Improving the engineering knowledge from Crossrail data: The effect of tunnelling on piled buildings

by

William Cairns (T)

Fourth-year undergraduate project in
Group D, 2014/15

I hereby declare that, except where specifically indicated, the work submitted herein is my original work.

Signed: ______________________________________ Date:________________

Technical Abstract

The research undertaken in this project aimed to improve the knowledge of the effects of tunnel construction beneath piled buildings. Historically, tunnelling beneath piled buildings has not been well practiced and therefore there is a limited about of research into the subject. This has led to very over conservative methods being used on recent tunnel constructions in cities such as London and Hong Kong when assessing the building damage, and expensive protective measures that could have been avoided with better knowledge of the pile - tunnel interaction.

In order to assess the behaviour of piled buildings when subject to tunnelling, data was analysed from three piled buildings located above the new Crossrail tunnels in Farringdon, London. A numerical *t-z* model using load displacement curves was then used to model the building behaviour to see how closely the true behaviour could be predicted, and to better understand the factors that influenced any discrepancy in predicted and actual behaviour.

Greenfield analysis was carried out at the Farringdon site and it was found that the greenfield transverse settlements closely fit a Gaussian trough and the longitudinal settlements matched the shape of a cumulative probability curve, both as expected. Volume losses were calculated of 0.9% for the Eastbound tunnel and 1.0% for the Westbound tunnel (for the combined pilot and enlargement tunnel stages), both well below the conservative Crossrail estimate of 1.25%. The building displacements were observed to generally be of similar magnitude to the greenfield settlements away from the tunnel centreline, and of slightly greater magnitude close to the tunnel centreline, qualitatively agreeing with predictions from numerical models and centrifuge tests, although the magnitude of these displacements was lower than expected. The buildings with a ground-bearing slab behaved with a degree of rigidity in both the longitudinal and transverse directions, suggesting pile group effects in conjunction with high slab stiffness were responsible for controlling the behaviour.

The *t-z* model was found to generally over predict the pile settlement, particularly for the piles close to the tunnel centreline, implying the mechanism for single piles is different to that of pile groups and that a more comprehensive analysis must be undertaken when considering piled buildings. A number of factors were considered to explain this difference in behaviour. Tensile cracking in piles close to the centreline was observed to have a small influence in reducing the discrepancy between the predicted

and actual behaviour, as was the pile factor of safety, which had a minor impact. However, the predominant reason for the difference in behaviour was thought to be the effects caused by the presence of a ground bearing slab into which the pile were cast. The slab provided extra bearing capacity, which limited the settlement and it also allowed for the redistribution of pile loads away from the tunnel centreline due to its high stiffness and rigidity, thus reducing the settlement of the piles close to the centreline and explaining the observed behaviour.

Despite a slight discrepancy, the *t-z* model has been proven to provide a reasonable estimate of the pile displacement and since it is conservative it would be suitable for use in carrying out initial building damage assessments due to its computational speed and simplicity. There is, however, scope for further development of the model to incorporate ground beam effects and also to model the pile-soil interaction as non-linear, in order to more accurately model the building response.

The interaction between tunnelling and pile groups is complex and is influenced by a number of factors. This study agreed with previous observations in that pile groups appear to have beneficial effects in reducing the overall settlement compared to single piles, and discovered that the primary reason for this was the presence of a ground bearing slab or ground beam that connects the piles, resulting in a reasonably rigid response and an overall reduction in displacement.

Contents

Chapter 1 Introduction

This project aims to better understand the effects of tunnel construction beneath piled structures. Tunnelling beneath piles is a relatively under-researched area, mainly because traditionally there has not been a need to tunnel beneath piles, where space has not been limited and underground construction less of a necessity. However, with increasing urbanisation a demand for space has led to ever increasingly complex underground infrastructure, including foundations, metros, and sewage and water works, among others.

