



Twin tunnelling induced ground settlements: A review

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ABSTRACT

In the past few decades, the number of tunnels constructed next to an existing tunnel has been gradually increasing in order to accommodate infrastructure needs in congested urban cities. When a new tunnel is constructed adjacent to an existing tunnel, both the relative position of the tunnels and the construction sequence affect the ground settlement and internal forces in the linings of both tunnels. Therefore, it is important to study the influence of these factors and their relationship to tunnel construction. A review of twin tunnelling induced ground settlement is presented in this paper. A wide range of data is collected, summarized, and compared with each other to infer interaction phenomenon related to ground settlement. This data is gathered from published field observations, laboratory tests, and finite element analyses. The paper begins with an overview of single tunnelling induced settlements, volume losses, and factors which can affect twin tunnelling induced ground settlements. Next, a summary of the effects of construction sequence, pillar width, and cover depth, among other influencing factors, has been presented for four twin tunnelling configurations including (i) side-by-side, (ii) piggyback, (iii) perpendicularly crossing, and (iv) offset arrangement twin tunnelling. The paper also presents a summary of available techniques to calculate ground settlements induced by a new tunnel excavation in the presence of an existing tunnel. Finally, the paper summarizes available knowledge on ground settlement induced by various twin tunneling arrangements and identifies known unknowns.

Nomenclature

Over 200 studies are cited in this paper, with many using inconsistent nomenclature, it is therefore necessary to define the nomenclature and terminology employed in this paper, as follows:

A	Tunnel cross-sectional area
B	The multiple of the trough width parameter in a half settlement trough
C	Distance from Surface to top of the tunnel (Cover depth) (Fig. 1)
d	The distance between tunnels centers (Fig. 1)
D	Diameter of the tunnel
E	Soil Young's modulus
F	Modification factor to estimate modified second tunnel induced settlement
G	Shear modulus of soil
i	Trough width of a Gaussian settlement profile, i.e. the distance from the tunnel centerline to the inflection point of the trough (Fig. 2)
i_0	Trough width parameter at surface.
K	Trough width parameter, empirically determined based on soil type (Table 1)
M	Relative increase in settlement
P	Spacing between two tunnels (Pillar width) (Fig. 1)
q	Distance above tunnel crown
Q	

(continued)

	Angular spacing between two parallel tunnels located at different elevations (Fig. 1)
R	Radius of the tunnel
S	Settlement at a point
$S(x)$	Settlement at surface at a given horizontal distance (x) from the tunnel centerline
S_{max}	Maximum settlement of tunnel i (Fig. 2)
(i)	
V	Volume of surface settlement trough (Volume between the settlement trough and the original ground surface)
V_l	Volume of surface settlement trough per unit length
V_g	Greenfield tunnel ground loss
V_s	Volume loss as a percentage of tunnel face volume per unit length
x	Distance from the centerline of a tunnel to the settlement measurement point (Fig. 2)
z	Depth of sub-surface level from ground level (Fig. 7)
z^*	Distance from sub-surface level to center of the tunnel (Fig. 7)
Z_0	Distance from surface to center of the tunnel (Fig. 7)
θ	Angular relative position (Fig. 1)
ϵ	Radial shrinkage strain
γ	Unit weight of the soil
ν	Soil Poisson's ratio

(continued on next column)

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1. Introduction

Tunnelling induced ground movements are radial displacements towards the tunnel cavity and longitudinal displacements towards the advancing tunnel heading. This phenomenon has been described by the term “volume loss” or “ground loss” (Peck, 1969). In the undrained case the volume of ground loss around a tunnel cavity should manifest itself as equal to the volume of the surface settlement trough. However, many soils offer some drainage, especially urban fills, sands, and unsaturated clays and silts; where the observed settlement trough may have a different volume than the ground loss due to volume change within the soil. In addition to ground loss and volume change within the soil, tunnelling induced ground movement may also be caused by consolidation of soft clay and this is correlated with sublayer subsidence (Wu et al., 2017).

There is a strong correlation between ground loss and risk to nearby structures and utilities, because the larger the loss, the greater the nearby structures are impacted. Sublayer subsidence can put subsurface infrastructure at risk. Lyu et al. (2020) presented an improved trapezoidal fuzzy analytic hierarchy process (FAHP) to assess the risk to mega-city infrastructure related to ground subsidence. Mair et al. (1996) compared the deflection of buildings resulting from tunnelling induced settlements and concluded that both the magnitude of the settlements and the extent and shape of the settlement trough need to be considered by practicing engineers. It is therefore necessary to predict ground settlements arising from tunnelling as well as possible interaction effects during the design stage of a tunnel. A number of charts that can be used to assess the potential damage of a tunnelling project, and the strains transferred to buildings through differential settlements, at the design stage are available (Boscardin and Cording, 1989; Mroueh and Shahrou, 2003; Franzius et al., 2006; Cording et al., 2008; Devriendt, 2010; Giardina et al., 2015).

Effective use of underground space in crowded urban areas often imposes the construction of new shallow tunnels close to existing ones, in soft ground. New tunnel construction may cause large differential settlement and cracks in the lining of existing tunnels. Therefore, knowledge of anticipated ground movements will aid new tunnel construction to be carried out without damage either to the buildings above the excavation or to existing tunnels.

Ground movements due to twin tunnelling, both horizontal and vertical, have been reported by many authors for different tunnelling situations. These studies have shown that the ground movements above tunnels are influenced by many factors including tunnel diameter, tunnel depth, construction method, soil type, and volume loss. Additionally, the effects of soil removal inside a tunnel (i.e., the effects of weight loss) also influence the shape and magnitude of ground surface settlement (Verruijt and Booker, 1996; Verruijt and Strack, 2008).

This paper discusses ground settlements induced by excavation of twin tunnels. Four types of twin tunnels, based on relative position (Fig. 1) are considered, including (i) Side-by-side, (ii) Piggyback, (iii) Perpendicularly crossing, and (iv) Offset arrangement twin tunnels. The maximum ground settlement, trough width and position of the maximum ground settlement reported in a variety of field, experimental, and numerical studies have been summarized to aid designers with planning of new tunnel construction projects and for continuing study of twin tunnelling.

2. Settlements and volume losses due to tunnelling

2.1. Settlements due to single tunnelling

In order to investigate the effects of twin tunneling, it is desirable to first identify the factors that contribute to the settlement of single tunnels. Cording and Hansmire (1975) and Mair and Taylor (1997) summarized the main sources of settlements induced by shield tunneling to include: (i) Deformation of the ground towards the face, (ii) Radial

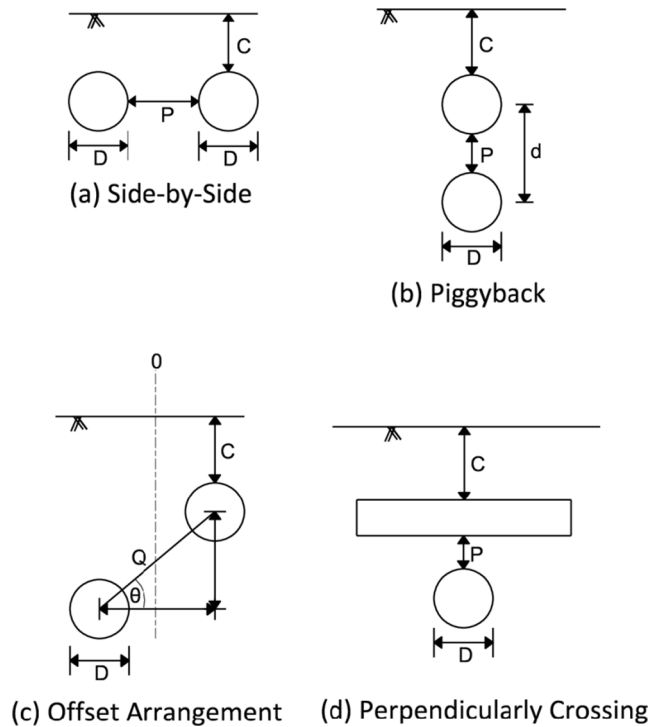


Fig. 1. Twin tunnels geometric arrangements.

ground movement towards the shield, (iii) Radial ground movement into the tail void, (iv) Deflection of the lining and (v) Consolidation. Methods of calculating each component of the ground loss have been reported by Attewell and Boden (1971), Attewell and Farmer (1974a,b, 1975), Cording et al. (1978) and Attewell et al. (1978, 1986).

Martos (1958) first observed that the shape of the surface settlement trough above mining excavations can be represented by a Gaussian curve. Later, Peck (1969) and Schmidt (1969) investigated surface settlement data from a large number of tunnels and proposed the Gaussian distribution curve shown in Fig. 2 to describe the ground settlement profile. The Gaussian shape was later verified by Fujita (1989) and New and O'Reilly (1991), among others. The shape of the settlement trough can be described using the following equation:

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

where all terms are defined in nomenclature. The trough width (i) for surface and sub-surface settlement can be calculated by methods proposed by O'Reilly and New (1982) and Mair et al. (1993) as follows:

Trough width for surface settlement,

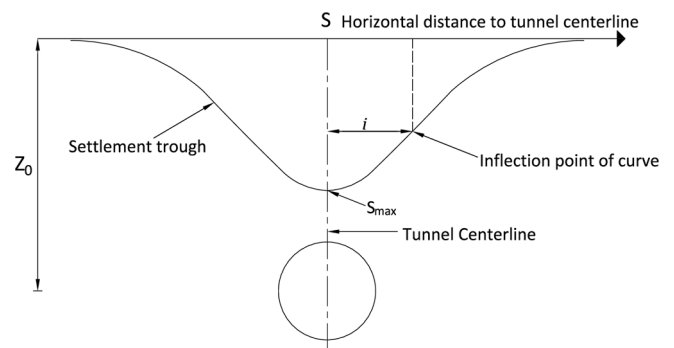


Fig. 2. Gaussian distribution curve representing tunnel settlement profile.

$$i = K \cdot z_0 \quad (2a)$$

Trough width for sub-surface settlement,

$$i = K(z_0 - z) \quad (2b)$$

K values for different soils are summarized in Table 1. These K values are widely used as a benchmark and many authors found them to be representative of values they encountered in their studies. Variations of i with depth (Z_0) have also been reported by many authors and are summarized in Table 2.

In addition to Peck's Gaussian method (Eq. (1)) several other empirical, semi-empirical and analytical methods are available to estimate ground surface movements due to underground excavation activities. They are commonly not in use due to the simplicity of Peck method's where one single parameter is sufficient to calculate settlement (i.e. only trough width parameter, i , is required, which is easy to determine using Table 2).

2.2. Ground losses induced by single tunnel

In tunnelling, ground loss or volume loss indicates the magnitude of settlement that occurs due to tunnel excavation. Generally, ground loss is defined in terms of the volume of the surface settlement trough per unit length of tunnel, normalized by the volume of the tunnel.

Volume loss is strongly influenced by the excavation technique, tunnel diameter, tunnel depth and soil conditions (Erdem and Solak, 2005). For shield tunnelling, Attewell (1977) divides the sources of volume loss into four categories: Face loss, Shield loss, Ground loss during and after lining erection, and Ground loss after grouting.

Many researchers reported volume loss for various types of soils and tunnelling techniques; a wide array of reported volume loss data is summarized in Table 3. The increased use of pressurized face Tunnel Boring Machine (TBMs) to control the excavation face combined with shield annulus bentonite injection and two-part grouting around the segments from the tail shield has reduced volume loss on tunnel projects from a few percent (of the excavated volume), a decade ago, to less than 0.5% on more recent projects (Mooney et al., 2014). For example, Shen et al. (2016) demonstrated the benefit of using large volumes of well-controlled thixotropic bentonite slurry with EPB microtunnel boring machines (MTBM) to reduce settlement.

Cording (1991) demonstrates sources of volume loss for a shield driven tunnel. Although some analytical solutions for estimating ground loss have been proposed (Chi et al., 2001; Park, 2005), the calculation still heavily relies on empirical factors and past experience. In any case, if the shape of surface settlement is known and the settlement occurs with no change in the volume of the soil, then the volume of the soil (V) between the settlement trough and the original ground surface is obtained by the integration of Eq. (1) as follows:

$$V_l = \int_{-\infty}^{\infty} S_{max} \exp\left(-\frac{x^2}{2i^2}\right) dx = \sqrt{2\pi} i S_{max} \quad (3)$$

where, V_l is the volume of the surface settlement trough per unit length of tunnel, i is trough width parameter and S_{max} is the settlement of the point directly above the tunnel.

The relationship between trough width and volume loss is complex. Grant and Taylor (2000) measured in their centrifuge tests in clay that

Table 1
Trough Width Parameter, K values for various soils (O'Reilly and New, 1982).

Soil Type	Trough Width Parameter, K
Granular soils above Ground Water	0.2–0.3
Granular soils below Ground Water	0.4–0.5
Stiff clay	0.4–0.5
Glacial deposits (NC Clay)	0.5–0.6
Soft clay/Silty clay	0.6–0.7

Table 2

Suggested value of trough width parameter “ i ” by various researchers.

References	Value of i	Comment
Peck (1969)	$\frac{i}{R} = \left(\frac{Z_0}{2R}\right)^n$ $n = 0.1$ to 0.8	Based on field observations
Atkinson and Potts (1977)	$i = 0.25(Z_0 + R)$ In case of loose sand $i = 0.25(1.5Z_0 + 0.5R)$ In case of dense sand and over consolidated clay	Based on field observations
Attewell (1977)	$\frac{i}{R} = \alpha \left(\frac{Z_0}{2R}\right)^n$ $\alpha = 1$ and $n = 0.8$ to 1.0	Based on field observations of UK tunnels
Glossop (1978)	$i = 0.5Z_0$	Based on field observations of UK tunnels
Clough and Schmidt (1981)	$i = R \left(\frac{Z_0}{D}\right)^{0.8}$	Based on field observations of USA tunnels
O'Reilly and New (1982)	$i = 0.43Z_0 + 1.1$ In case of cohesive soil $i = 0.28Z_0 - 0.1$ In case of granular soil	Based on field observations of UK tunnels
Selby (1988)	$i = 0.43Z_2 + 1.1 + 0.28Z_1$ In case of clay overlain by sand $i = 0.28Z_2 - 0.1 + 0.43Z_1$ In case of sand overlain by clay	For tunnelling in two-layer soil
Rankin (1988)	$i = kZ_0$ ($k = 0.5$ for clay)	Field observations
Arioglu (1992)	$i = 0.386Z_0 + 2.84$	
Mair (1993)	$i = 0.5Z_0$	Based on field observations worldwide
Moh et al. (1996)	$i(z) = \left(\frac{D}{z}\right) \left(\frac{Z_0}{D}\right)^{0.8} \left(\frac{Z_0 - z}{Z_0}\right)^m$ With $m = 0.4$ for silty sand and $m = 0.8$ for silty clay.	Based on field observations of Taipei tunnels
Heath and West (1996)	$\frac{i}{i_0} = \left(\frac{z}{Z_0}\right)^{1/2}$	Based on field observations of UK tunnels
Loganathan and Poulos (1998)	$\frac{i}{R} = 1.15 \left(\frac{H}{2R}\right)^{0.9}$	
Hamza et al. (1999)	$i = 0.43Z_0 + 1.1$	Based on field observations of Cairo metro
Tan and Ranjith (2003)	$i_0 = (0.57 + 0.45Z_0) \pm 1.01m$ for sites where consolidation effects are insignificant	
Bilotta and Russo (2012)	$i = bD \left(\frac{Z_0 - z}{D}\right)^m$ With $b = 0.8$ & $m = 0.2$.	Based on field observations of Naples metro

the settlement trough had a constant width between volume losses of 2 and 20%. However, Hergarden et al. (1996) showed in centrifuge tests of mixed soil types (sand overlying clay) that an increase in volume loss corresponded to a decreased parameter i .

The volume loss for a circular tunnel is expressed by Eq. (4), as follows:

$$V_l = V_s \left(\frac{\pi D^2}{4}\right) \quad (4)$$

where, D is the tunnel diameter and V_s is the volume loss as a percentage of tunnel face volume per unit length of tunnel. Rearranging Eq. (3) and Eq. (4), if the percentage of volume loss is known the maximum settlement for a circular tunnel can be estimated as (Eq. (5)):

$$S_{max} = 0.313 V_s \frac{D^2}{i} \quad (5)$$

Thus, if the trough width parameter is known, or assumed, the maximum anticipated settlement can be computed for a tunnel of known diameter.

Table 3Reported volume loss, V_s .

References	Soil	Volume Loss, V_s	Type of Tunnelling
Eden and Bozozuk (1969)	Sensitive Leda Clay	1.5%	NATM
Sauer and Lama (1973)	Frankfurt clay	1.8%	
Attewell and Farmer (1974a)	London clay	1.44%	
Cording and Hansmire (1975)	Clay	3.0%	
O'Reilly and New (1982)	London clay	1.0–1.4%	Open faced shield driven tunnels
Rowe and Kack (1983)	Stiff clay	1.3%	Green Park Tunnel
Temporal and Lawrence (1985)	Oxford clay	0.5	EPBM closed face tunnelling
Broms and Shirlaw (1989)	Soft clay	<1.0%	
Harris et al. (1994)	London clay	1.1%	Heathrow Trial Tunnels
New and Bowers (1994)	London clay	1.0–1.3%	
Atahan et al. (1996)	Sand/Gravel	1.5%	SS
Moh et al. (1996)	Silty sand	1.3%	EPB
Linney and Friedman (1996)	Dense sand/stiff clay	1.0%	EPB
Simic and Gittoes (1996)	Sand/Soft clay	0.8–1.2%	EPB
Bowers et al. (1996)	London clay	1.1–1.5%	NATM
Kavvadas et al. (1996)	Weak rocks	0.2%	NATM
Umney and Heath (1996)	London clay	1.5–1.8%	Shield + Segments
Standing et al. (1996)	London clay	2.9–3.3%	Shield + Segments
Addenbrooke and Potts (1996)	London clay	1.4%	Open faced method Barcelona Subway Extension Tunnel Open faced method NATM Closed faced TBM Closed faced TBM Full face TBM
Barakat (1996)	London clay	0.7–1.6%	
Ledesma and Romero (1997)	Clay with some gravel	1.2%	
Mair and Taylor (1997)	Stiff Clay	1.0–2.0%	
	Stiff Clay	0.5–1.5%	
	Sand	0.5%	
	Soft Clay	1.0–2.0%	
Macklin and Field (1998)	London Clay	2.4%	
Nyren (1998)	Very Stiff London Clay	1.1–1.8%	Jubilee Line Extension, NATM
Sugiyama et al. (1999)	Stiff Clay	<1.0%	Slurry Shield Method
Loganathan et al. (2000)	Sand	0.2–0.6%	TBM Sydney
Cooper et al. (2002b)	Clay	1.3–2.5%	Piccadilly Line Tunnels
Wu and Lee (2003)	Clayey Soils	1–2%	Taipei MRT Projects
Coulter and Martin (2006)	Glacial moraine	0.35%	Excavated using a Jet Grout Arch
Hunt (2005)	London Clay	1.2%	EPBM Singapore EPBM Tokyo
Tjie-Liong (2005)	Soft Marine Clay	Up to 3%	
	Gravels below water level	0.2%	Shield-machine Bored Tunnels Singapore Circle Line Tunnels
Hsiung (2011)	Sand	0.38–0.53%	
Zhang et al. (2011)	Old Alluvium	0.1–0.9%	EPBM London
Wan et al. (2017)	London clay	0.8%	

2.3. Sub-surface settlement due to single tunnelling

It is now increasingly common to build tunnels under existing foundations, pipelines, and tunnels. Hence, predicting the sub-surface settlements is now as important as the surface movements described in earlier sections. Potts (1976) reported on early case histories of sub-surface movements of single tunnels. These studies observed that the maximum sub-surface settlement was greater than the maximum surface

settlement, and that the width of the subsurface settlement trough was narrower. Consequently, sub-surface utilities above the tunnel would likely experience a larger angular distortion than surface facilities. These results were recently confirmed in experimental studies that employed transparent soils to measure sub-surface strains and relate them to support pressure (Ads et al., 2020; Ahmed and Iskander, 2010, 2012).

