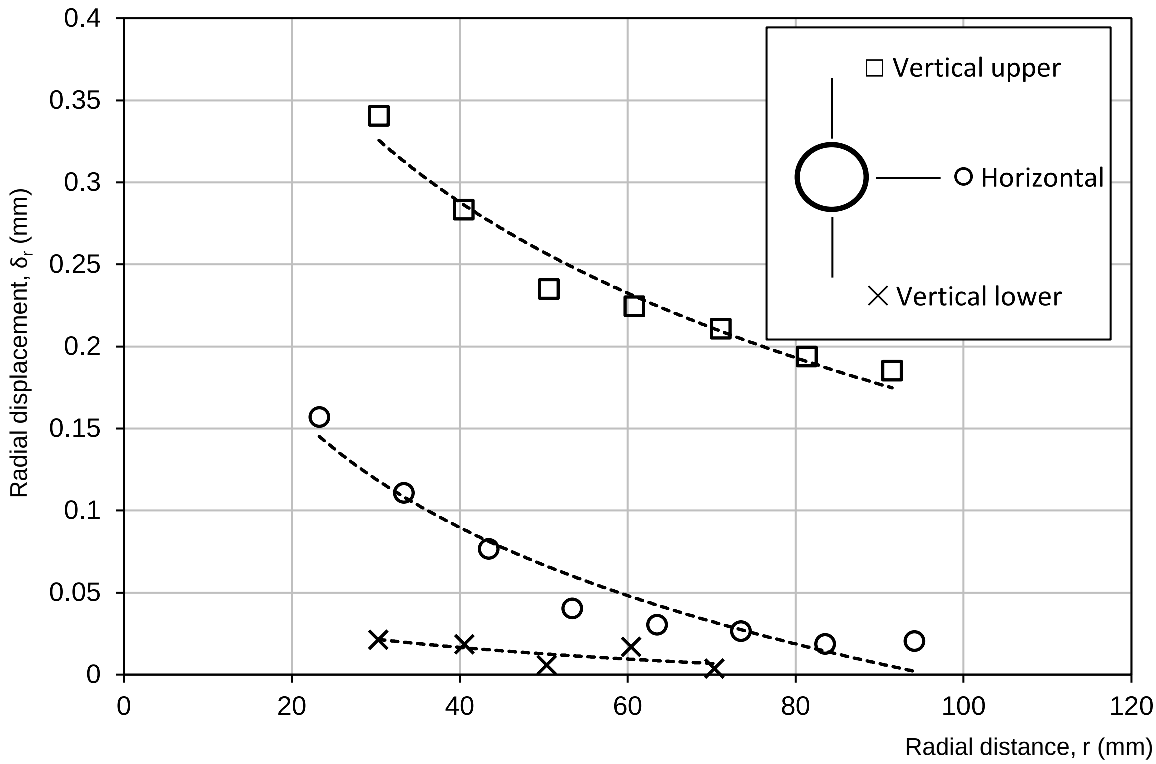

Figure 6: Movements towards the tunnel cavity generated during simulated excavation

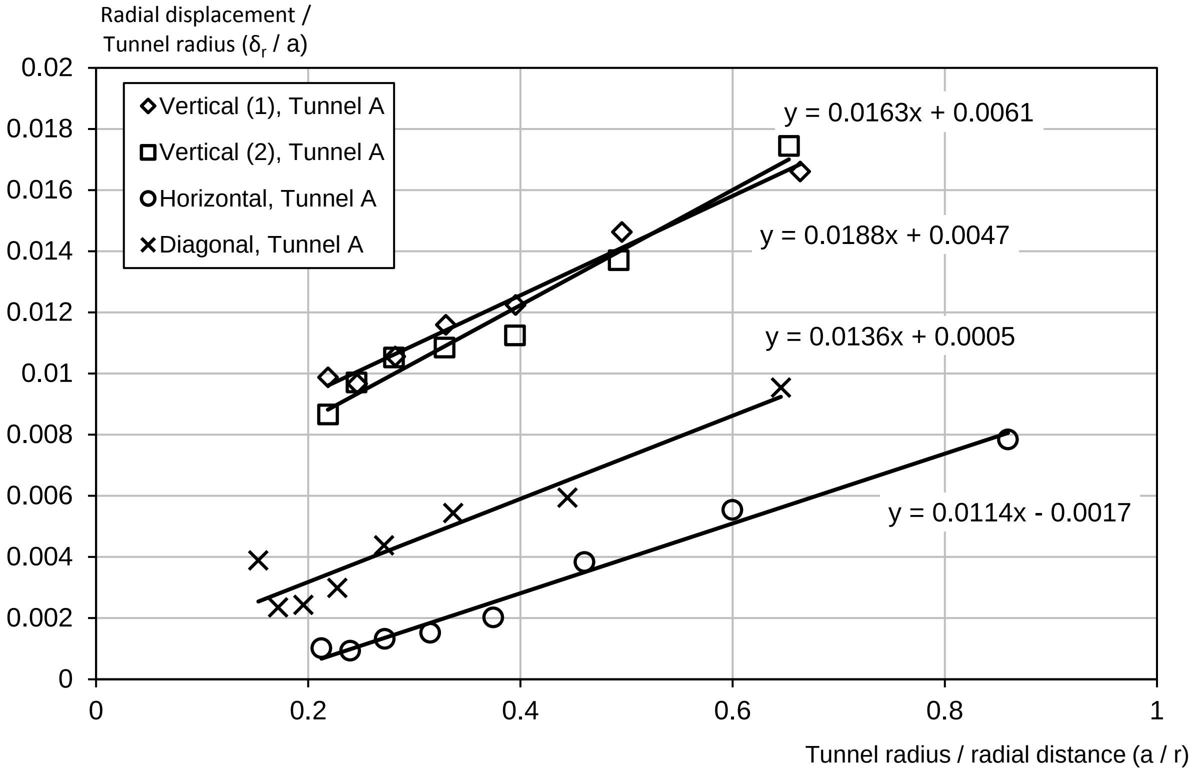

Figure 7: Vertical, horizontal and diagonal subsurface movements in the vicinity of the first simulated tunnel construction in test SD14