The construction of tunnels induces ground movements, which get transferred to overlying structures and can potentially lead to damage. These movements need to be predicted by Engineers in order to ensure the safety of the structures, and implement protective measures if necessary. The interaction between tunnelling and buildings founded on shallow footings is relatively well researched and understood, and well-established methods are in place for predicting the extent of the building damage. However, the interaction between tunnels and piled (deep) foundations is less well understood and is more complex for a number of reasons, such as varying layers of soil with differing properties, and a varying soil settlement profile with depth. This uncertainty in behaviour has resulted in very conservative approaches to predicting building damage with high factors of safety being adopted, often leading to expensive building protection measures which may have been unnecessary and could have been avoided with better understanding.

This project aims to analyse the effect of tunnelling beneath piled buildings, by carrying out a case study at Farringdon in London using data from the Crossrail project. Three piled buildings were chosen for this study, all in close proximity to the new Crossrail platform tunnels. A parametric study was then undertaken using a numerical *t-z* model, with the aim to model the key aspects as closely as possible, and better understand the contributory factors to any differences in predicted and actual behaviour.

Chapter 2 Literature review

2.1 Tunnelling induced ground movements

Tunnelling through any type of ground causes a change to the in-situ stress, resulting in a movement of soil towards the tunnel. This is observed as displacements at the surface giving a settlement profile similar to that shown in Figure 2.1.

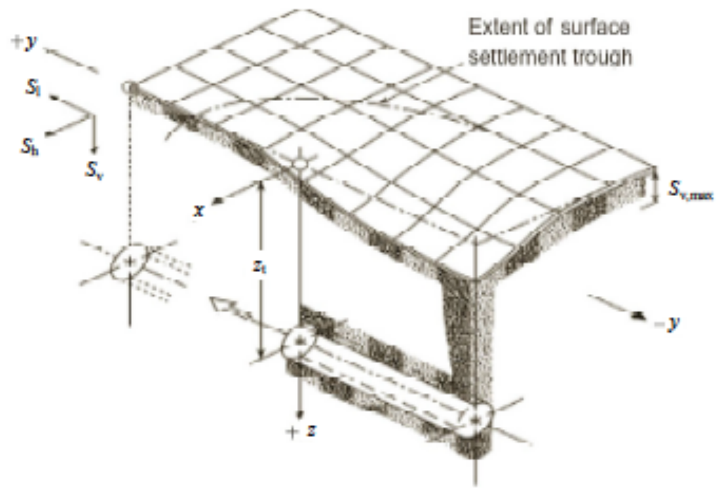


Figure 2.1: Assumed surface settlement profile, after Attewell et al. (1986)

Greenfield settlements are the observed ground settlements that are not influenced by surrounding structures or foundations, and these are commonly used in design. The constant volume condition in clays forms the basis for analysis; the behaviour in sands is more complex due to the inaccuracy of this assumption, however in practice, this is commonly used. This project is, however, focused on tunnelling in clay.

2.1.1 Transverse movements

The transverse surface settlement cross section far away from the excavation face is well described by an inverse Gaussian curve, first shown by Martos (1958) and subsequently shown by Peck (1969), using the following equation,

$$S_v(x) = S_{v,max} \, . \exp\left(-\frac{x^2}{2i^2}\right) \qquad (2.1)$$

where S_v is the settlement at a transverse distance x from the tunnel centre-line, $S_{v,max}$ is the settlement above the tunnel centre-line, and i is the is the transverse distance between the point of inflection and the tunnel centre-line.

The parameter i is used to determine the settlement trough width. O'Reilly and New (1982) proposed Equation 2.2 below to describe the relationship between the depth of the tunnel and i:

$$i = K\ (z_t - z) \tag{2.2}$$

where z_t is the depth of the tunnel and z is the depth of interest. A value of $K = 0.5$ is often used in clays, shown by Mair and Taylor (1997) based on case study data. The value of K in sand typically varies from 0.25 to 0.35 (Mair and Taylor, 1997), thus, tunnelling in clay results in a wider and shallower settlement trough.