Sub-surface ground movements have been measured by monitoring the movement of markers placed into the soil and using photographs taken during the tunnelling operation (Mair, 1979; Taylor, 1984; Grant, 1998). O'Reilly and New (1982) suggested that the sub-surface settlement trough due to tunneling can be described by the normal probability function. Similarly, Mair et al. (1993) studied the location of the inflection point, and the maximum subsidence of the sub-surface settlement trough in centrifuge test. It was concluded that both the surface and sub-surface settlement troughs could be approximated by the normal probability curve. Alternatively, Park (2004) used elastic solutions to estimate the tunneling-induced ground deformations in soft ground. Surface and sub-surface settlements from five case studies were compared with the proposed analytical solutions, and good agreement of the predicted and monitored ground deformations were seen for tunnels in uniform soft clay. Maximum sub-surface ground movements from five different field observations at various sub-surface elevation level are plotted in Fig. 3. It can be observed that the cover to diameter ratio (C/D) played an important role in explaining the observed sub-surface settlements; the higher the C/D ratio, the lower the respective subsurface settlement is. In addition, low cover depth along with low C/D ratio further increase the settlement.

2.4. Settlements due to twin tunnelling

Terzaghi (1942) published the first paper presenting twin tunnel settlement field data. Since then, field data remains the key to understanding the interaction between adjacent tunnels and is often used to validate data obtained from numerical analysis and experimental studies. Deere et al. (1969) summarized the available field data prior to 1969. Unfortunately, however, field data is often incomplete to describe phenomena, making numerical and experimental studies necessary.

In the past all tunnelling operations were done with compressed air and when twin tunnelling is encountered, it has been found that the settlements due to the second tunnel construction are larger than those observed above the first tunnel (Moretto, 1969; Bartlett and Bubbers, 1970; Barla and Ottoviani, 1974). The observed larger volume loss causes a predictable increase in the magnitude of the displacements obtained (Hanya, 1977; Brahma and Ku, 1982; Hunt, 2005). But in recent years, due to changes in TBM technology volume loss is reduced and the settlement trough caused by the excavation of the new tunnel is shallower and wider than the one caused by the existing tunnel (Do et al., 2014a).

Some case studies have shown that surface settlement troughs caused by twin tunnels have a variety of shapes (Cooper et al., 2002b). Perez Saiz et al. (1981), Ottaviano and Pelli (1983), Cooper and Chapman (1998) and Fagnoli et al. (2015) all reported asymmetry of settlement trough which happened because of additional movements caused by the interaction between tunnels. The position of the maximum settlement is typically shifted towards the first tunnel driven (Lo et al., 1987).

Twin tunnelling induced ground settlement predictions initially developed by superposition of the settlement curves of two single tunnels. For example, New and O'Reilly (1991) proposed that settlement be obtained as the sum of identical Gaussian curves, disregarding any interaction effects. Numerical and experimental studies including Ng et al. (2004) indicate that superposition may not necessarily be accurate. Fang et al. (1994) states that superposition could be used to estimate settlements above parallel tunnel construction if the interaction is negligible. Addenbrooke and Potts (2001), Chapman et al. (2003), Hunt (2005) and Divall and Goodey (2015) presented a variety of modification techniques to calculate tunneling-induced ground movements

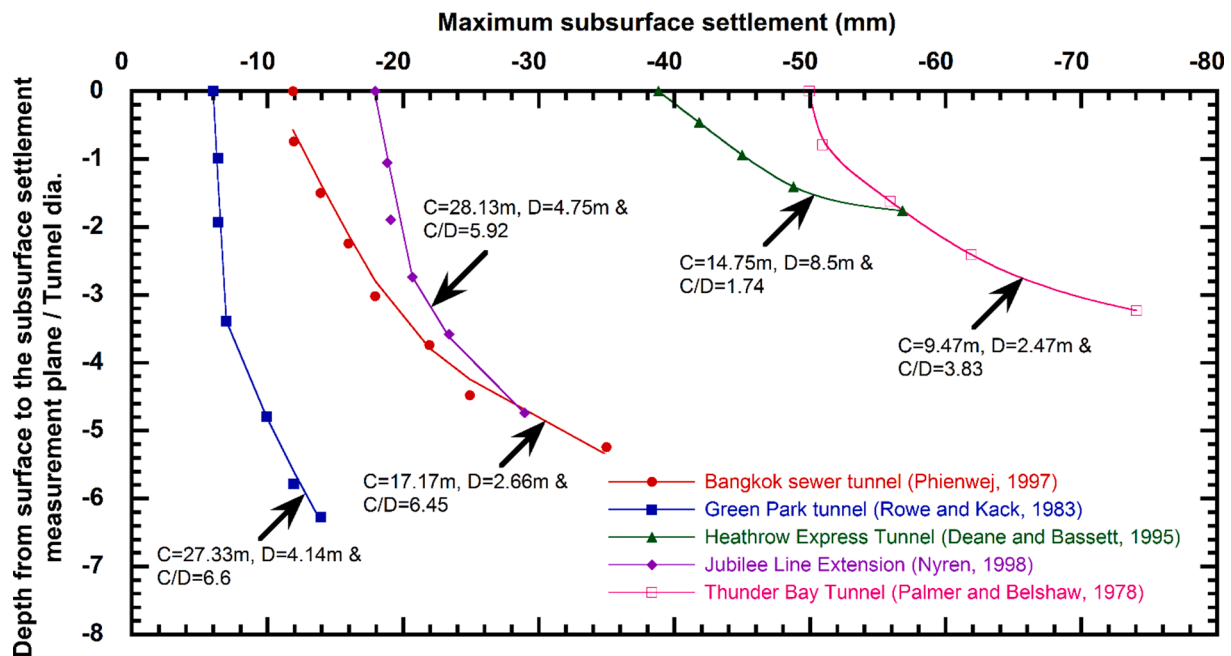


Fig. 3. Subsurface settlement at various subsurface elevation level (Based on data from Phienwej, 1997; Rowe and Kack, 1983; Deane and Bassett, 1995; Nyren, 1998; Palmer and Belshaw, 1978).

caused by the new second tunnel excavation, taking interaction effects in account, which are described later in this paper.

2.5. Factors affecting twin tunnelling induced settlements

The factors causing settlement of twin tunnels can be grouped into three major categories encompassing (i) tunnel geometry, (ii) geological conditions, and (iii) shield operation factors (Finno and Clough, 1985; Clough and Leca, 1993; Matsushita et al., 1995).

- i. **Tunnel Geometry:** Most of the reported twin tunnelling studies consider the effect of the tunnel size, depth, tunnel spacing, relative position between two tunnels, and construction sequence on the induced settlements (Hefny et al., 2004; Karakus et al., 2007; Song et al., 2008; Afifipour et al., 2011; Chakeri et al., 2011; Mirhabibi and Soroush, 2012; Shahin et al., 2013 among others). Their results are consistent in that the influence of the second tunnel on the first tunnel has been shown to depend on the relative position of the tunnel and/or the spacing between the two tunnels. Sterpi and Cividini (2004) demonstrated that the depth of both tunnels and the width of the central pillar play a major role in defining the collapse load and the shape of the failure mechanism. Similarly, in a study by Choi and Lee (2010), it was found that the displacements decreased and stabilized as the distance between the tunnel's centers increased, depending on the size of the existing tunnel.
- ii. **Geologic Conditions:** The changes in the settlement profile above a second tunnel, in close proximity to the first tunnel, are influenced by a variety of factors such as changes in volume loss, changes in trough width and the effect of pre-failure soil stiffness. These factors are dependent on ground conditions *i.e.* type of soil (cohesive or cohesionless), presence of a single layer or multi-layered soil and the prevalence of drained or undrained soil conditions, among others.
- iii. **Shield Operation Factors:** Suwansawat and Einstein (2007) and Ocak (2013) found that operational parameters, such as face pressure, penetration rate, grouting pressure and filling, have significant effects on the maximum settlement and extent of the settlement trough. In addition, in double-O-tube (DOT)

tunnelling method, three different moving trajectories: pitching, yawing, and rolling result in over-excavation compared to single circular shield tunnelling. Ren et al. (2018) evaluated the gap area between the DOT shield machine and the linings and proposed a modified equation to compute ground loss ratio.

Shield operation factors are project specific and are difficult to generalize. Similarly, at this time, it is not possible to adequately resolve the effect of many geologic factors on tunneling operations due to the paucity of field data. In particular, the effect of volume loss on ground movements is not directly covered in this paper due to the absence of well controlled studies where volume loss is kept constant to compare settlements under different geometric and geologic conditions. Numerical modeling simulations are typically used to address this deficiency. Therefore, this review focuses on the effect of geometric factors for four tunnel configurations, which are addressed in the following sections.

3. Settlements from side-by-side (horizontal alignment) twin tunneling

Terzaghi (1942), Ward and Thomas (1965) and Moretto (1969) are among the first few to investigate the effect of side-by-side parallel tunnel interaction. Later, Hanya (1977) and Akins and Abramson (1983) reported larger displacements and volume losses for the second tunnel driven. Chapman et al. (2006) also reported greater movements above the second tunnel constructed. They also observed that the maximum settlement offsets towards the first tunnel as distance above the tunnel crown increases.

Dhar et al. (1981) performed model tests to study the fracture pattern of twin circular tunnels in weak materials at different orientations and locations under controlled loading conditions. They reported that the stability of twin tunnels oriented parallel to the direction of major loading was better than for other orientations. Interaction effects become more significant as the tunnels' inter-axis distance (*aka.* Pillar distance plus tunnel radii) decreases. Results show that in some configurations, the interaction could largely affect the soil settlement and that the design of twin-tunnels requires numerical analyses to explain monitoring results during tunnel construction. In particular, cover to diameter (C/D) ratio and Pillar to Diameter (P/D) ratio are reported

where available.

3.1. Available observations of settlement due to twin tunneling

A summary of surface settlements above the second tunnel in side-by-side twin tunnel configurations is shown in Fig. 4, for a combination of field observations, model tests and numerical studies. The settlement, S , is normalized by the maximum settlement, S_{max} , and is plotted against the distance from the center line of the second tunnel being excavated normalized by tunnel diameter. A greenfield settlement profile calculated using Peck (1969) formula, which represents a no interaction situation is also plotted for reference. Note that throughout this study continuous lines are used for field observations and experimental data, while dashed lines are employed for numerical calculation. Also the data source is identified with an ID# number, where the numbers refer to Appendix A. The shape of the settlement trough above the second tunnel is like the greenfield profile. It is evident that, peak settlement trough occurs further away from the centerline of the new tunnel as the ratio of C/D increases. It is not possible to distinguish the effect of soil type because most available observations are in clay. Surprisingly, The P/D ratio did not play an important role. But, the magnitude of settlement decreases with increasing the distance between the tunnels. The lateral position of the maximum settlement is offset towards the existing tunnel. Chapman et al. (2002) found that the predicted maximum relative increase in settlement occurred above the centerline of the first tunnel driven in twin side-by-side construction and was independent of tunnel spacing, for FE analysis having $P/D = 1.22$ to 12.33 and $C/D = 2.39$.

3.2. Approaches to predict settlement due to twin tunneling

Both numerical modeling and in situ observations were used to analyze the interaction between twin-tunnels (Kawata and Ohtsuka, 1993; Soliman et al., 1993; Perri, 1994; Shahrour and Mroueh, 1997; Galli et al., 2004; Fang et al., 2015). Although numerical analysis is capable of overcoming the limitations of empirical approaches their success depends largely on proper selection of the constitutive soil parameters, correct simulation of the tunnel excavation sequence and details of the structural modelling all of which are difficult, or impractical, especially for preliminary analysis. In addition, accurate three-

dimensional numerical analyses are typically time consuming for the complex geometries and soil conditions typically encountered. Therefore, simplified two dimensional analyses are often employed to obtain results quickly. For example, Peck's equation was found to be superior in estimating the ground surface settlement induced by tunneling in comparison to FE analysis (Chen et al., 2012). They also concluded that, the larger the number of excavations (superpositions), the larger the deviation between the field and the FE results. This happened because of the FE simulation of a series of tunnel excavations seemed to repeatedly generate undesirable shear strains around the existing tunnels and caused a larger ground loss than that in the "all tunnels excavated simultaneously" analysis. This demonstrates why, empirical approaches remain widely popular.

The Superposition Method is a simplified approach for predicting surface settlements above any twin-tunnel configuration. According to this simplified method, a tunnelling induced ground settlement curve positioned over the centerline of each tunnel is obtained, ignoring any influence from the other tunnel. The summation of these two overlapping curves describes the total settlement. Fujita (1985) and Fang et al. (1994) concluded that the principle of superposition could be applied to estimate ground surface settlements for parallel twin tunnels if the ratio of the distance between tunnel centers to the diameter of the tunnels was larger than 2.7, irrespective of the ratio of the cover to the diameter of the tunnels. Similarly, Suwansawat and Einstein (2007) found that the additional settlement trough induced by the second tunnel can also be described by a Gaussian curve and the total settlement trough can be constructed by superimposing the additional curve on the settlement trough observed after the first shield passing. Finally, Ma et al. (2014) proposed a double peak Gaussian model to describe the ground settlement trough over twin tunnels; they also proposed the method for calculating ground loss over twin tunnels based on the proposed double Gaussian model.

3.2.1. Simultaneous excavation of twin tunnels

O'Reilly and New (1982) proposed a formula for evaluation of twin tunnelling induced ground settlements by superposition disregarding any interaction effects. Their approach remains one of the most popular empirical methods in use. The method sums together the settlement trough above each tunnel as shown in Eq. (7).

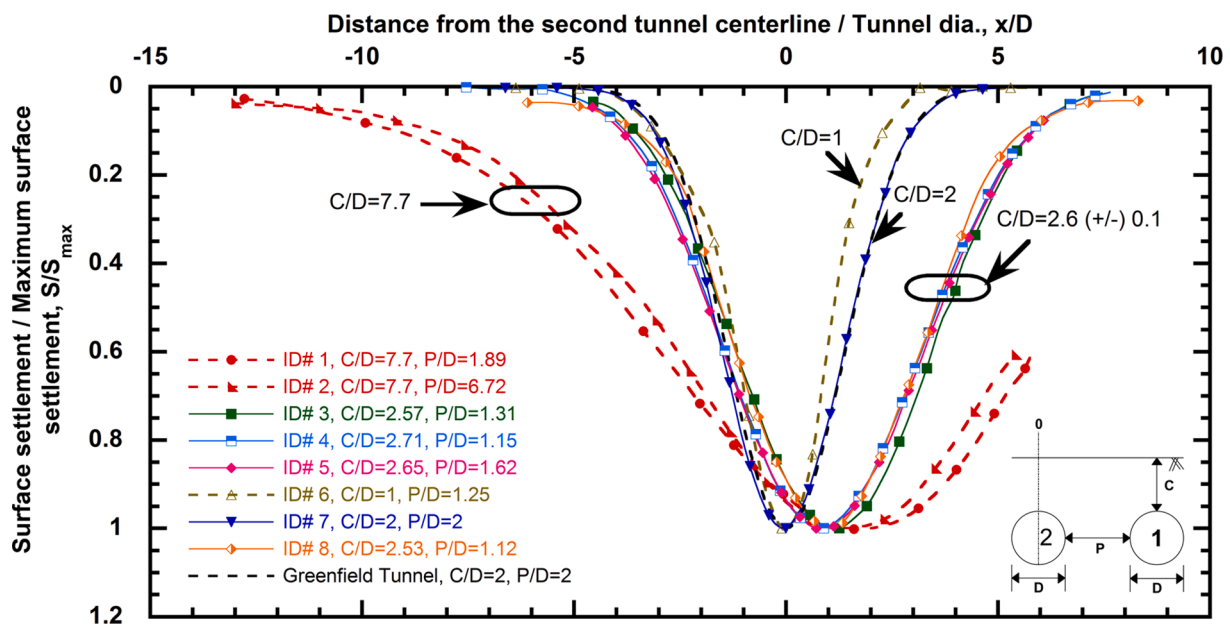


Fig. 4. Surface settlement above second tunnel of two Side-by-Side twin tunnel (Based on data from Addenbrooke and Potts, 1996; O'Reilly and New, 1982; Ocak, 2014; Do et al., 2014; Divall and Goodey, 2015; Chakeri et al., 2015).

$$S_{x,z} = S_{max} \left[\left(\exp \frac{-x^2}{2i^2} \right) + \left(\exp \frac{(x-d)^2}{2i^2} \right) \right] \quad (7)$$

where, d represents the distance between tunnels centers and x is the lateral distance from the centerline of the first bored tunnel (Fig. 5). Fig. 5 is drawn in such a way to demonstrate the eccentricity that the lateral position of peak settlement trough of a new second tunnel is offset with respect to the tunnel centerline, towards the existing tunnel.

O'Reilly and New's method have been found to give realistic predictions when twin tunnels are driven simultaneously. The method was derived for predicting surface displacements, although it can easily be extended to sub-surface regions by assuming unchanged bounds to movement. However, twin tunnels are not always driven simultaneously, and a time delay may occur between drives. This delay can lead to asymmetry, eccentricity and an increase in volume loss, and none of these can be considered in Eq. (7).

Despite its popularity, several studies indicated that the superposition formula mentioned above is not accurate to estimate ground settlement (Nyren, 1998; Ercebebi et al., 2011; Ocak, 2013; Ocak and Seker, 2013) because the superposition method cannot take into account the interaction between the first tunnel and the newly constructed second tunnel. It also does not take into account the repeated unloading of the soil (Divall and Goodey, 2015) and therefore, the predicted settlement curve does not always represent the observed field displacement.