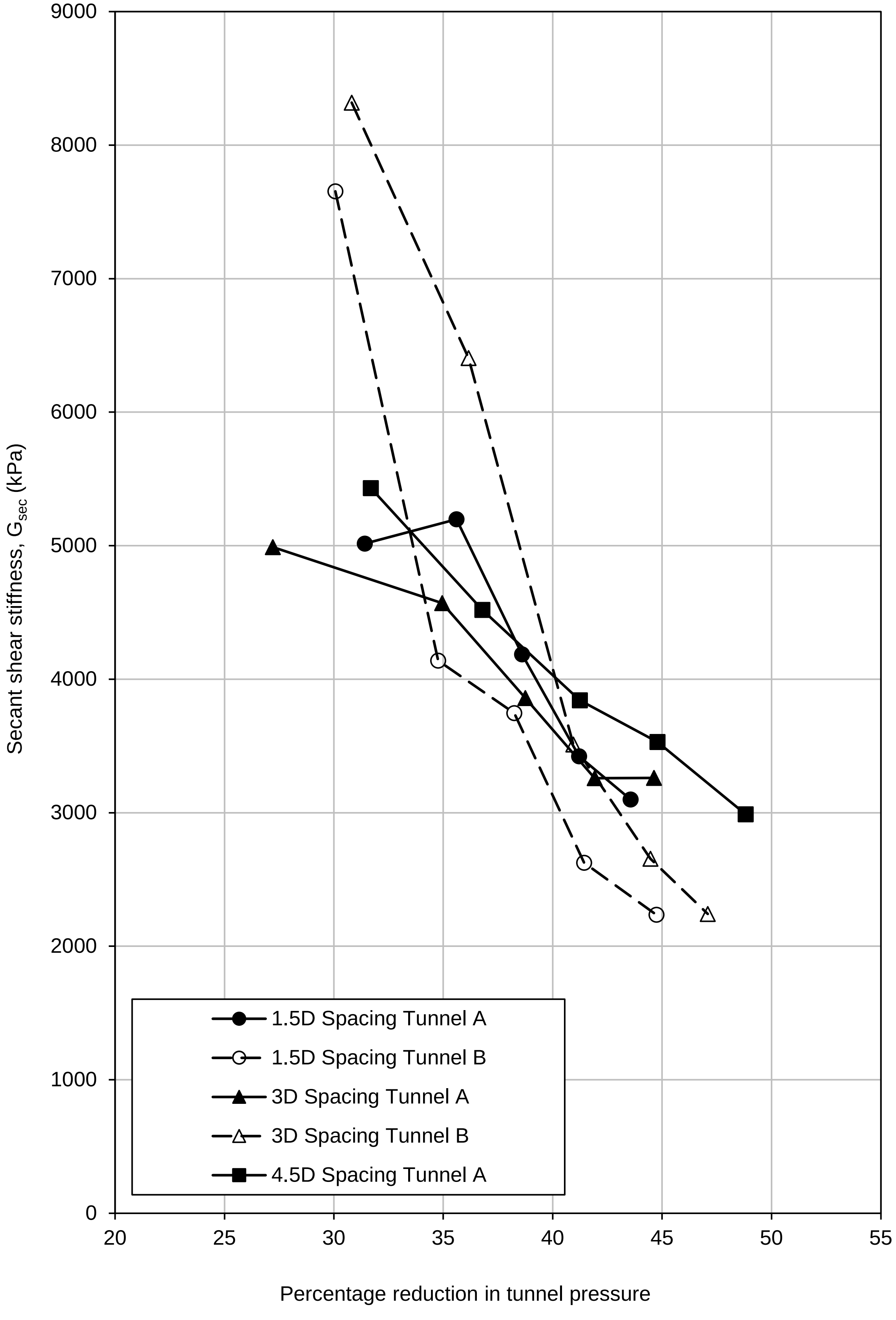

Figure 8: Variation of G during tunnel excavation events

TABLES

Test ID	Imposed V_L (%)	Cover (C)	Spacing (d)
SD10	3	2D	1.5D
SD11	3	2D	3.0D
SD12	3	2D	4.5D
SD13	5	2D	1.5D
SD14	5	2D	3.0D
SD15	5	2D	4.5D

Table 1: Summary of centrifuge tests

			Tunnel A			Tunnel B			$\Delta\sigma_T$
d	**Test ID**	**Su**	**St σ_T**	**Fin σ_T**	**$\Delta\sigma_{TA}$**	**St σ_T**	**Fin σ_T**	**$\Delta\sigma_{TB}$**	
1.5D	SD10	44.6	172.1	111.9	60.2	160.9	95.1	65.8	5.6
3D	SD11	43.7	139.0	73.3	65.7	159.8	89.2	70.5	4.8
4.5D	SD12	41.1	139.9	105.3	34.6	132.5	87.4	45.1	10.4
1.5D	SD13	45.3	183.7	103.7	80.0	178.5	98.7	79.9	-0.2
3D	SD14	42.6	181.3	100.4	80.9	178.8	94.6	84.2	3.3
4.5D	SD15	40.1	149.0	76.3	72.7	NR*	NR*	NR*	NA

Table 2: Summary of Tunnel Support pressure changes (* readings not recorded due to equipment malfunction)