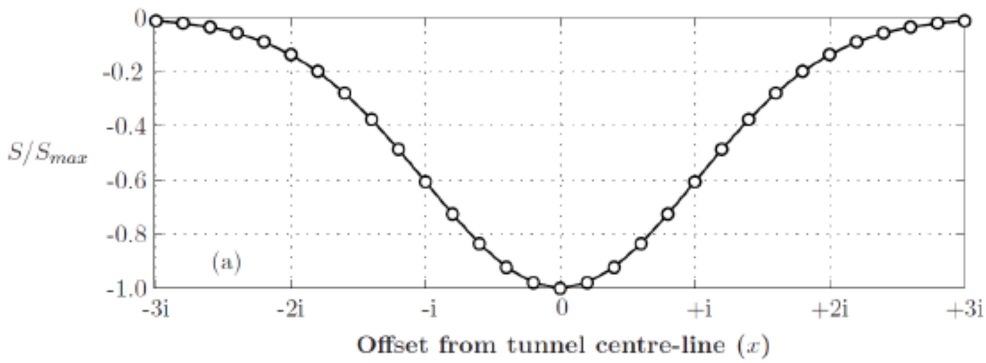


Figure 2.2: Gaussian settlement trough (Selemetas, 2005)

It is known that subsurface settlements exhibit different characteristics to the surface settlements. Mair et al. (1993) highlighted that using $K = 0.5$ at all depths underestimates the width of the settlement profile close to the tunnel crown and therefore overestimates the settlement above the tunnel centre-line. A different trough width parameter was proposed by O'Reilly and New (1982), and Mair (1993) subsequently proposed a modification to the value of K:

$$K = \frac{0.175 + 0.325(1 - z/z_t)}{(1 - z/z_t)} \tag{2.3}$$

Care must be taken when using this equation since it was shown to break down within ~0.5D of the tunnel crown, shown by Grant and Taylor (2000). However, for deep tunnels this is not considered a significant issue.

2.1.2 Longitudinal movements

Attewell and Woodman (1982) showed the longitudinal settlement profile can be modelled as a cumulative probability curve, where the settlement directly above the excavation face is 0.5 $S_{v,max}$. Depending on the tunnelling method used, settlement ahead of the face may heave upwards and not settle. This is the case for EPB machines where the pressures ahead of the tunnel excavation are greater than the in-situ earth pressure, as described by Konda (1987). The effect of face support measures can also change the profile, as shown in Figure 2.3, where the profile is shifted away from the tunnel face leading to settlements above the excavation face of 0.2-0.3 $S_{v,max}$.

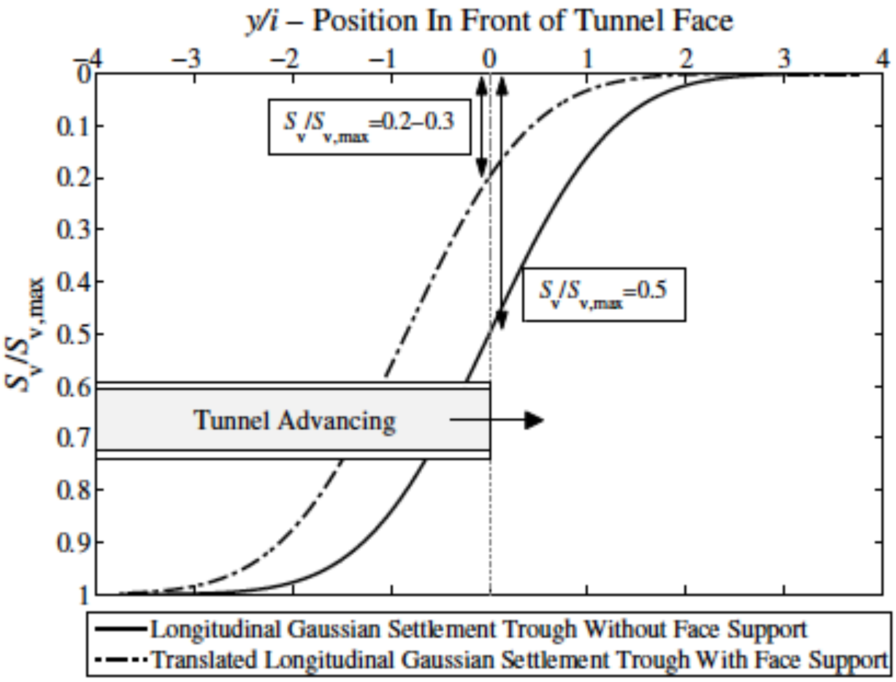


Figure 2.3: Longitudinal Settlement Trough (Williamson, 2014, after Attewell and Woodman, 1982)