3.2.2. Staggered excavation of twin tunnels

Staggered twin tunneling refers to when one tunnel advances ahead of a second tunnel advancing in the same direction. Settlements induced by staggered twin tunneling results from three incremental contributions (Hulme et al., 1989; Moh et al., 1996), as follows: (i) the settlement accumulated due to construction of the first tunnel up to the stage of construction corresponding to the face of the second tunnel located at the monitored section. The magnitude of this component depends on the pillar distance between the two tunnels, and to a lesser degree on the lagging distance between the two tunnels. (ii) the settlement added during the passage of the TBM from the face to the shield tail; and (iii) the residual settlement up to the steady-state value. The largest contribution is that related to the shield passage (ii), with a smaller but non-negligible fraction occurring later (iii), usually during the back-filling operation (Sugiyama et al., 1999 and Fargnoli et al., 2013).

3.3. Settlement calculations for second tunnel excavation

Addenbrooke and Potts (2001) pioneered numerically derived design charts that can be employed to determine the (1) eccentricity, (2) maximum settlement and, (3) the increase in volume loss of the second tunnel's settlement profile (Fig. 6). The design chart indicates that the volume loss resulting from the second tunnel increases as the spacing between the tunnels decreases. Once the modified volume loss has been obtained the second tunnel settlement can be calculated. The modified

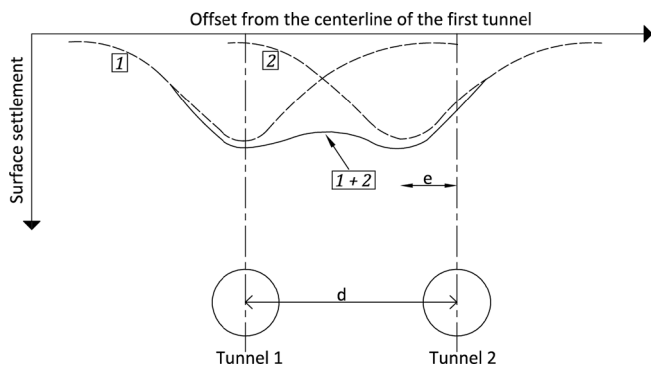


Fig. 5. Surface settlement by superposition of two side by side tunnels.

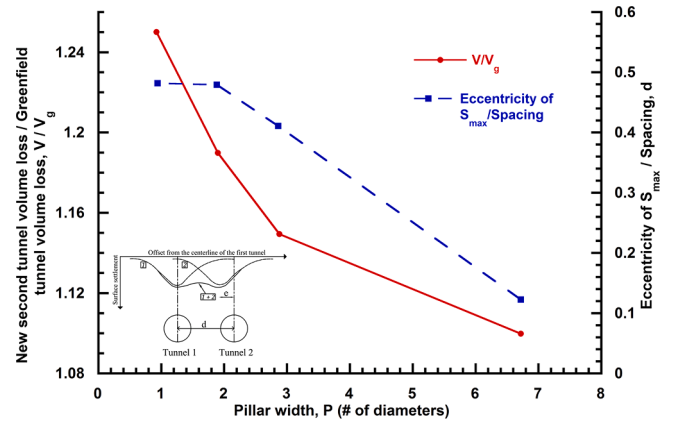


Fig. 6. Increase in volume loss (V) of the second tunnel's settlement profile in comparison to greenfield tunnel (V_g) (left) and eccentricity of the maximum settlement relative to center of new tunnel (right). (Replotted using data from Addenbrooke and Potts, 2001).

second tunnel settlement can then be summed with those of the unchanged first tunnel to predict the total settlement induced by the twin tunnel excavation. The design chart also indicates that the eccentricity (distance between new second tunnel centerline and peak settlement trough) increases as the spacing between the tunnel decreases.

Later studies (Chapman et al., 2003; Hunt, 2005; Divall and Goodey, 2015) presented a variety of modifications to calculate tunneling-induced ground movements caused by the new second tunnel excavation. The second tunnel settlements can be predicted using equations by Peck (1969), O'Reilly and New (1982) and Mair et al. (1993) along with the modifications. For example, Chapman et al. (2003), Hunt (2005) and Divall and Goodey (2015) all assume an "overlapping zone" in which the soil has been previously disturbed by the creation of the first tunnel (as shown in Fig. 7).

Chapman et al. (2003) proposed a modification factor, F , to be multiplied by the greenfield settlement for estimating the settlement profile above a second tunnel (Eq. (8)).

$$F = \left(1 + \left(M \left(1 - \frac{|d' + x|}{BK_1 Z^*} \right) \right) \right) W_{max} \exp \left(-\frac{x^2}{2(K_2 Z^*)^2} \right) \quad (8)$$

where $Z^* = (Z_0 - Z)$, B is the number of trough width parameters in a half trough width (usually taken as 3), d' is the spacing of the tunnels, K_1 is the value of K (trough width parameter) for first tunnel and K_2 is the value of K for the second tunnel. The modification factor is then applied to a greenfield settlement profile inside the "overlapping zone" of bounds to movement. The overlapping zone decreases with the increase

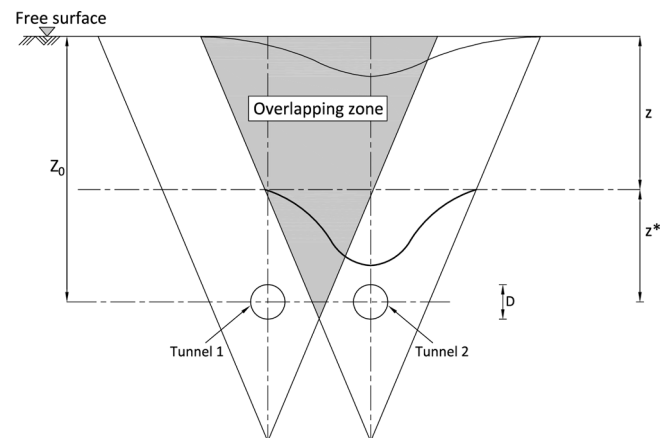


Fig. 7. Overlap of tunnel influence zones (modified from Hunt, 2005).

of depth as shown in Fig. 7. The predicted total settlement is obtained by adding the modified second tunnel settlement with the first tunnel settlement.

Similarly, Hunt (2005) also proposed a modification factor (Eqs. (9) and (10)) to compute the tunneling-induced ground movements caused by the second tunnel. Hunt's method is also based on modifying the ground movements of the second tunnel in an "overlapping zone".

$$S_{mod} = FS_v \quad (9)$$

where,

$$F = \left\{ 1 + \left[M \left(1 - \frac{|d + x_A|}{BK_A Z^*} \right) \right] \right\} \quad (10)$$

where, S_{mod} = the modified settlement, S_v = the unmodified settlement above the second tunnel calculated by semi-empirical methods, $Z^* = (Z_0 - Z)$, B = the multiple of the trough width parameter (usually taken as 2.5 or 3) in a half settlement trough, d = the center-to-center spacing of the tunnels, K_A = the value of K in the region of the first bored tunnel and M = relative increase in settlement (typically 0.6, described in Chapman et al. (2006)). The maximum relative increase in settlement, $M = 1.0$, is aligned with the centerline of the existing tunnel and reduces to zero at some lateral distance from it. Hunt (2005) concluded that the maximum percentage increase in settlement was usually between 60 and 80%.

Divall and Goodey (2015) employed centrifuge test results to develop Fig. 8 and Fig. 9, which can be used to predict the magnitude of additional volume loss and asymmetry that may be expected. The increase in volume loss, for all observed depths, against center-to-center spacing in terms of tunnel diameter is shown in Fig. 8. As the separation between the tunnels increase, the effect on the additional volume loss reduces. The values of the Trough Width Parameter, K , are shown in Fig. 9. K values on the side of the settlement trough near the first tunnel are systematically higher for lower values of center-to-center spacing. At spacings above 3D the settlement trough produced by construction of a new second tunnel was symmetrical. These values could then be inserted into the relationships described by Peck (1969) or 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 (proposed by O'Reilly and New, 1982) to give the total twin-tunnel settlement.

3.4. Factors affecting side by side tunnelling induced ground settlements

3.4.1. Effects of cover depth

For twin tunnels, when a new tunnel is excavated in the presence of an existing one, limited available data suggests that the larger the cover depth, the wider the settlement trough is and the more the offset/shift of the maximum settlement towards the existing tunnel (Fig. 4). Wang et al. (2003) also reported that, the shallower the tunnel is, the stronger the interaction is. Also, as the C/D ratios increases, surface settlements decrease (Ocak, 2013). However, a comprehensive parametric study can help to clarify the behavior.

3.4.2. Effects of pillar width

Early numerical models designed to investigate the influence of pillar width on liner stresses in order to determine the spacing required to minimize tunnel interaction were carried out by Barla and Ottoviani (1974) as well as Ghaboussi and Ranken (1977). Barla and Ottoviani (1974) found that there was approximately a 150% increase in liner stresses for a pillar width of 0.25D when compared to those calculated for a pillar width of 1D. Ghaboussi and Ranken (1977) determined that a tunnel spacing of 1.2D would be acceptable to minimize spacing and increase in liner stresses, while a spacing of 2D was required to completely eliminate the interaction. Previous shield tunneling construction used compressed air to control ground water and to provide face support and for twin tunneling it was required to brace the first tunnel while excavating the second tunnel. Therefore, increase in liner stress was an issue. Now, modern tunnel excavation methods can mitigate this issue.

Several recent studies explored the effect of pillar width on settlement. The point of zero interaction would be defined where the measured settlements are the same irrespective of pillar width. These studies exhibit a consistent general trend, but their specific findings are somewhat divergent; they can be summarized as follows:

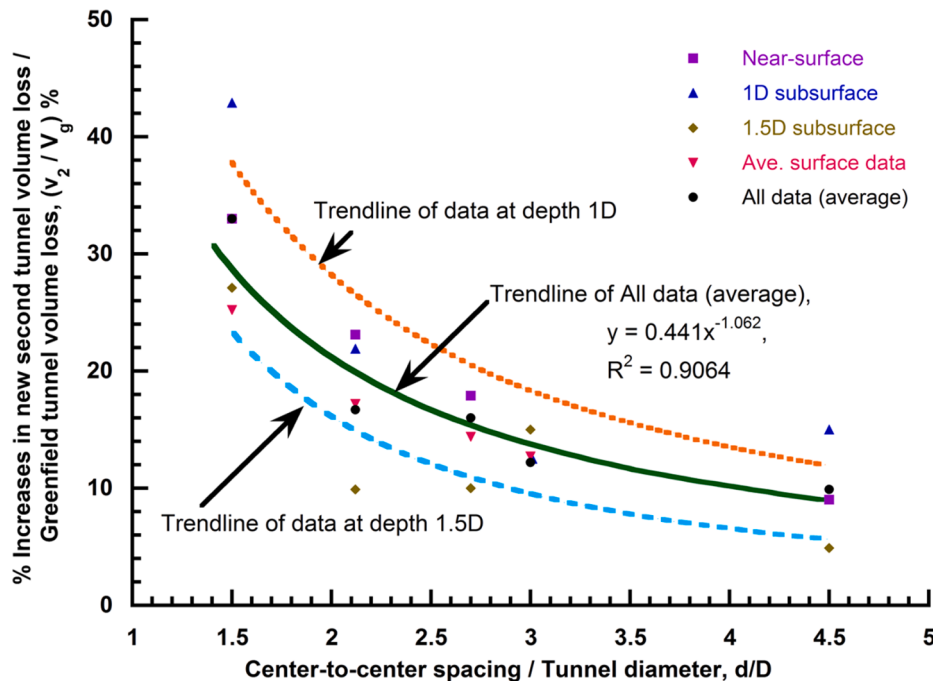


Fig. 8. Increases in new second tunnel volume loss in comparison with first tunnel plotted against the normalized distance between tunnel centers (replotted from Divall and Goodey, 2015).

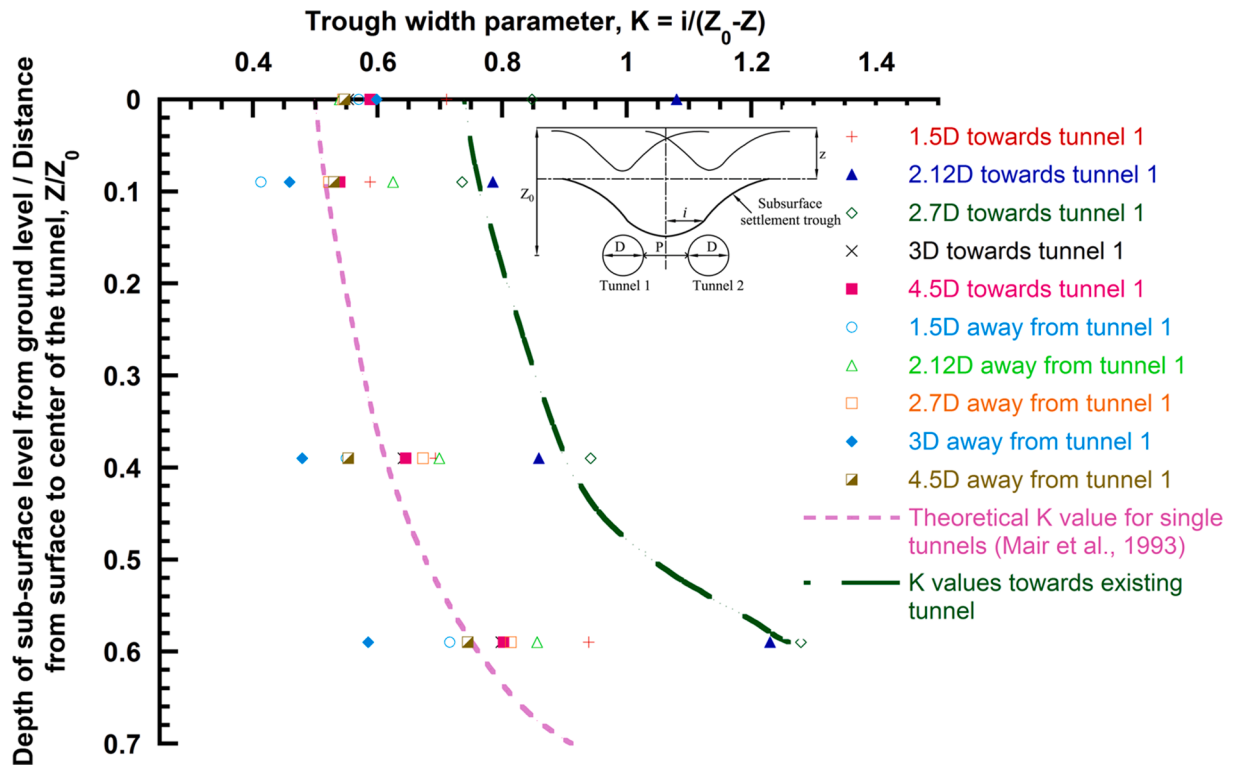


Fig. 9. Variation of trough width parameter, K with depth (replotted using data from Divall and Goodey, 2015; Mair et al., 1993).

- Interaction increased considerably at pillar widths less than 1.25D (Chen et al., 2009)
- When the distance between the tunnels is 1.5 times the tunnel diameter, the interaction was found to be small (Chakeri et al., 2011). For spacing of more than 3D, the shape of surface settlement becomes similar to the shape of two separate tunnels' Gaussian curves. Also, the interaction factor approaches zero when spacing is larger than 4D (Chakeri et al., 2015).
- For a pillar width of about twice the tunnel diameter or greater, the displacements of each of the two parallel tunnels were essentially identical to those of a corresponding single tunnel construction (Do et al., 2014a).
- Interaction effects appear to be present up to a pillar width of three to four diameters (Koungelis and Augarde, 2004; Kim et al., 1998; Wang et al., 2003; and Chehade and Shahrour, 2008).

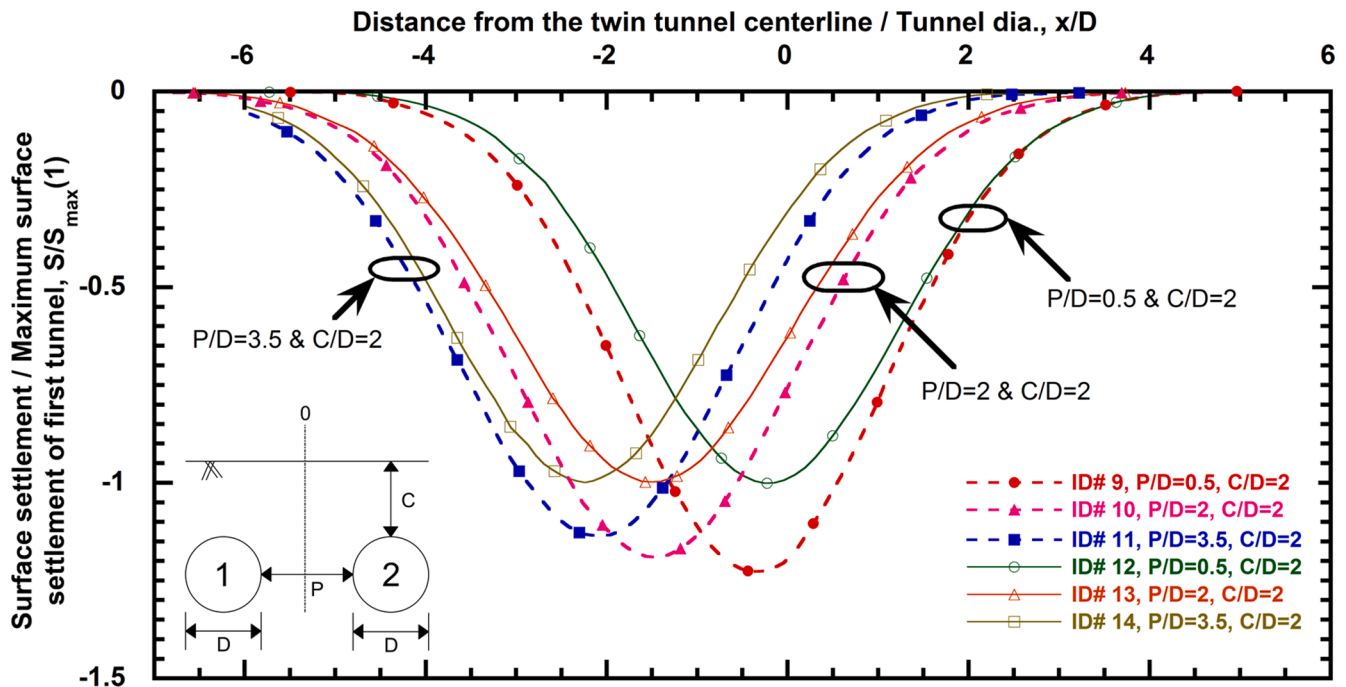


Fig. 10. Effect of pillar width on surface settlement of second tunnel during twin tunnel excavation (Based on data from Addenbrooke and Potts, 2001; Divall, 2013).

- Interaction became negligible for pillar width greater than 7D for side-by-side parallel tunnel (Addenbrooke and Potts, 2001; Cooper et al., 2002b).

The effect of Pillar width on the surface settlement due to the construction of a second tunnel in side-by-side twin tunnel configuration is explored in Fig. 10 and Fig. 11 using numerical data from Addenbrooke and Potts (2001) and experimental data from Divall et al. (2012) and Divall (2013). It is evident that when the Cover to diameter ratio is fixed, settlement increases with decreasing pillar width (Fig. 11). This finding is consistent with earlier findings by Cording and Hansmire (1975) and Kim et al. (1996). It is believed that the increase in settlement results from an increase in overlapping stresses occurring as the pillar distance decreases. The larger the pillar width, the more the maximum settlement shift towards the existing tunnel (Fig. 10).

3.4.3. Effects of excavation sequence

The maximum interaction between two tunnels occurs when the shield tail of the new tunnel passes over the measured section (He et al., 2012; Do et al., 2014b, 2015). Interaction of the twin tunnels gradually decreases when the new tunnel face is far from the measured section. The greatest surface settlement is observed when the two mechanized tunnels are simultaneously excavated. Due to the interaction of the twin tunnels, an increase in the surface settlement can be expected compared to that induced above a single tunnel. This could be explained by the accumulated loss of the ground in two tunnels.

Twin tunnel construction procedures have a great influence on the surface settlement. During the new tunnel advancement, the settlement trough shifts gradually towards the existing tunnel, in most studies. This asymmetric profile of the settlement trough has been observed through field measurements (Chen et al., 2011), analytical results (Suwansawat, 2006) and laboratory model tests (Chapman et al., 2006; 2007). The settlement is larger in the side of the tunnel excavated first (Wang and Wu, 2012). However, Chakeri et al. (2011) concluded that the various possible excavation sequences have minor effects on surface settlement.

The settlement trough caused by the excavation of the new tunnel is typically shallower and wider than the one caused by the existing tunnel during the excavation of twin tunnels through silty (Chen et al., 2011) and sandy soils (He et al., 2012). However, these results are contrary to

laboratory measurements in clay obtained by Chapman et al. (2007), where a greater settlement was observed above the second tunnel in clay. This difference could reflect the influence of the soil strength, type or drainage.

3.4.4. Effects of soil layering and inclination

The theoretical predictions of ground movements induced by tunnelling are usually based on the assumption that the ground is homogeneous. But in most cases, inclined and mixed face soil layering with different material properties are commonly encountered.

Lin (1996) and Chu et al. (2007) performed model tests to study the ground deformation due to tunnelling in layered soils. The experimental results demonstrate that the soil's non-homogeneity has significant effects on the observed ground deformation. For two-layered formations, settlement at the tunnel crown is reduced if there is a stiffer formation located above the formation containing the twin tunnels. However, displacements around the new second tunnel increase if the formation above the twin tunnels is weaker. For three-layered formations, the tunnels can be stable if protected by the upper and lower stiffer formations. On the other hand, displacements at the crown and invert increase if the tunnels are surrounded by weaker formations. Zhang et al. (2011) developed an analytical method for predicting tunneling induced ground movements in multi-layered formations that may be useful for preliminary design of tunnels.

Park and Adachi (2002) carried out both model tests and finite element analyses to understand the impacts of inclined soil layers on tunneling. They concluded that surface settlement increases when the ground deformation occurs along the direction of the inclination of the layers. For ground with relatively small inclination of layers, it may be effective to lengthen tunnel supports in the direction of stratification that is perpendicular to the direction of the inclined layers. However, for ground with relatively high inclination in its layers, it is effective to lengthen tunnel supports in the direction of the inclination of the layers.

3.5. Sub-surface settlements due to side-by-side tunnelling

Sub-surface settlements at various depths due to the excavation of a second tunnel in the presence of an existing tunnel are summarized in Fig. 12, using data from Chapman et al. (2006) and Divall (2013). It is

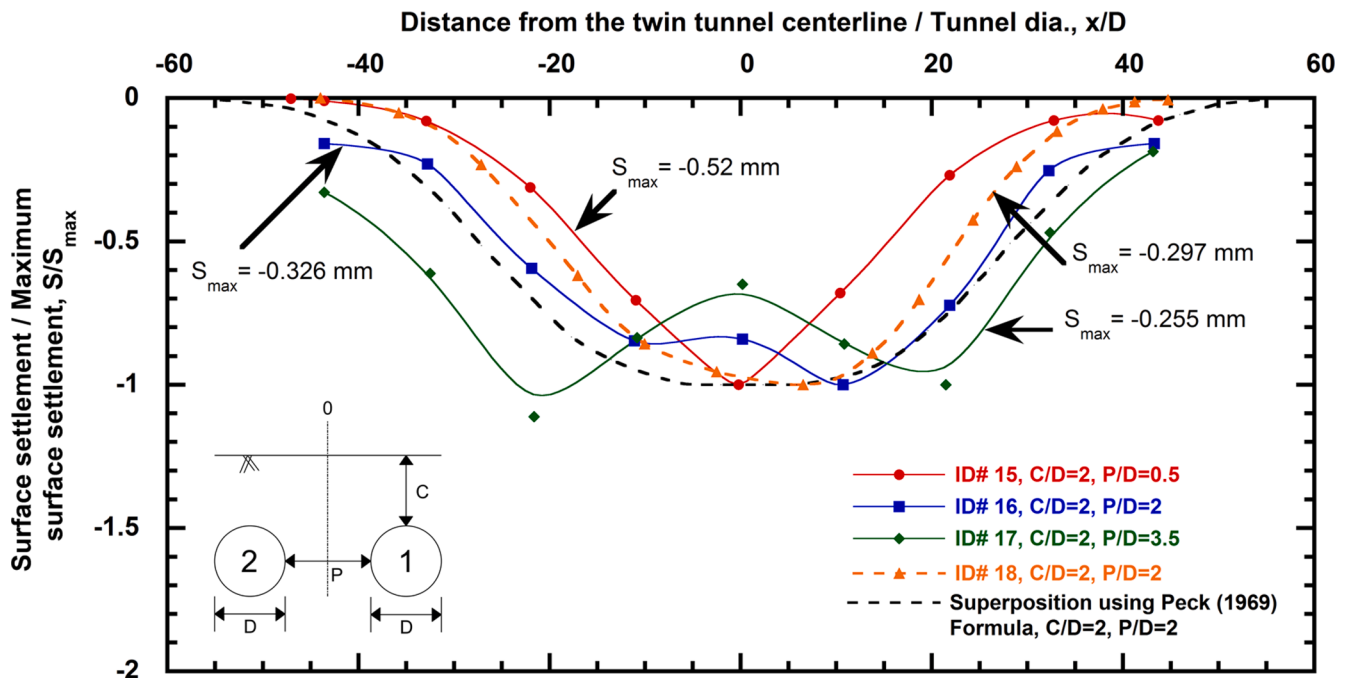


Fig. 11. Total surface settlement caused by two Side-by-Side twin tunnels (Based on data from Divall et al., 2012; Addenbrooke and Potts, 2001).

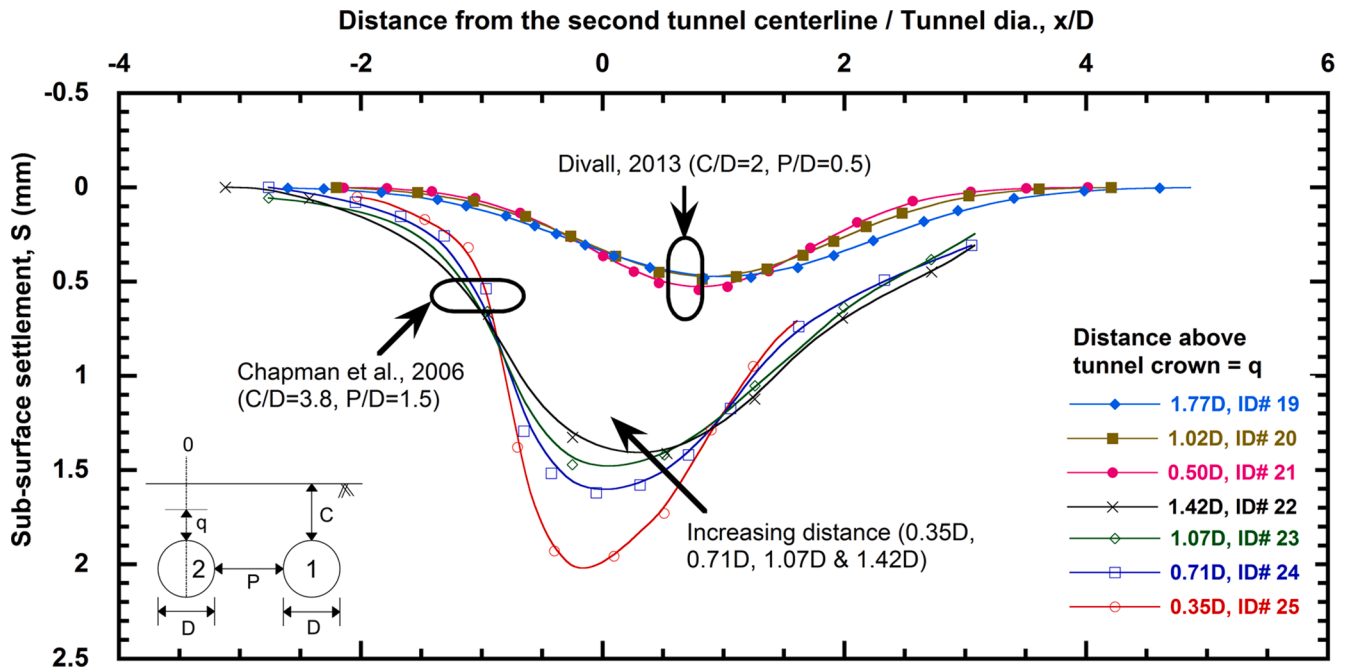


Fig. 12. Sub-surface settlement at various depths due to second tunnel excavation in presence of first tunnel.

evident that the deeper the sub-surface level, the narrower the trough width, and the larger the magnitude of maximum settlement. Non-symmetric troughs with wider trough width were obtained because of the previous straining in the ground caused by the first tunnel construction (Cooper et al., 2002b). The maximum sub-surface settlement point shift towards the new tunnel as cover depth increases (Fig. 12). Similarly, the maximum sub-surface settlement observed increases as the pillar width decreases (Fig. 13). The settlement troughs, however, shifts towards the new tunnel as the pillar width increases as evident in Fig. 14.

4. Settlements from piggyback/stacked (vertical alignment) twin tunnelling

One of the earliest studies on piggyback tunnelling was performed by Kuesel (1972), where he observed surface settlements above two pairs of side-by-side tunnels, which were constructed as part of the bay area rapid transit system (BART), one pair stacked above another. The study focused on the flexible ring used in the design of tunnel linings, particularly pertaining to the types of distortion that are anticipated under varying soil conditions. With respect to soil deformation, the authors reported that no cases of unacceptable differential settlement of

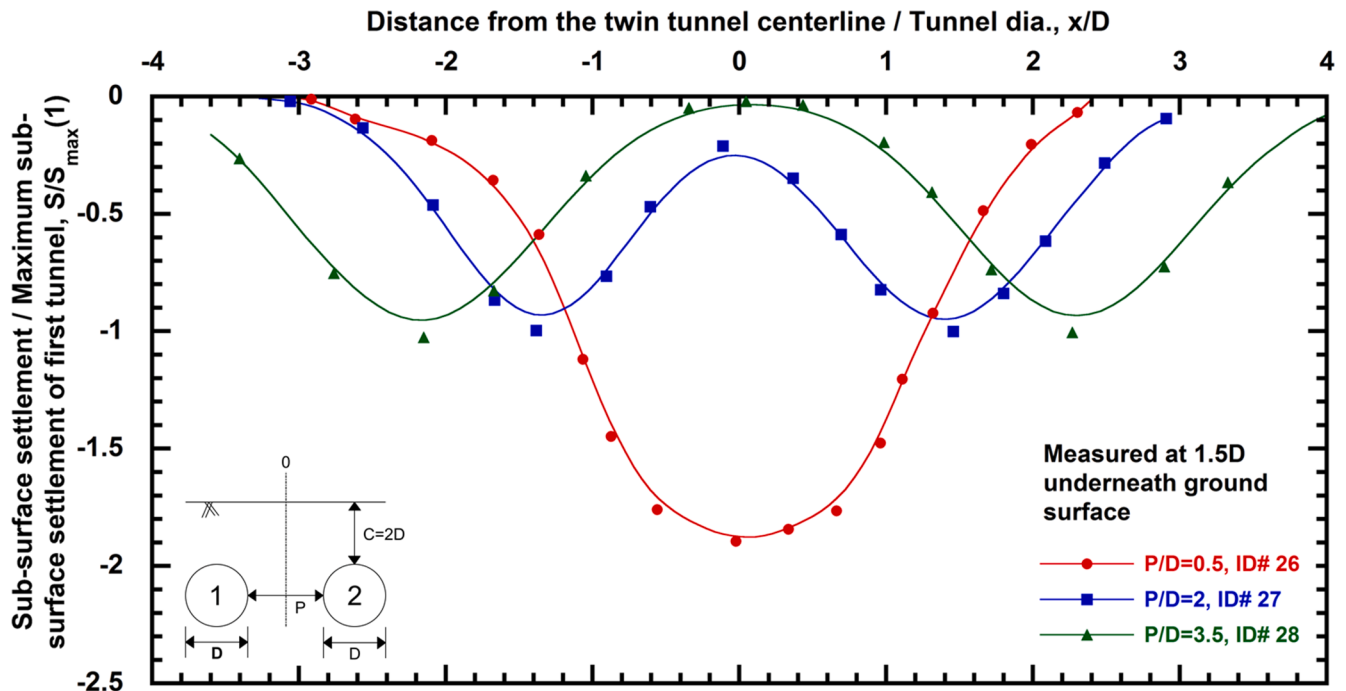


Fig. 13. Effect of pillar width on total sub-surface settlement due to twin tunnel excavation (Based on data from Divall, 2013).

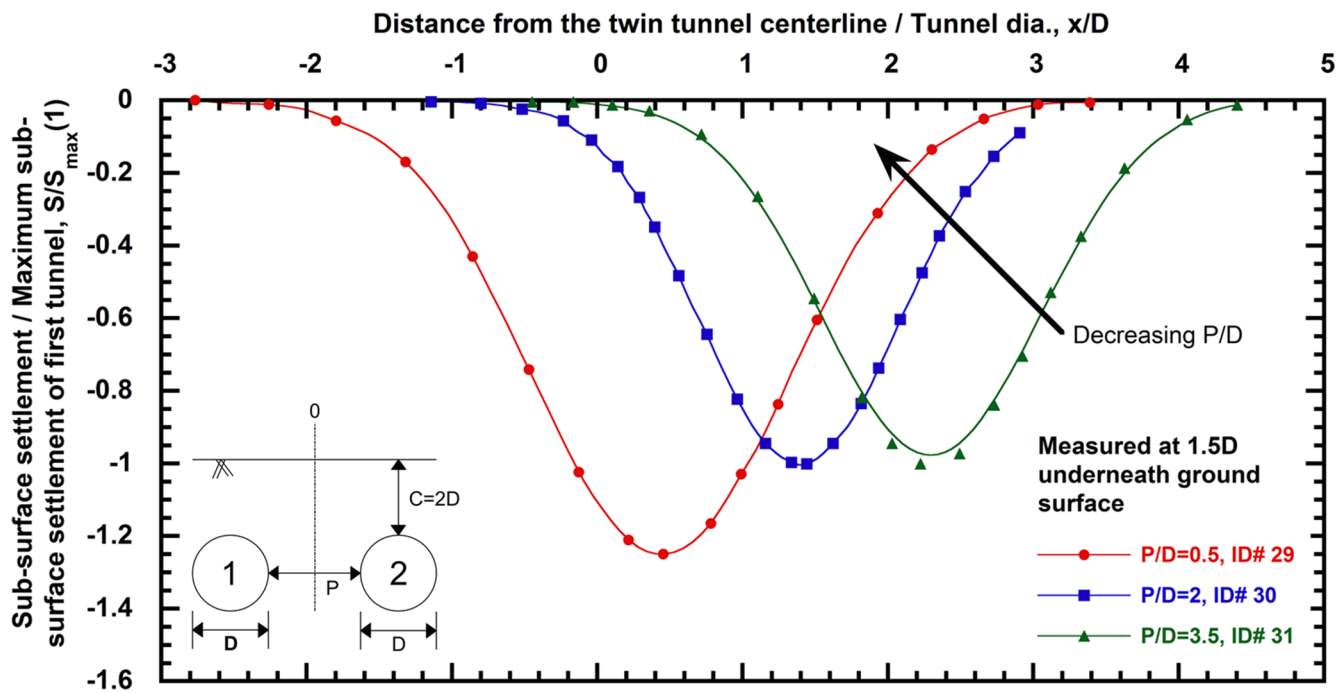


Fig. 14. Effect of pillar width on sub-surface settlement due to second tunnel in presence of first tunnel (Based on data from Divall, 2013).

buildings along the tunnels were observed. Wang and Chang (1992) reported that it is a common practice to drive the lower tunnel first during piggyback tunnelling. The authors concluded that, the ground settlement can be more than twice than that of a single tunnel because the upper second tunnel would be driven through a highly disturbed zone created by the construction of the lower existing tunnel. This report is supported by results from numerical studies and limited field observations suggesting that a wider settlement trough and larger settlement occur due to the excavation of the second tunnel above the first one (Cooper et al., 2000; Shahin et al., 2013). The shape of the settlement profile above the second tunnel excavation is also not of a Gaussian form, as assumed for greenfield situations.

The magnitude of the maximum settlement is dependent on the depth of the tunnels, spacing between the tunnels, construction sequence and the volume loss, which is influenced by the reduced soil stiffness in the zone of the second excavation. Fang et al. (2016) studied the ground surface settlement profiles due to the construction of closely spaced piggyback twin tunnels and found the maximum surface settlement of each cross-section after both the first and second tunnel passing is reported above the centerline. They found, the parameters describing a surface settlement trough, such as the ground loss percentage and the trough width are greatly influenced by the ground reinforcement schemes. The maximum surface settlements induced by each of the twin tunnels generally increase with the decrease of the overburden thickness under the same reinforcement schemes. They also suggested that a stronger ground reinforcement scheme can decrease the magnitude of surface settlement.

Koungelis and Augarde (2004) performed numerical simulations to investigate the influence of multiline tunnelling overlapped on existing tunnels. In their study instead of examining the surface settlement profiles exclusively, they looked for evidence of interaction from observing the predicted tunnel lining shapes. By observing the shapes, they concluded that settlements appear to be greater for closely spaced tunnels when the upper tunnel is excavated first. However, as pillar width increases, settlements are more significant when the lower tunnel is excavated first. Hunt (2005) concluded that, there is no relative increase in settlement taking place over the centerline of the existing tunnel when controlling for volume loss. He also reported the changes to

the settlement profile are caused through pre-failure soil stiffness alone and do not consider any increase in volume loss for the second tunnel.

4.1. Factors affecting piggyback twin tunnelling induced settlements

4.1.1. Effect of construction sequence

Many studies have been performed to understand the influence of construction sequence on the interaction between piggyback twin-tunnels and reported that it affects the soil settlement and internal forces. The soil settlement induced by the piggyback twin-tunnels at various construction sequences is shown in Fig. 15, using numerical data replotted from Do et al. (2014c). The construction of the upper tunnel first leads to higher settlement, compared to that obtained by the construction of the lower tunnel at first. This finding is consistent with that of Chehade and Shahrour (2008) and Channabasavaraj and Visvanath (2013). The data demonstrates that an increase in the surface settlement, compared to that induced above a single tunnel can be expected due to interactions between the twin piggyback tunnels. The maximum settlement computed above the piggyback twin tunnels was about 40% higher than that developed above a single upper tunnel. The figure also demonstrates the anticipated settlement due to one upper or lower tunnel only, along with the settlement predicted using Peck's formula for a single tunnel having an area equal to both tunnels. Peck's formula overestimated the magnitude of settlement. This could be attributed to the fact that the additional settlement caused by the excavation of the new tunnel through the soil mass, which has been disturbed by the excavation of the existing tunnel, is usually smaller than that induced when this tunnel is excavated first through an undisturbed zone. Addenbrooke and Potts (2001) also reported when the upper tunnel has been excavated first it is inevitably within the region of displacement caused by the lower second tunnel excavation. They likewise reported if the second tunnel is excavated above the first, the existing tunnel heaves upward, if the second tunnel is excavated below the first, the existing tunnel settles. However, to manage risk associated with twin tunnelling, it is common to excavate the lower tunnel first during piggyback twin tunnel construction.

The excavation of the piggyback twin tunnels can be concurrent or staggered. Li and Yuan (2012) reported that when the twin tunnels are

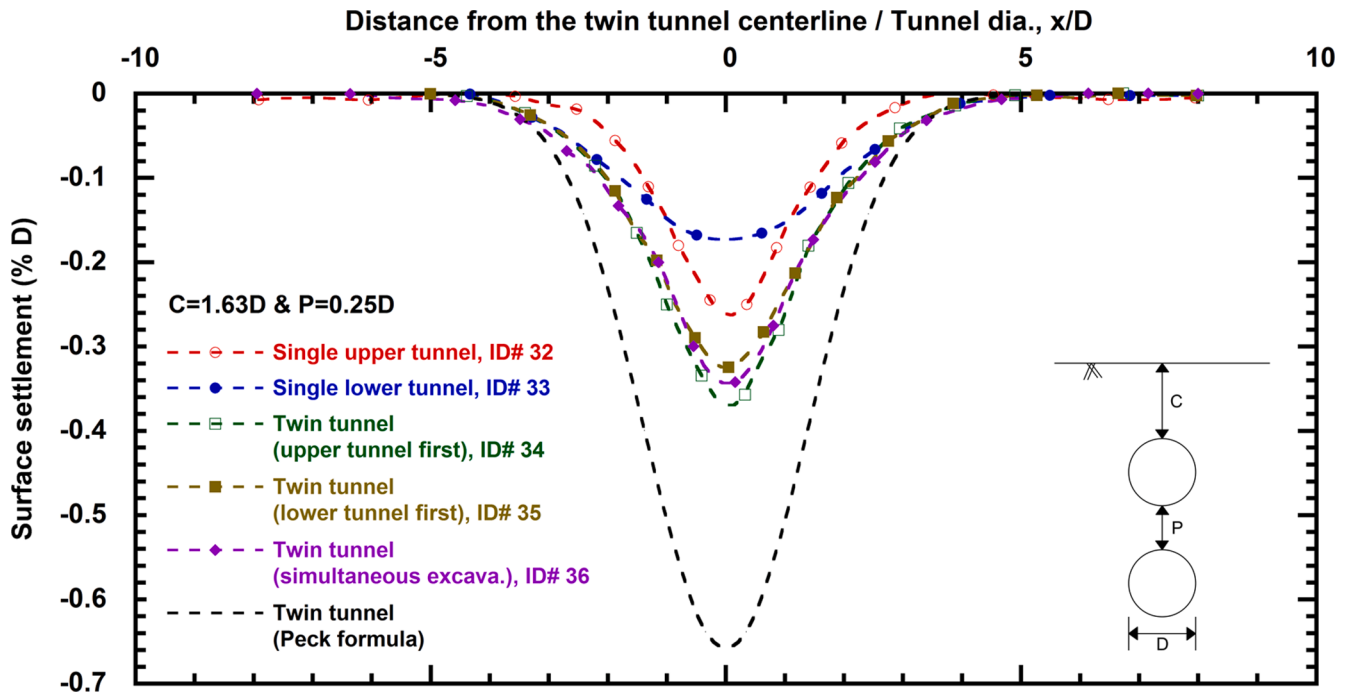


Fig. 15. Comparison of the settlement trough in the transverse section of the stacked tunnels for different construction procedures (Based on data from Do et al., 2014c).

excavated concurrently, the interactions are strong and cause the symmetrical settlements of the overlying tunnel. They also suggested that staggered new second tunnel excavation underneath the existing tunnel can be employed as an intelligent scheme to reduce overall settlement by using the top tunnel as reinforcement, but this opinion is not consistent with the data presented in Fig. 15, where the differences between various excavation schemes is relatively small.

4.1.2. Effects of cover depth

The magnitude of surface settlements can be influenced by both the spacing between two tunnels (pillar distance) and the cover depth of the upper tunnel. Do et al. (2014c) performed 3D numerical simulation of mechanized twin stacked tunnels and found, the deeper the tunnels, the smaller the displacement. This is also evident in Fig. 16, where surface settlement from piggyback tunneling is plotted using data from Hunt (2005). As mentioned earlier, when the new tunnel is excavated above the first, the existing tunnel heaves upward. However, it is evident from

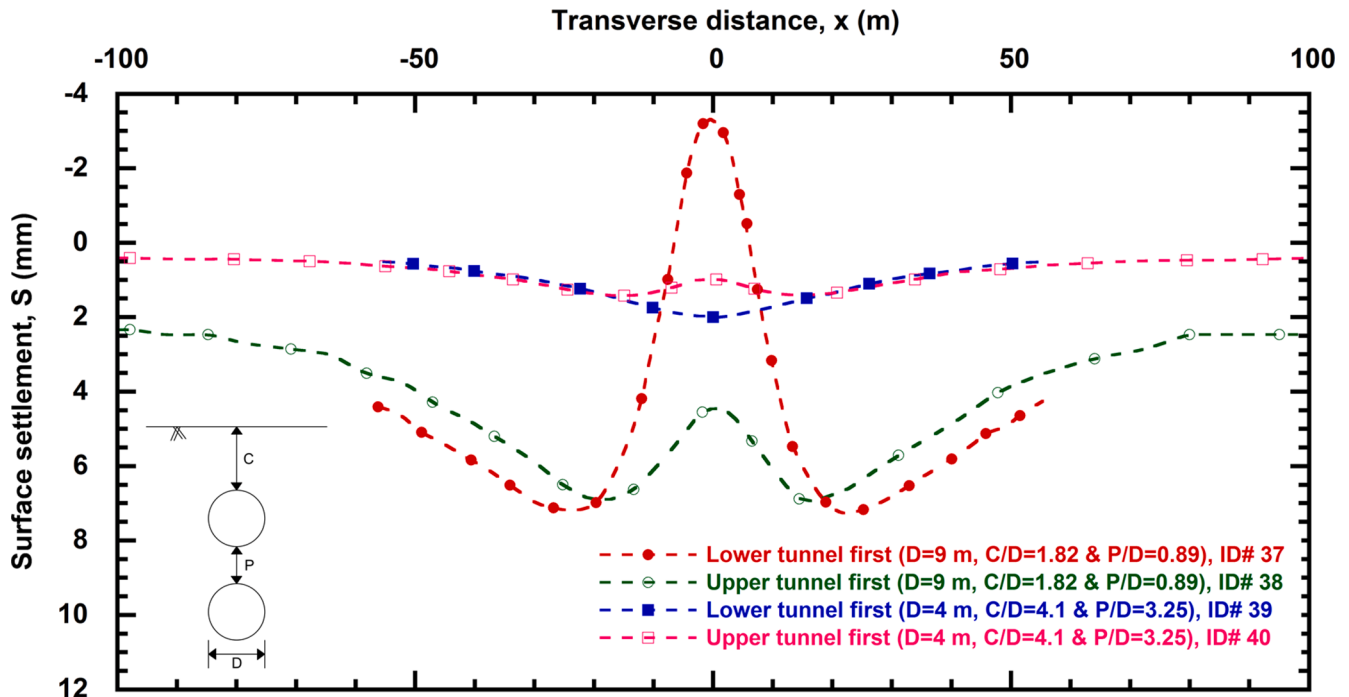


Fig. 16. Surface vertical settlement above second tunnel in presence of first tunnel for different cover depths and pillar distances (Based on data from Hunt, 2005).

Fig. 16 that, upheaval always occurs regardless of construction sequence when the cover depth (C) is less than twice of the tunnel diameter ($2D$). However, the available data is relatively limited to draw conclusive inferences.

4.1.3. Effects of tunnel spacing/pillar distance

Addenbrooke and Potts (1996) and Addenbrooke et al. (1997) performed finite element analysis to study ground movements and lining behavior due to piggyback twin tunnel construction and presented settlement above the new second tunnel. They found that, the closer the spacing, the flatter the settlement profile and the wider the trough width of settlement profile becomes above the tunnel center line (Fig. 17). Addenbrooke and Potts (2001) performed two-dimensional FE analyses of multiple tunnels using a non-linear elastic-perfectly plastic soil model and found the pillar depth at which interaction ceased at pillar distances greater than is $1D$. Koungeles and Augarde (2004) reported that where a second tunnel is driven above an already existing one, small interaction effects seem to be present at a pillar depth distance up to one diameter, but disappear beyond three diameters. At a pillar distance of $3D$ the in-situ stresses at the location for the second tunnel are unchanged from their greenfield counterparts, and hence the surface settlements induced because of the new second tunnel excavation are similar to those from a single isolated tunnel. However, when the upper tunnel constructed first interaction effects seem to appear no matter how deep the second tunnel was driven. Experimental data from a three-station tunnel construction close to existing tunnels on the Piccadilly line in London support these numerical findings where no interaction for pillar depths beyond $6D$ and $7D$ (Cooper et al., 2002b).

The settlement profile of the second tunnel is always wider, and the shape is different than the greenfield profile irrespective of pillar depth. This is illustrated in Fig. 18, where settlement, S , has been normalized by the maximum settlement, S_{max} and the distance from the tunnel centerline, X , has been normalized by the tunnel diameter, D .

5. Settlements from perpendicularly crossing twin tunnels

It is sometimes unavoidable that new tunnels run perpendicular to,

above or below, existing tunnels; and in these cases, the response of the existing tunnel to the under-crossing or above-crossing shield tunnel is of great concern. Perpendicularly crossing tunnels in soft ground will inevitably disturb the surrounding soil, which may induce adverse effects on adjacent surface and sub-surface structures. Interaction between closely-spaced tunnels has been studied in the past using a variety of approaches including field observations and theoretical analyses (Sharma et al., 2001; Hu et al., 2003; Huang et al., 2006; Klar et al., 2008; Li and Yuan, 2012; Zhang et al., 2013; Fang et al., 2015), physical model tests (Byun et al., 2006; Marshall et al., 2010), empirical/analytical methods, and finite element analysis (Dolezalova, 2001; Liao et al., 2009; Liu et al., 2011; Jiang and Yin, 2012; Li and Du, 2012a, 2012b; Zhang and Huang, 2014). Attewell et al. (1986) reported methods for finding the movement of pipes due to tunnels driven underneath at a skew and this method could presumably be applied, with caution, to tunnels.

Field observations of the interactions between closely spaced crossing tunnels on the Jubilee Line Extension in London were conducted by Kimmins et al. (1996) to measure the deformation created in existing tunnels at the crossing point caused by cross-cutting excavations. They reported that the movement of existing overlying tunnels, situated at 90° to a new tunnel construction below, could be assumed to deform to a shape that was identical to a green-field sub-surface settlement profile. These findings have been confirmed by Standing and Selman (2002). Similarly, Mohamad et al. (2010) adopted a distributed strain sensing technique to examine the performance of an old masonry tunnel during the construction of a tunnel beneath it. They observed a symmetrical strain pattern between the two sides of the tunnel when the new tunnel was located directly below the old tunnel. It's evident from Fig. 19 that, (1) settlement follows a Gaussian distribution, with the maximum vertical displacement occurring in the plane of symmetry during the construction process of the new shield tunnel, and (2) that more settlement occurs away from the plane of symmetry, than what would be predicted using Peck's distribution. However, with the increase of offset from the plane of symmetry, the vertical displacements gradually decline and finally perish.

Yamaguchi et al. (1998) analyzed the construction of four subway

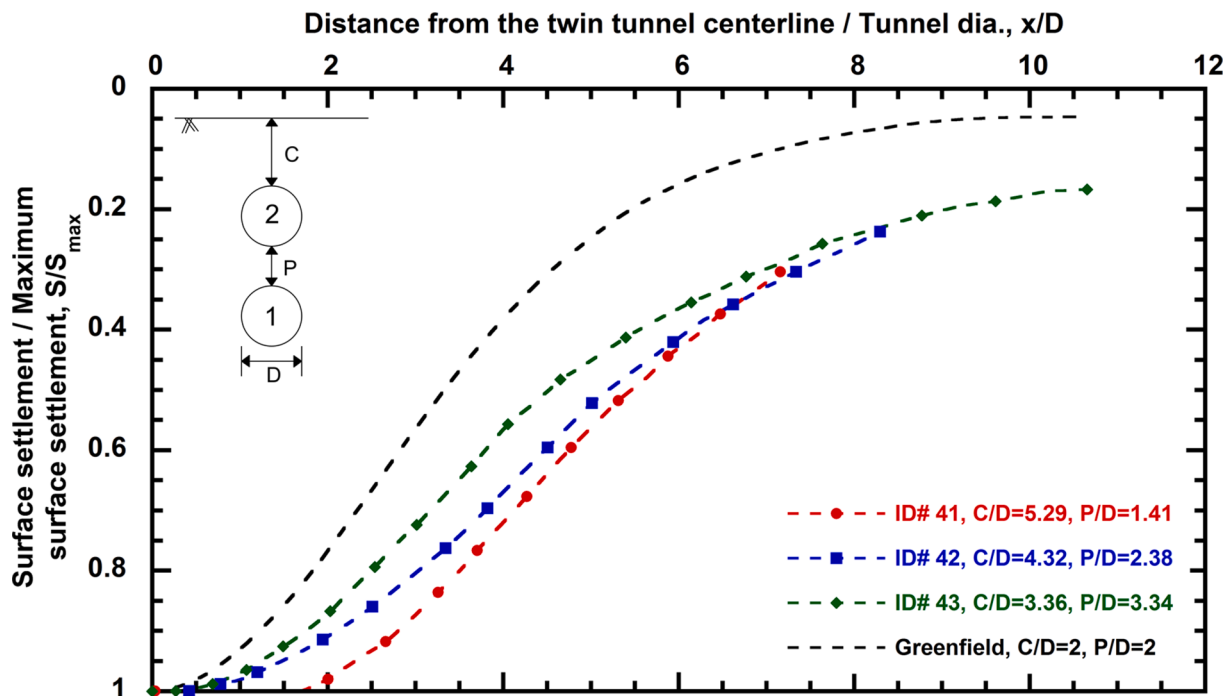


Fig. 17. Surface vertical settlement above second tunnel in presence of first tunnel (Based on data from Addenbrooke and Potts, 1996; Addenbrooke and Potts, 2001).

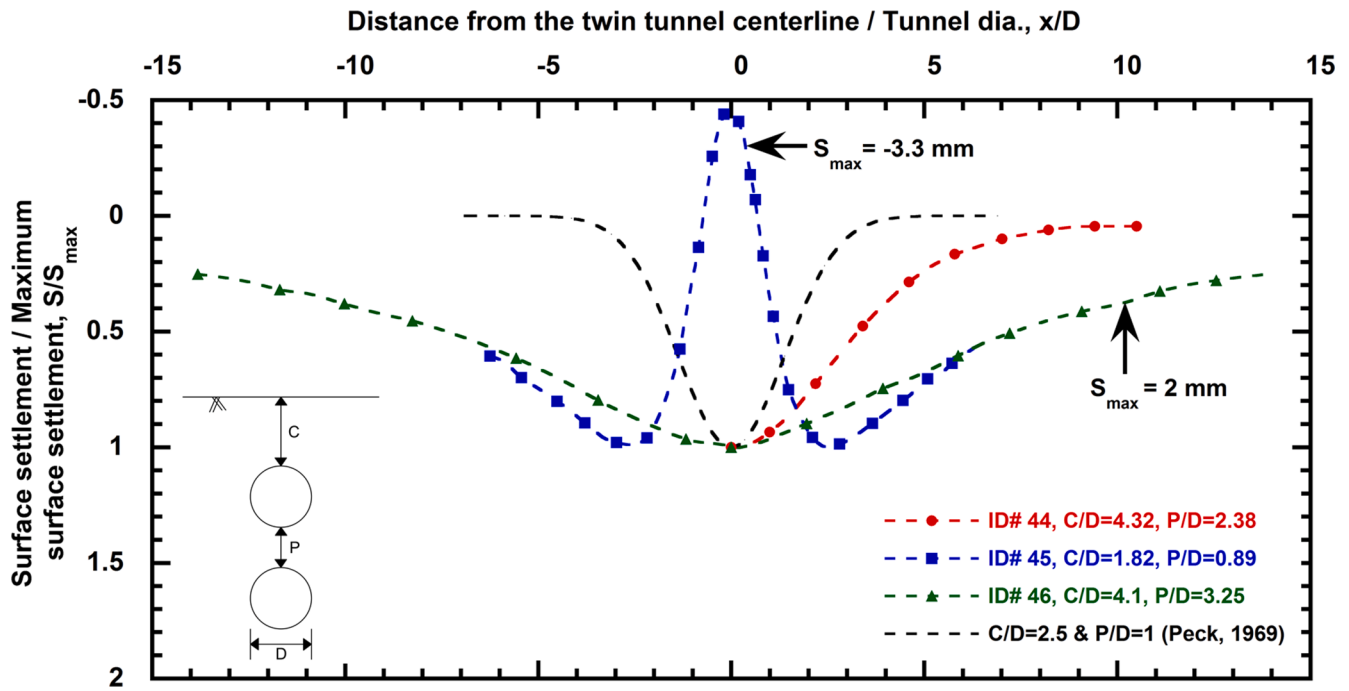


Fig. 18. Surface vertical settlement above second tunnel in presence of first tunnel (Based on data from Addenbrooke and Potts, 1996; Hunt, 2005).

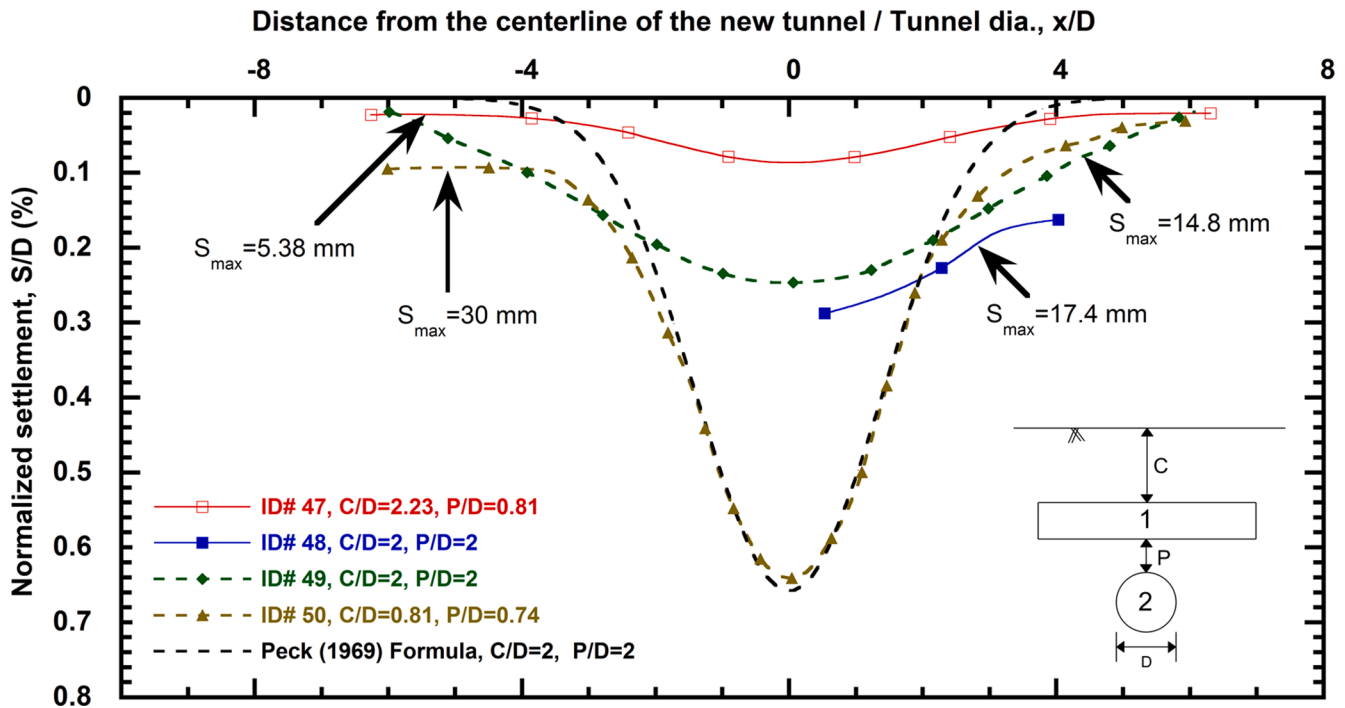


Fig. 19. Surface vertical settlement of existing tunnel (Based on data from Li et al., 2014; Ng et al., 2015; Chakeri et al., 2011).

tunnels that run close and intersect each other in Kyoto City, and concluded that a redistribution of the ground stress is evident from shield excavations when a shield was crossing an existing tunnel. The results of 1 g model tests performed by Kim et al. (1998) to study the interaction of perpendicular crossing tunnels suggests that, interaction effects are predominately caused by the jacking forces applied to the liner and the model tunnelling machine during tunnel installation.

The vertical displacement of the existing tunnel typically increases because of the excavation of the new shield tunnel (Li et al., 2014). However, the vertical displacement may also decrease if grout is injected

for the new tunnel. Thus, the vertical displacement exhibits an approximately linear change with increases in ground loss ratio and an inverse linear relationship with the grouting ratio of the new shield tunnel. The settlements of the existing tunnel were also observed to be larger when the effects of volume loss alone were simulated than when the effects of both volume loss and weight loss were modeled simultaneously Ng et al. (2013). This is because weight loss caused stress relief, which resulted in a reduction in the amount of tunnel settlement induced by volume loss.

5.1. Factors affecting settlements induced by perpendicularly crossing tunnels

5.1.1. Effects of cover depth

For single tunnels, it is well-understood that tunnelling induced settlement is larger when tunnelling at a shallower depth (*i.e.*, reducing C/D) (*e.g.*, Mair and Taylor, 1997; Marshall et al., 2012). The vertical displacement of the existing tunnel at the end of excavation on a perpendicularly intersecting tunnel are shown in Fig. 20, where the vertical displacement is normalized by the diameter of the new tunnel and the C/D ratio refers to the new second tunnel. When the new tunnel was excavated underneath the existing tunnel it is found that as the cover depths of the existing and new tunnels increase, settlement of the existing tunnel due to the new tunnel construction beneath it decrease. This is because with the larger cover depths of the tunnels, the increase in mobilized shear stiffness of the soil dominated the increase in stress relief caused by the tunnel excavation (Boonyarak and Ng, 2014). However, when the new tunnel advanced above the existing tunnel, heave of the existing tunnel increased with increasing C/D (*i.e.*, heave is larger when $C/D = 3.5$ than $C/D = 2$).

5.1.2. Effects of pillar depth

Little information is available about the effect of pillar depth on perpendicularly crossing tunnel induced settlements. In the case of piggyback tunnels, the effects of the second tunnel construction on the intersecting tunnel depend to a large degree on the spacing between the tunnels. When the distance between them is great, such as seven diameters, presented by Addenbrooke and Potts (2001) in London Clay, the two tunnels are expected to have no influence on each other. The influence of the first tunnel is negligible, and the settlement profile is practically centered on the second tunnel. The closer the tunnels, the greater their interaction is.

To investigate the influence zone of perpendicularly crossing-tunnel interaction two tests were performed by Ng et al. (2015) with pillar-diameter ratio of $P/D = 0.5$ and $P/D = 2$. In their study, the newly constructed tunnel crossed below the existing upper tunnel perpendicularly. The tests were conducted in a centrifuge using sand as the soil. In these tests the C/D of the existing upper tunnel was kept constant at 2,

and the P/D was varied. It's evident from Fig. 21 that, maximum measured tunnel settlement in the test where $P/D = 0.5$ is about 40% larger than that in the test where $P/D = 2$. This is because of the larger reduction in vertical stress along the invert of the existing tunnel when pillar depth to diameter ratio is low which means that the new tunnel is in close proximity of existing tunnel. Different tunnel deformation mechanisms were observed with different P/D ratios. The existing tunnel was elongated horizontally when $P/D = 0.5$. This is because stress reduction in the horizontal direction was greater than that in the vertical direction. The stress relief caused by the new tunnel not only led to a reduction in the vertical stress at the invert, but it also resulted in substantial stress reduction at the springline of the existing tunnel. On the contrary, the existing tunnel was elongated vertically as the new tunnel advanced at $P/D = 2.0$ since the reduction in stress in the vertical direction dominated.

5.1.3. Effects of construction sequence

The influence of construction sequence on crossing-tunnel interaction is complex. When a pipeline is crossing a tunnel, the maximum soil movement transverse to the pipe occurs when the pipeline is directly above the tunnel (Attewell et al., 1986). This behavior is likely to be true for a tunnel crossing a tunnel.

Liu et al. (2009) found that the interaction between perpendicularly crossing tunnels during the tunnel advancing process was larger than those at the end of tunnel excavation. They also reported compressive failure of the concrete lining at the crown of the existing tunnel that was observed when the new tunnel is constructed perpendicular to and above the existing tunnel. In contrast, tensile strain caused cracks to appear in the lining of the existing tunnel at the springlines when the new tunnel was excavated underneath. The tunneling of the crossing tunnels may be concurrent or staggered. When the twin tunnels are constructed simultaneously, the interactions are strong and cause the symmetrical settlements of the overlying tunnel, as indicated by the nearly concurrent construction of the Jubilee Line Extension (JLE) under the Bakerloo and Northern Line tunnels presented by Standing and Selman (2002). An appropriate construction sequence for crossing tunnels can help minimize the adverse impact on the existing tunnel.

To investigate the effects of construction sequence on crossing-

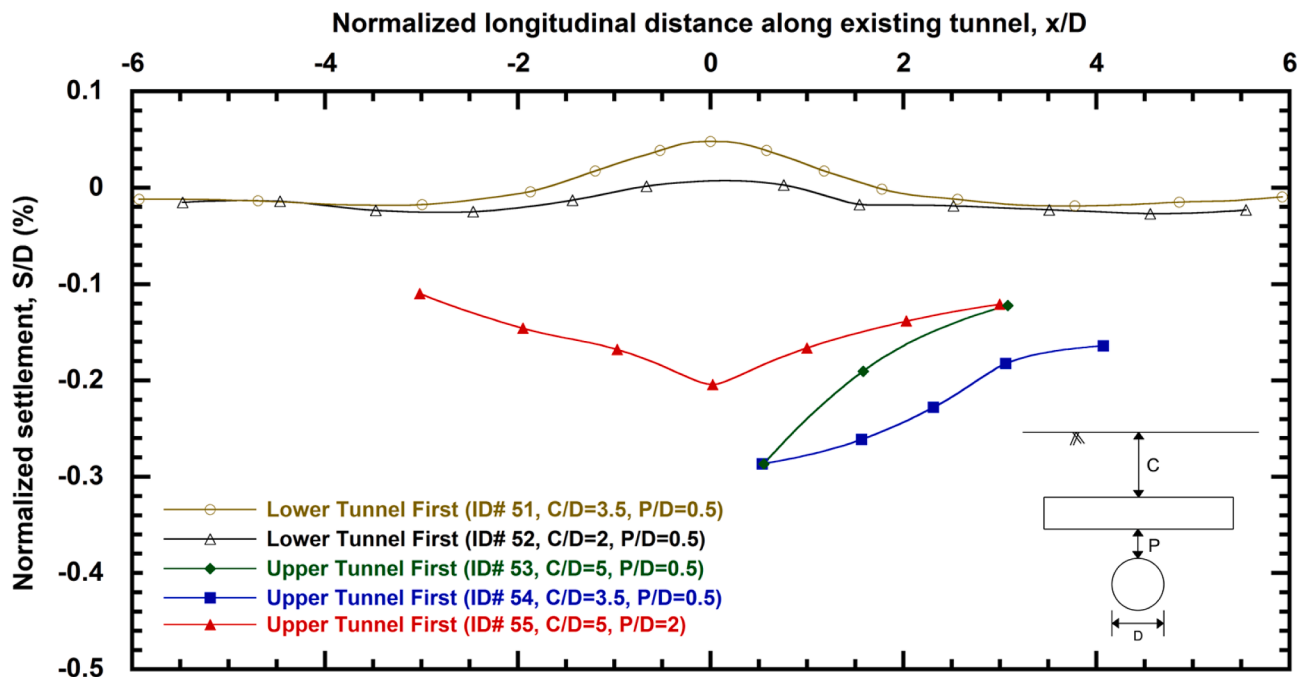


Fig. 20. Comparison of surface vertical settlement of existing tunnel for different geometric parameters and construction sequence s (Based on data from Boonyarak and Ng, 2014, 2015).

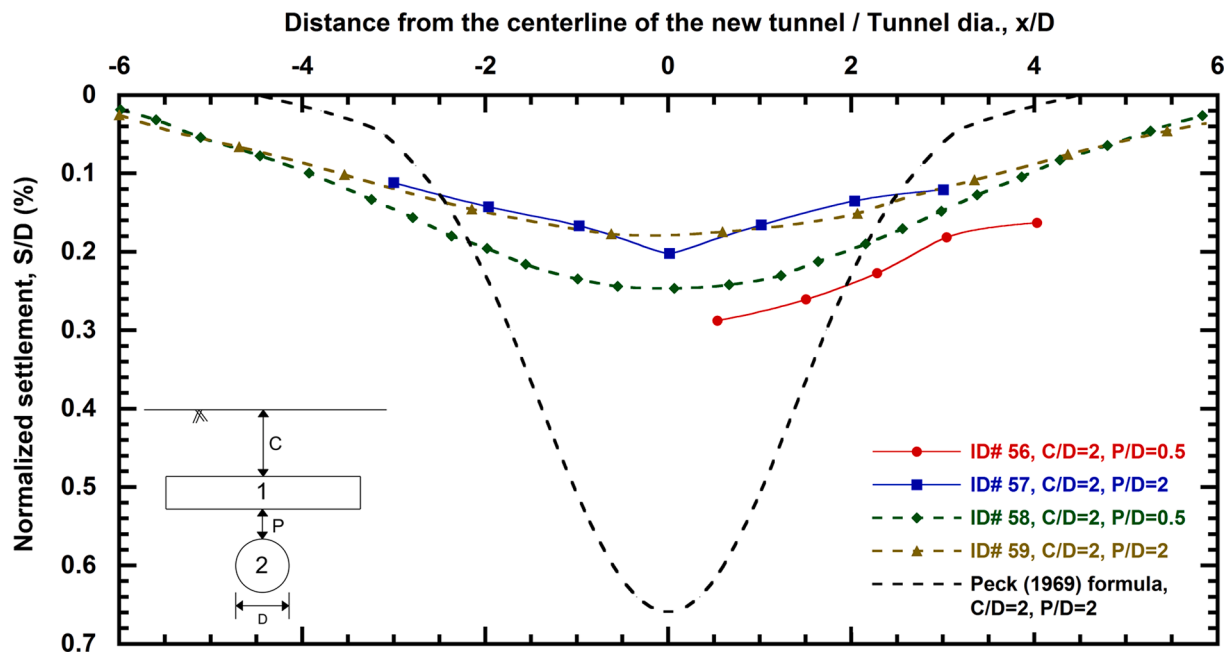


Fig. 21. Surface vertical settlement of existing tunnel for different pillar depth (Based on data from Ng et al., 2015).

tunnel interaction Boonyarak and Ng (2014, 2015) carried out three-dimensional centrifuge tests and a numerical back analysis. In one test, the new tunnel was excavated beneath the existing tunnel in a reference test, while in the other test the new tunnel advanced above the existing tunnel. Vertical displacement of the existing tunnel at the end of excavation are shown in Fig. 20, where the vertical displacement is normalized by the diameter of the new tunnel. The existing tunnel was vertically compressed when the new tunnel was excavated underneath, but vertically elongated when the new tunnel advanced above. These observations are in agreement with field observations reported by Saitoh et al. (1994). One explanation is that the reduction of stress acting on the existing tunnel in the horizontal direction was larger than in the vertical direction when the new tunnel was constructed beneath. On the other hand, the decrease in vertical stress on the existing tunnel was larger than the horizontal stress reduction when the new tunnel was excavated above. The magnitude of the measured vertical displacement of the existing tunnel caused by the new tunnel excavation beneath was much larger than when the new tunnel advanced above, which was also inferred from FEM analyses carried out by Kim et al. (1996). In these 3D centrifuge tests reduction in vertical stress acting on the existing tunnel was larger in the test conducted beneath an existing tunnel than in the opposite construction sequence.

6. Settlements from offset arrangement twin tunnelling

It is now common to have tunnels running parallel to each other, but at various elevations while maintaining horizontal distance. These tunnels are referred to as offset arrangement twin tunnels or diagonally aligned twin tunnels (Fig. 1d). As the tunnel excavations are undertaken at different elevation levels, there will be interactions that can have a significant influence on stress distributions and consequently deformations within the tunnels and surface settlement.

It is important to distinguish offset arrangement twin tunnels from both side-by-side and piggyback twin tunnels. Fang et al. (2016) found that, newly constructed offset arrangement tunnels demonstrated twice as much settlement than newly constructed piggyback tunnels. Addenbrooke (1996) found that offset arrangement tunnels demonstrate characteristics of both side-by-side and piggyback tunnels. Standing et al. (1996) reported larger volume loss for the new offset arrangement upper tunnel together with a bigger trough width and maximum

settlement offset towards the first tunnel excavated at St. James Park, London. Conversely, Nyren (1998) reported no offset in the position of the maximum settlement above the second tunnel driven, at the same location.

6.1. Factors affecting offset arrangement twin tunnelling induced settlements

6.1.1. Effect of angular spacing and angular relative position

Offset arrangement tunnels can be described in terms of the angular relative position of a new tunnel relative to a parallel existing tunnel (Hefny et al., 2004). The angular relative position is measured by the angle θ between the center-to-center connecting line of two tunnels and the horizontal line drawn from the center of existing tunnel (Fig. 1d). An angle of 90° represents a new tunnel directly above the crown or below the invert of an existing tunnel (piggyback tunnels), while an angle of 0° represents a new tunnel located beside and at the same depth as the existing tunnel (side-by-side tunnels). It is observed that, as the angle between the tunnels increases with respect to horizontal axis, the surface soil settlement decreases (Divall, 2013; Channabasavaraj and Visvanath, 2013).

Tunnel offset can also be described in terms of angular spacing, Q (Fig. 1d), although it is a less sensitive measure than the angular relative position, since distance does not always correlate with influence zone. Influence of tunnel angular spacing on offset arrangement twin tunnels has been numerically studied by Chehade and Shahrour (2008) and experimentally by Divall (2013). The influence of the angular spacing on the soil settlement above newly constructed upper tunnel is summarized in Fig. 22. For offset arrangement tunnels excavated in sand, the larger the angular spacing, the greater the vertical settlement. On the contrary, tunnels excavated in clay demonstrate opposite behavior. However, in all cases, the larger the angular spacing, the more the maximum settlement shifts towards the new tunnel.

6.1.2. Effect of excavation sequence

Four closely spaced subway tunnels have been monitored during construction in Kyoto City, Japan. At each stage of construction, Yamaguchi et al. (1998) observed the influence of one tunnel on another existing offset arrangement tunnel. Large subsidence of existing tunnels was observed during construction of lower tunnels; while relative

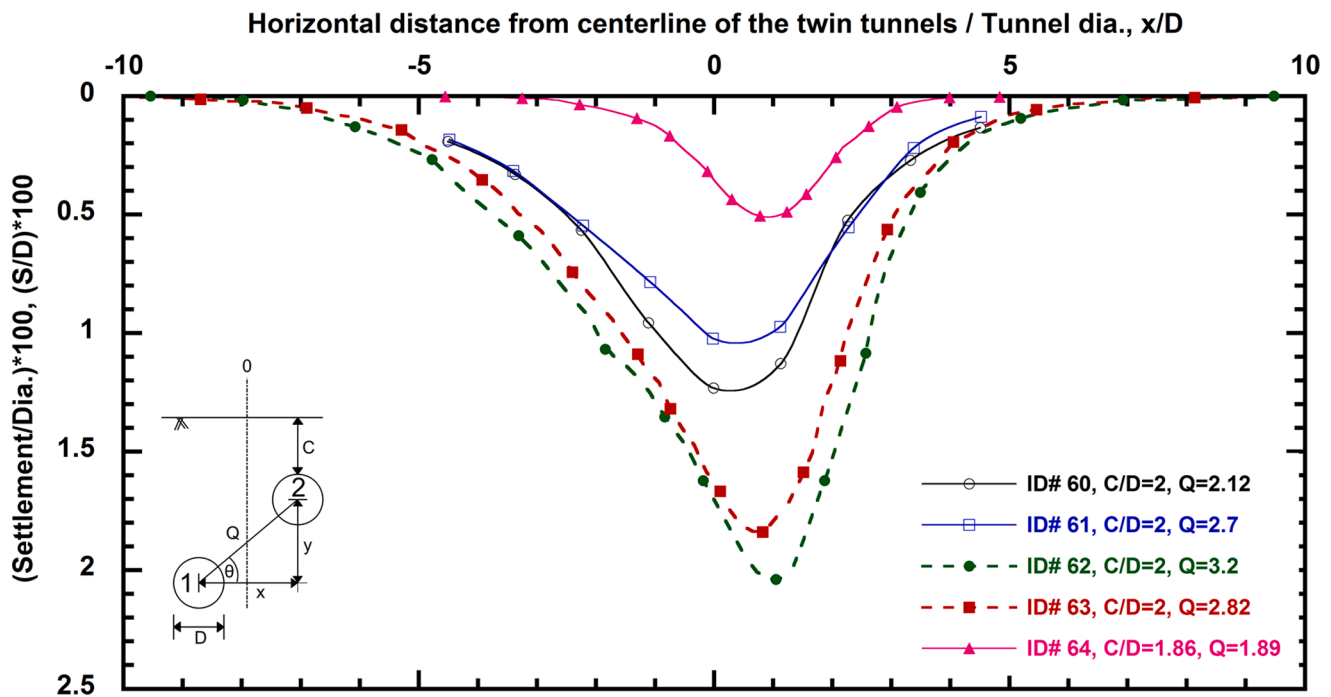


Fig. 22. Surface Vertical settlement above second tunnel in presence of first tunnel (Based on data from Divall, 2013; Chehade and Shahrour, 2008; Chen et al., 2012).

upheaval was occasionally observed when upper tunnels were constructed above two preceding tunnels, and the resulting subsidence of the new tunnel was very small, presumably because the existing tunnels provided reinforcement support.

Shirlaw et al. (1988) reported the movements for offset arrangement tunnels constructed using the NATM in stiff boulder clay. The 6.0 m diameter tunnels had a separation of only 1.7 m between the outer linings. The surface settlement profiles are shown in Fig. 23. The figure also presents data reported by Divall (2013) and Du and Huang (2009). The settlement trough was wider and deeper over the shallower tunnel, which is the second tunnel driven. The influence of the construction sequence is also shown in Fig. 24. It can be observed from Fig. 24 that the construction of the upper tunnel at first leads to higher soil settlement than that induced when the lower tunnel is first constructed.

When the lower tunnel is constructed first, it provides stiffness within the soil, and helps shield the previously strained soil above the new tunnel from further settlement. Therefore, it causes lower soil settlement than that for when upper tunnel constructed first. It can be concluded that the effect of this previously strained soil on the profile above the second tunnel is less apparent when driving the lower tunnel second.

Offset arrangement tunneling induced settlement data reported by Hunt (2005) has shown that the settlement profile is directly influenced by the construction sequence. When the upper tunnel is constructed first, at close center-to-center spacing of the tunnels there are reductions in displacement over existing tunnel. This is opposite to the behavior found when constructing the lower tunnel first. The differences in behavior for this case are due to the presence of a tunnel within the bounds to movement for the second tunnel. The behavior is similar to that of the piggyback tunnel case when constructing the upper tunnel first.

In any case, available data is insufficient to make definitive conclusions for offset arrangement tunnels. More definitive conclusion can be made when additional case history data and/or numerical/experimental studies for tunnels constructed using this sequence becomes available.

7. Summary and conclusions

7.1. Technical summary

This paper is focused on ground settlements due to twin tunnel excavation, and it summarizes the degree to which the excavation sequence, pillar width, and cover depth affect the magnitude and extent of ground settlement. The following observations represent a brief summary from the preceding review of available literature:

- Side by Side Tunnels:** Staggered construction of side by side twin tunnels is recommended, because simultaneous excavation may result in more surface settlement. For cases when a tunnel is excavated near an existing tunnel, a relative increase in settlement typically occurs directly above, or next to the centerline of the existing tunnel, in comparison to a similarly sized single tunnel. Interaction between the two tunnels and the resulting induced settlements decrease as spacing increases; and it is generally safe to assume that there is no effective interaction between the two tunnels beyond a spacing of $3D$. The changes in the settlement profile are caused by the redistribution of soil displacements that are primarily related to changes in soil stiffness. The eccentricity (distance between the existing tunnel centerlines and maximum settlement) decreases with increased tunnel spacing and depth. Surface settlement decreases as cover depth increases, but it is not possible to identify the depth where interaction effects cease to occur without further studies.
- Piggyback Tunnels:** The behavior of piggyback tunnels is different than that of side-by-side tunnels, owing to the fact that soil strains resulting from side-by-side tunnelling are similar in magnitude, since both tunnels are located at the same depth. For piggyback tunnels the resulting soil strains are different, with the upper second tunnel typically inducing more strain than the existing lower tunnel because it is excavated in previously strained soil. Additionally, the soil strength is depth dependent especially for frictional materials, with shallower layers typically possessing a lower strength. In general, the displacement profiles above the new second tunnel are perfectly

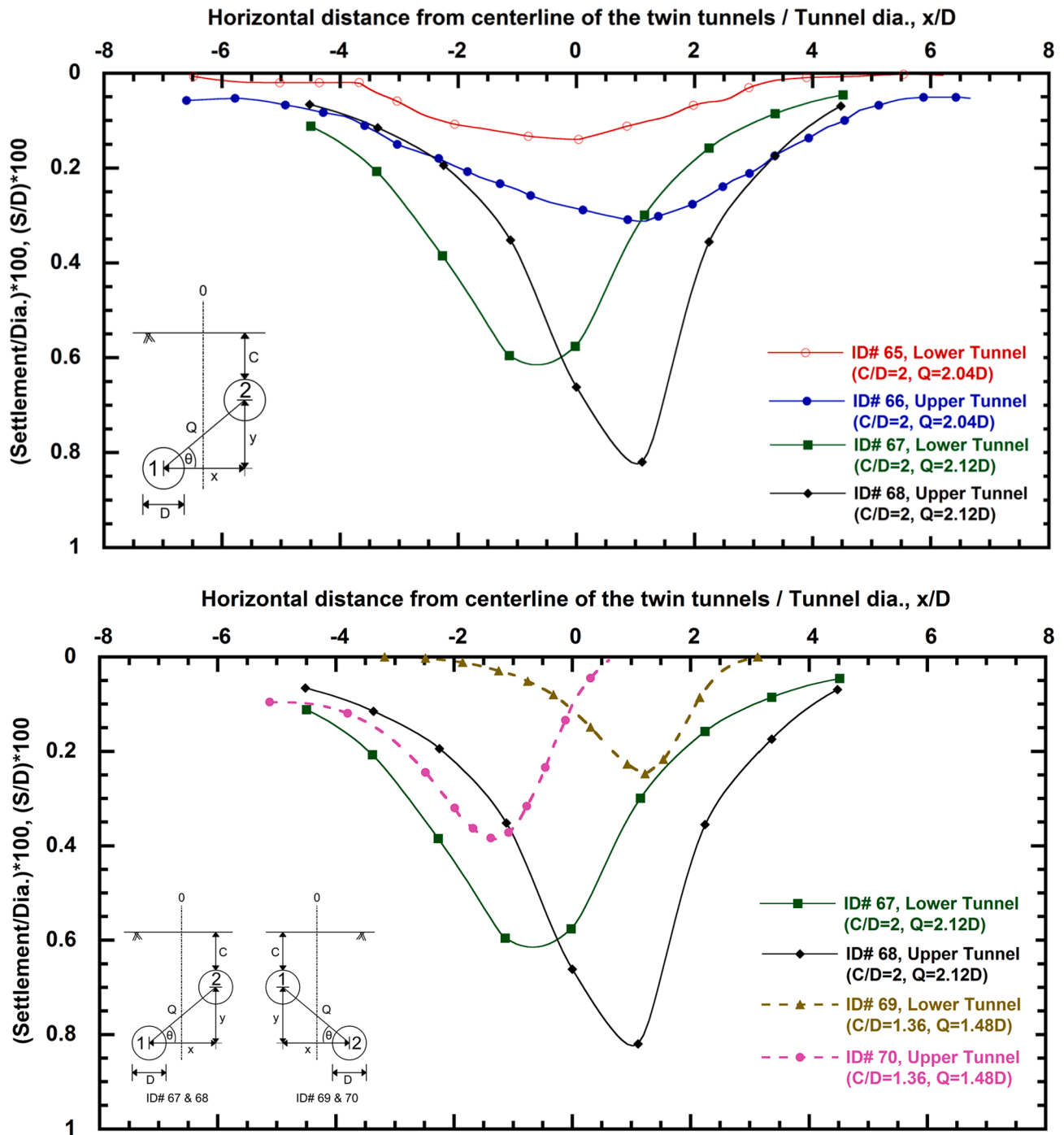


Fig. 23. Individual surface vertical settlement induced by both first and second tunnels (Based on data from Shirlaw et al., 1988; Divall, 2013; Du and Huang, 2009)
 Note: Both figures depict similar information for different tunnels to reduce clutter.

symmetric for most observations, regardless of the construction sequence, but they are not generally of Gaussian form. When constructing the lower tunnel first, the displacement profile above the second tunnel is perfectly symmetric in most observations, but there is a chance of ground upheaval, rather than settlement. In this case, the larger the tunnels the higher the chance of heave. On the other hand, when the second tunnel is constructed below an existing one, interaction typically occur regardless of the depth. Additionally, during the excavation of the second tunnel the earth pressure increases at the springline and decreases at the tunnel invert.

- **Perpendicularly Crossing Tunnels:** The settlement of the existing tunnel due to the new crossing tunnel excavated underneath

increases as the cover depth to diameter (C/D) ratio of the existing tunnel decreases. This is because the shear stiffness of soil around the existing tunnel typically increases with depth. The magnitude of settlement of the existing tunnel when the new crossing tunnel is excavated beneath is significantly larger than the magnitude of heave of the existing tunnel when the new crossing tunnel is constructed above. This is because a larger volume of soil is impacted while crossing beneath than crossing above. Heave due to new crossing tunnel excavation above the existing tunnel increased with increasing C/D s of both the existing and new tunnels. This occurs due to a relatively larger relief of stress acting on the existing tunnel at larger C/D s than at shallower ones. Additionally, a larger reduction

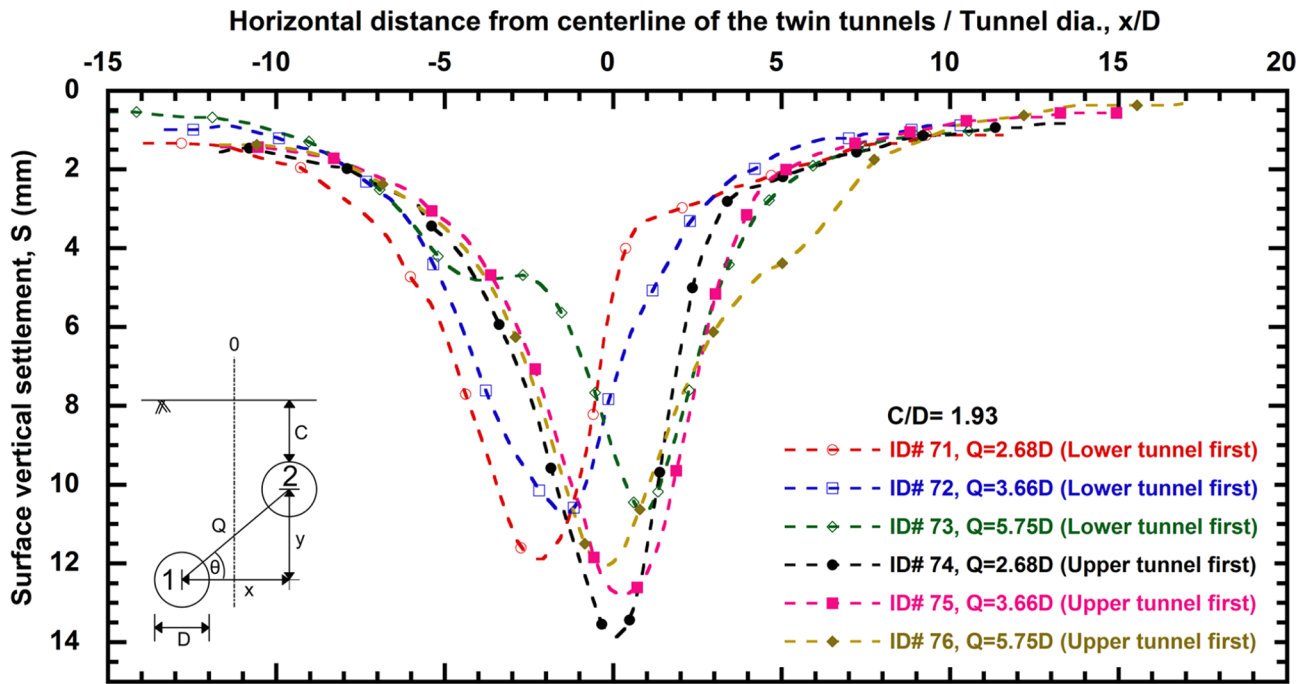


Fig. 24. Surface vertical settlement above second tunnel of two Offset Arrangement twin tunnels (Based on data from Hunt, 2005).

in vertical stress along the invert of the existing tunnel occurs when the pillar to diameter (P/D) ratio is low, which results in increased settlement when P/D ratio decreased.

- **Offset Arrangement Tunnels:** The settlement profile due to offset arrangement tunnelling has characteristics of both side-by-side tunnels and piggyback tunnels. The offset arrangement can be viewed as a middle point in a spectrum ranging from horizontally aligned (side by side) on one end, and vertically aligned (Piggyback) on another, as the relative angular position, θ , increases from zero to 90° . Available observations do not permit specifying the angle θ , where transition from one type to the other occurs. Nevertheless, ground settlement due to offset arrangement tunnelling is influenced by the construction sequence. Excavation of the upper tunnel first leads to a higher settlement than that when the lower tunnel is excavated first. When the lower tunnel is driven first, it provides reinforcement within the soil that helps shield the previously strained soil above the new tunnel from further settlement. The position of the combined maximum settlement is eccentrically displaced towards the new upper tunnel; because a region of large strain concentration is believed to occur between the twin tunnels due to excavation of the new second tunnel. The eccentricity tends to be much larger than the values observed for side-by-side tunnels. The eccentricity decreases with increased distance (angular spacing) between the tunnels; with the decrease in the magnitude of maximum settlement being similar to that observed for piggyback tunnels.

The effect of cover depth, pillar width, and excavation sequence on four twin tunnel geometric configurations is summarized in Table 4.

7.2. Practical implications of the review

Tunnelling induced ground settlement has been a topic of research interest for over half a century. Over the years, as twin tunnel excavation became increasingly common, numerous research groups have focused on ground settlements resulting from twin tunnelling. The objective of this paper is to gather and summarize available twin tunnelling induced settlement data. The data is compared, analyzed, and summarized to explore the effect of geometric factors such as pillar width and cover

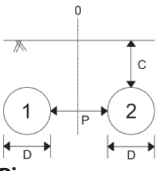
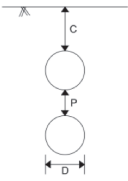
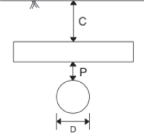
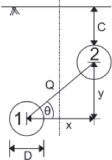
depth as well as construction sequence on ground movements. It is evident that the presented data is helpful to understand settlement trends for four widely used twin tunnel geometric configurations. This paper also explores the effect of geometric factors and construction sequence on twin tunnelling induced settlements using available data.

In summary, it is evident that side-by-side tunnelling typically results in a lower magnitude of soil settlement compared with the other three tunnelling arrangements. However, there is no consensus among studies on which twin tunnel configurations results in the highest soil settlement. Nevertheless, side-by-side tunnels are associated with the largest lateral extension of the settlement (settlement trough). The behavior of the side-by-side tunnels compared to the other types is believed to result from their mobilizing completely different strain field regimes within the soil above. Both tunnels in side-by-side configurations have similar cover depth therefore they induce similar magnitudes of strains which is not the case for other three configurations. In addition, the upper soils have not been previously strained and are therefore stronger than soils that may have undergone tunneling operations, which may be the case in other configurations. In particular, natural sedimentary clays are generally subjected to the soil structure developed during depositional and post-depositional processes. Tunnelling disturbance will result in the decrease of the volume of surrounding soils. It appeared that the pillar width is the most important factor affecting tunnel interaction in side by side tunnelling. A larger tunnel pillar width can help reduce the twin tunnel interaction but the eventual decision of increasing the spacing, selecting a geometry, and/or construction sequence typically depends on the project conditions, soil type, and underground space use characteristics of the particular construction site. In general, little interaction can be expected once the pillar width exceeds three diameters.

Construction of the upper tunnel at first leads to higher settlement regardless of the geometric arrangement, compared to that when the lower tunnel is constructed first. The reason is the upper tunnel induces relatively larger strains in the soil than the lower tunnel. Therefore, it is best to construct the upper tunnel first while planning the construction sequence of piggyback, perpendicular crossing and offset arrangement twin tunnels. Nevertheless, construction sequence remains a matter of discussion during the planning phase of twin tunnel construction

Table 4

Summary of effect of various geometric parameters on ground displacement.

Geometric Arrangement	Parameters that can affect ground displacement		
	Effect of Cover Depth	Effect of Pillar Distance	Effect of Excavation Sequence
Side by Side Tunnels 	Ground settlement decreases if cover depth increases. The larger the cover depth, the wider the settlement trough and the more the shifts towards the existing tunnel.	Settlement increases when pillar width decrease. The larger the pillar width the more the maximum settlement shifts towards the existing tunnel.	The surface settlement is maximum when two tunnels are simultaneously excavated. Overall, various excavation sequences have minor effects on surface settlement.
Piggy Back Tunnels 	Ground settlement is smaller when the cover depth is higher.	The closer the pillar spacing the flatter the settlement profile becomes above the tunnel centerline.	New tunnel excavated above the existing often causes upheaval. When the new tunnel is constructed below the existing one interaction always occurs and the existing tunnel settles. The construction of the upper tunnel at first leads to higher settlement.
Perpendicularly Crossing Tunnels 	Settlement of the existing tunnel due to the new tunnel excavated beneath decreases as the cover depth of the existing and new tunnels increases. Heave of the existing tunnel increases with increasing C/D when the new tunnel is excavated above.	Ground settlement is larger when P/D is smaller.	The existing tunnel is vertically compressed when the new tunnel excavated underneath, but vertically elongated when the new tunnel advances above. Vertical displacement of the existing tunnel caused by the new tunnel excavation beneath is much larger than when the new tunnel advanced above.
Offset Arrangement Tunnels 	Insufficient information is available to infer conclusions.	The larger the angular spacing the greater the vertical settlement (in Sand) and the lower the settlement (in Clay).	The construction of the upper tunnel at first leads to higher soil settlement than that induced when the lower tunnel is first constructed.

projects since it tends to be strongly influenced by the particulars of the site and project.

Presently, twin tunnel behavior and induced ground settlements are well understood for side-by-side twin tunnels and to some extent for piggyback tunnels. In particular, several methods have been proposed to estimate settlements induced by the second tunnel excavated during side-by-side twin tunnelling. However, further attention is needed to properly understand the behavior and to develop techniques for estimating new second tunnel induced settlement for piggyback, offset arrangement, and perpendicularly crossing tunnels.

Further numerical and/or experimental investigations of the effect of

varying the time between staggered twin tunnel excavation, depth, pillar distance, and different soil types are required to fully understand the settlement behavior occurring during twin-tunnel excavation of various geometric arrangements. Analysis of field data is also required to validate the numerical and experimental data.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Appendix A. Sources of Twin-tunnel Data Used in Figures and Associated Information

ID No.	Fig. No.	Reference	Geometry	Soil Type	Data Type	Geometric Parameters			S_{max}
						C/D	P/D*	D	
1.	4	Addenbrooke and Potts (1996)	Side-by-Side	Clay	FEA	7.7	1.89	4.146 m	Not Available
2.	4	Addenbrooke and Potts (1996)	Side-by-Side	Clay	FEA	7.7	6.72	4.146 m	Not Available
3.	4	O'Reilly and New (1982)	Side-by-Side	Clay	Case Study	2.57	1.31	6.54 m	39.28 mm
4.	4	O'Reilly and New (1982)	Side-by-Side	Clay	Case Study	2.71	1.15	6.54 m	37.13 mm
5.	4	O'Reilly and New (1982)	Side-by-Side	Clay	Case Study	2.65	1.62	6.54 m	20.1 mm
6.	4	Do et al. (2014)	Side-by-Side	Clay	FEA	1	1.25	9.4 m	20.45 mm

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ID No.	Fig. No.	Reference	Geometry	Soil Type	Data Type	Geometric Parameters			S _{max}
						C/D	P/D*	D	
7.	4	Divall and Goodey (2015)	Side-by-Side	Clay	Centrifuge, 100g	2	2	40 mm	11.57 mm
8.	4	Chakeri et al. (2015)	Side-by-Side	Sand	Case Study & FEA	2.63	1.12	6.3 m	41 mm
9.	10	Addenbrooke and Potts (2001)	Side-by-Side	Clay	FEA	2	0.5	4.146 m	4.75 mm
10.	10	Addenbrooke and Potts (2001)	Side-by-Side	Clay	FEA	2	2	4.146 m	4.25 mm
11.	10	Addenbrooke and Potts (2001)	Side-by-Side	Clay	FEA	2	3.5	4.146 m	4 mm
12.	10	Divall (2013)	Side-by-Side	Clay	Centrifuge, 100g	2	0.5	40 mm	0.441 mm
13.	10	Divall (2013)	Side-by-Side	Clay	Centrifuge, 100g	2	2	40 mm	0.479 mm
14.	10	Divall (2013)	Side-by-Side	Clay	Centrifuge, 100g	2	3.5	40 mm	0.489 mm
15.	11	Divall et al. (2012)	Side-by-Side	Clay	Centrifuge, 100g	2	0.5	40 mm	0.52 mm
16.	11	Divall et al. (2012)	Side-by-Side	Clay	Centrifuge, 100g	2	2	40 mm	0.326 mm
17.	11	Divall et al. (2012)	Side-by-Side	Clay	Centrifuge, 100g	2	3.5	40 mm	0.255 mm
18.	11	Addenbrooke and Potts (2001)	Side-by-Side	Clay	FEA	2	2	4.146 m	0.297 mm
19.	12	Divall (2013)	Side-by-Side	Clay	Centrifuge, 100g	2	0.5	40 mm	0.486 mm
20.	12	Divall (2013)	Side-by-Side	Clay	Centrifuge, 100g	2	0.5	40 mm	0.485 mm
21.	12	Divall (2013)	Side-by-Side	Clay	Centrifuge, 100g	2	0.5	40 mm	0.543 mm
22.	12	Chapman et al. (2006)	Side-by-Side	Clay	1g Test	3.8	1.5	0.08 m	1.375 mm
23.	12	Chapman et al. (2006)	Side-by-Side	Clay	1g Test	3.8	1.5	0.08 m	1.4625 mm
24.	12	Chapman et al. (2006)	Side-by-Side	Clay	1g Test	3.8	1.5	0.08 m	1.5625 mm
25.	12	Chapman et al. (2006)	Side-by-Side	Clay	1g Test	3.8	1.5	0.08 m	2.125 mm
26.	13	Divall (2013)	Side-by-Side	Clay	Centrifuge, 100g	1.5	0.5	40 mm	0.979 mm
27.	13	Divall (2013)	Side-by-Side	Clay	Centrifuge, 100g	1.5	2	40 mm	0.852 mm
28.	13	Divall (2013)	Side-by-Side	Clay	Centrifuge, 100g	1.5	3.5	40 mm	1.263 mm
29.	14	Divall (2013)	Side-by-Side	Clay	Centrifuge, 100g	1.5	0.5	40 mm	0.543 mm
30.	14	Divall (2013)	Side-by-Side	Clay	Centrifuge, 100g	1.5	2	40 mm	0.427 mm
31.	14	Divall (2013)	Side-by-Side	Clay	Centrifuge, 100g	1.5	3.5	40 mm	0.637 mm
32.	15	Do et al. (2014)	Piggyback	Clay	FEA	1.63	0.25	9.4 m	22.09 mm
33.	15	Do et al. (2014)	Piggyback	Clay	FEA	1.63	0.25	9.4 m	16.92 mm
34.	15	Do et al. (2014)	Piggyback	Clay	FEA	1.63	0.25	9.4 m	35.25 mm
35.	15	Do et al. (2014)	Piggyback	Clay	FEA	1.63	0.25	9.4 m	28.2 mm
36.	15	Do et al. (2014)	Piggyback	Clay	FEA	1.63	0.25	9.4 m	31.96 mm
37.	16	Hunt (2005)	Piggyback	Clay	FEA	1.82	0.89	9 m	3.3 mm
38.	16	Hunt (2005)	Piggyback	Clay	FEA	1.82	0.89	9 m	4.6 mm
39.	16	Hunt (2005)	Piggyback	Clay	FEA	4.1	3.25	4 m	2 mm
40.	16	Hunt (2005)	Piggyback	Clay	FEA	4.1	3.25	4 m	1 mm
41.	17	Addenbrooke and Potts (1996)	Piggyback	Clay	FEA	5.29	1.41	4.146 m	Not Available
42.	17	Addenbrooke and Potts (1996)	Piggyback	Clay	FEA	4.32	2.38	4.146 m	Not Available
43.	17	Addenbrooke and Potts (1996)	Piggyback	Clay	FEA	3.36	3.34	4.146 m	Not Available
44.	18	Addenbrooke and Potts (1996)	Piggyback	Clay	FEA	4.32	2.38	4.146 m	Not Available
45.	18	Hunt (2005)	Piggyback	Clay	FEA	1.82	0.89	9 m	3.3 mm
46.	18	Hunt (2005)	Piggyback	Clay	FEA	4.1	3.25	4 m	2 mm
47.	19	Li et al. (2014)	Perpendicularly Crossing	Clay	Centrifuge, 100g	2.23	0.81	14.5 m	5.38 mm
48.	19	Ng et al. (2015)	Perpendicularly Crossing	Sand	Centrifuge, 60g	2	2	100 mm	17.4 mm
49.	19	Ng et al. (2015)	Perpendicularly Crossing	Sand	FEA	2	2	100 mm	14.8 mm
50.	19	Chakeri et al. (2011)	Perpendicularly Crossing	Clay	FEA	0.81	0.74	9.4 m	30 mm
51.	20	Boonyarak and Ng (2014)	Perpendicularly Crossing	Sand	Centrifuge, 60g	3.5	0.5	6 m	12.75 mm
52.	20	Boonyarak and Ng (2014)	Perpendicularly Crossing	Sand	Centrifuge, 60g	2	0.5	6 m	18 mm
53.	20	Boonyarak and Ng (2014)	Perpendicularly Crossing	Sand	Centrifuge, 60g	5	0.5	6 m	18 mm
54.	20	Boonyarak and Ng (2014)	Perpendicularly Crossing	Sand	Centrifuge, 60g	3.5	0.5	6 m	1.5 mm
55.	20	Boonyarak and Ng (2015)	Perpendicularly Crossing	Sand	Centrifuge, 60g	5	2	6 m	2.5 mm
56.	21	Ng et al. (2015)	Perpendicularly Crossing	Sand	Centrifuge, 60g	2	0.5	100 mm	17.3 mm
57.	21	Ng et al. (2015)	Perpendicularly Crossing	Sand	Centrifuge, 60g	2	2	100 mm	12.1 mm
58.	21	Ng et al. (2015)	Perpendicularly Crossing	Sand	FEA	2	0.5	100 mm	14.8 mm
59.	21	Ng et al. (2015)	Perpendicularly Crossing	Sand	FEA	2	2	100 mm	10.7 mm
60.	22	Divall (2013)	Offset Arrangement	Clay	Centrifuge, 100g	2	2.12	40 mm	0.304 mm
61.	22	Divall (2013)	Offset Arrangement	Clay	Centrifuge, 100g	2	2.7	40 mm	0.316 mm
62.	22	Chehade and Shahrour (2008)	Offset Arrangement	Sand	FEA	2	3.2	2.5 m	51 mm
63.	22	Chehade and Shahrour (2008)	Offset Arrangement	Sand	FEA	2	2.82	2.5 m	46 mm
64.	22	Chen et al. (2012)	Offset Arrangement	Sand	Case Study & FEA	1.86	1.89	6.25 m	32 mm
65.	23	Shirlaw et al. (1988)	Offset Arrangement	Sand	Case Study	2	2.04	6 m	8.9 mm
66.	23	Shirlaw et al. (1988)	Offset Arrangement	Sand	Case Study	2	2.04	6 m	18.1 mm
67.	23	Divall (2013)	Offset Arrangement	Clay	Centrifuge, 100g	2	2.12	40 mm	0.188 mm
68.	23	Divall (2013)	Offset Arrangement	Clay	Centrifuge, 100g	2	2.12	40 mm	0.304 mm
69.	23	Du and Huang (2009)	Offset Arrangement	Weak Rock	FEA	1.36	1.48	156 mm	35.9 mm
70.	23	Du and Huang (2009)	Offset Arrangement	Weak Rock	FEA	1.36	1.48	156 mm	44.4 mm
71.	24	Hunt (2005)	Offset Arrangement	Clay	FEA	1.93	2.68	9 m	11.9 mm
72.	24	Hunt (2005)	Offset Arrangement	Clay	FEA	1.93	3.66	9 m	10.8 mm
73.	24	Hunt (2005)	Offset Arrangement	Clay	FEA	1.93	5.75	9 m	10.7 mm
74.	24	Hunt (2005)	Offset Arrangement	Clay	FEA	1.93	2.68	9 m	13.9 mm
75.	24	Hunt (2005)	Offset Arrangement	Clay	FEA	1.93	3.66	9 m	12.8 mm
76.	24	Hunt (2005)	Offset Arrangement	Clay	FEA	1.93	5.75	9 m	12.1 mm

Appendix B. Supplemental Source of Information Related to the Study of Twin Tunnelling

No.	Citation	Type	Geometry	Soil
1	Addenbrooke (1996)	FEA	Side-by-side, Piggyback	Clay
2	Addenbrooke and Potts (1996)	FEA	Side-by-Side, Piggyback	Clay
3	Addenbrooke et al. (1997)	FEA	Offset Arrangement	Clay
4	Addenbrooke and Potts (2001)	FEA	Side-by-Side	Clay
5	Affipour et al. (2011)	FEA	Side-by-Side	Silty sand
6	Akins and Abramson (1983)	Case Study	Side-by-Side	Rock
7	Barla and Ottoviani (1974)	Case Study	Side-by-Side	Sand
8	Bilotta and Russo (2012)	Case Study	Side-by-Side	Silty Sand
9	Boonyarak and Ng (2014)	Centrifuge, 60g	Perpendicularly Crossing	Sand
10	Boonyarak and Ng (2015)	FEA	Perpendicularly Crossing	Sand
11	Boonyarak and Ng (2015)	Centrifuge, 60g	Perpendicularly Crossing	Sand
12	Byun et al. (2006)	1g Test	Piggyback	Sand
13	Chakeri et al. (2011)	FEA	Perpendicularly Crossing	Clay
14	Chakeri et al. (2015)	Case Study & FEA	Side-by-Side	Sand
15	Channabasavaraj and Visvanath (2013)	FEA	Side-by-Side, Piggyback, Offset Arrangement	Sand
16	Chapman et al. (2002)	1g Test	Side-by-Side	Clay
17	Chapman et al. (2006)	1g Test	Side-by-Side	Clay
18	Chapman et al. (2007)	1g Test	Side-by-Side	Clay
19	Chehade and Shahrour (2008)	FEA	Side-by-Side, Piggyback, Offset Arrangement	Sand
20	Chen et al. (2006)	Case Study	Piggyback	Sand
21	Chen et al. (2011)	Case Study	Side-by-Side	Silt
22	Chen et al. (2012)	Case Study & FEA	Offset Arrangement	Sand
23	Choi and Lee (2010)	1g Test	Side-by-Side Offset Arrangement	Mixed
24	Cooper and Chapman (1998)	Case Study	Perpendicularly crossing	Clay
25	Cooper et al. (2000)	Case Study	Piggyback	Clay
26	Cooper et al. (2002a)	Case Study	Perpendicularly crossing	Clay
27	Cooper et al. (2002b)	Case Study	Perpendicularly crossing	Clay
28	Divall et al. (2012)	Centrifuge, 100g	Side-by-Side, Offset Arrangement	Clay
29	Divall (2013)	Centrifuge, 100g	Side-by-Side, Offset Arrangement	Clay
30	Divall and Goodey (2015)	Centrifuge, 100g	Side-by-Side, Offset Arrangement	Clay
31	Do et al. (2014a)	FEA	Side-by-Side	Clay
32	Do et al. (2014b)	FEA	Side-by-Side	Clay
33	Do et al. (2014b)	FEA	Piggyback	Clay
34	Do et al. (2015)	FEA	Side-by-Side	Clay
35	Du and Huang (2009)	FEA & 1g Test	Offset Arrangement	Weak Rock
36	Ercelebi et al. (2011)	FEA & Case Study	Side-by-Side	Clay
37	Fang et al. (2015)	Case Study	Side-by-Side, Perpendicularly Crossing	Mixed
38	Fang et al. (2016)	Case Study	Piggyback, Offset Arrangement	Mixed
39	Fujita (1994)	Theoretical	Side-by-Side	Clay
40	Ghaboussi and Ranken (1977)	FEA	Side-by-Side	
41	Hasanpour et al. (2012)	Analytical, FEA & Case Study	Side-by-Side	Mixed
42	He et al. (2012)	Case Study & 1g Test	Side-by-Side	Sandy Cobble
43	Hefny et al. (2004)	FEA	Side-by-Side	Clay
44	Hsiung (2011)	Case Study	Side-by-Side	Sand
45	Hunt (2005)	FEA	Side-by-Side, Piggyback, Offset Arrangement	Clay
46	Karakus et al., 2007	FEA	Side-by-Side	Clay
47	Kim et al. (1996)	1g Test	Side-by-Side	Clay
48	Kim et al. (1998)	1g Test	Side-by-Side, Perpendicularly Crossing	Clay
49	Koungelis and Augarde (2004)	FEA	Side-by-Side, Piggyback	Clay
50	Kuesel (1972)	Case Study	Side-by-Side, Piggyback	Mixed
51	Li and Yuan (2012)	Case Study	Perpendicularly Crossing	Mixed
52	Li et al. (2014)	Centrifuge, 100g	Perpendicularly Crossing	Clay
53	Liu et al. (2009)	FEA	Perpendicularly Crossing	Mixed
54	Mirhabibi and Soroush (2012)	FEA & Case Study	Side-by-Side	Clay
55	Mooney et al. (2014)	Case Study	Side-by-Side, Offset Arrangement	Mixed
56	Ng et al. (2004)	FEA	Side-by-Side	Clay
57	Ng et al. (2013)	FEA & Centrifuge, 60g	Perpendicularly Crossing	Sand
58	Ng et al. (2015)	FEA & Centrifuge, 60g	Perpendicularly Crossing	Sand
59	Nyren (1998)	Case Study	Side-by-Side, Offset Arrangement	Clay
60	Ocak (2013)	Case Study	Side-by-Side	Clay
61	Ocak (2014)	Case Study	Side-by-Side	Clay
62	O'Reilly and New (1982)	Case Study	Side-by-Side	Clay
63	Peck (1969)	Theoretical	Side-by-Side	Clay
64	Sagaseta et al. (1999)	Case Study	Side-by-Side	Clay
65	Saitoh et al. (1994)	Case Study	Perpendicularly Crossing	Sand
66	Shahin et al. (2013)	1g Test & FEA	Side-by-Side, Piggyback	Clay
67	Shirlaw et al. (1988)	Case Study	Offset Arrangement	Sand
68	Soliman et al. (1993)	FEA	Side-by-side	Not Available
69	Suwansawat (2006)	Analytical & Case Study	Side-by-Side, Piggyback	Clay
70	Suwansawat and Einstein (2007)	Analytical & Case Study	Side-by-Side, Piggyback	Clay
71	Tjie-Liong (2005)	FEA & Case Study	Side-by-Side	Sand
72	Wan et al. (2017)	Case Study	Perpendicularly Crossing	Clay
73	Wang et al. (2003)	FEA	Side-by-side	Clay
74	Yamaguchi et al. (1998)	Case Study	Offset Arrangement, Perpendicularly Crossing	Sand

(continued on next page)

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No.	Citation	Type	Geometry	Soil
75	Li et al. (2011)	FEA & Case Study	Side-by-side	Sand
76	Yang and Wang (2011)	Analytical	Side-by-side	Not Available
77	Zhang et al. (2013)	Analytical	Side-by-Side	Clay
78	Zhang and Huang (2014)	FEA	Side-by-side	Clay

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