2.2 Settlement of piled foundations

Piled foundations are a type of deep foundation which are used to transfer high loads through weak strata into deeper layers, preventing excessive settlements due to a greater bearing capacity.

The complex interaction between piles and soil requires simplifying assumptions to be made when predicting pile settlement. There are various methods used for predicting pile settlement in practice, two of which are outlined briefly below.

2.2.1 Boundary Element Methods

In the Boundary Element Methods (BEM), each pile element and the pile-soil interface are discretised. The Mindlin (1936) solution is applied to model the soil as an elastic continuum. Poulos and Davis (1968) used this approach to study incompressible piles, which provided a load settlement curve that was relatively good. This method was further developed, and then again modified by Poulos and Davis (1980) to allow for non-linear soil models. Chow (1986) provided a solution for including pile compressibility by expressing pile stiffness $[K_{pile}]$ and soil stiffness $[K_{soil}]$ as two separate matrices, and solving the equation:

$$([K_{pile}] + [K_{soil}])\{w_p\} = \{Q\} \tag{2.4}$$

where w_p is the pile displacement and $\{Q\}$ is the vector of input forces. This approach, however, was found to generally over-predict the settlements.

2.2.2 Load transfer *t-z* models

In *t-z* analyses, all influence factors are taken to be zero except at the location of the element being analysed. A method proposed by Coyle and Reese (1966) uses an arbitrary *t-z* curve, where a displacement is applied the base and the resulting load calculated. Via compatibility and equilibrium the load at each subsequent node is calculated to then derive a load at the pile head. By repeating this for a number of different displacements, the load displacement curve can be calculated. A great advantage of this method is that it is possible to model different soil properties such as variations in soil stiffness and strength with depth, since the selection of the load-transfer curve is empirical. Kraft Jr et al. (1981) proposed the use of Winkler springs, and Chow (1986) subsequently considered the analysis of pile groups, based on the work of Kraft Jr. et al. (1981) and Mindlin's (1936) solutions.

2.3 Effects of tunnelling on piled foundations

When considering the effects of tunnelling on piled foundations, the research can be split into two distinct cases: tunnelling adjacent to piles and tunnelling beneath piles. Historically, the latter has been far less practiced and thus the amount research in this area is comparatively small compared to the research into tunnelling adjacent to tunnels. That said, the amount of research in both areas in constantly growing, and can broadly be split into the following categories:

1. Case studies
2. Field trials
3. Centrifuge studies
4. Numerical modelling
5. Models used in practice

This section focuses on research conducted into the effects of tunnelling beneath existing piles, as this provides the basis for this project. Research into tunnelling effects adjacent to piles is far more extensive (as mentioned above), and as Mair and Williamson (2014) noted, the effects have been found to be relatively minor, with some lateral displacements caused but very little pile settlement.

2.3.1 Case studies

2.3.1.1 Channel Tunnel Rail Link – Ripple Road Flyover

This case study was undertaken by Jacobsz et al. (2005), and looks at the effects of the CTRL beneath the Ripple Road Flyover Bridge, founded on deep piles in London Clay.

8.2m diameter tunnels were constructed beneath the piles, with the closest distance 1m between the pile toe and tunnel crown. Jacobsz et al. (2005) explain that the subsurface movements differ around piles since the piles exhibit large stresses on surrounding soil, affecting the soil stiffness. Jacobsz et al. (2005) also predict that the pile would reduce in base load leading to an increase in shaft friction, and that the pile must settle more than the adjacent soil to maintain this positive friction force. To account for this reduction in base load, a grouted raft was inserted below the pile cap to increase the bearing capacity. It was then expected that the pile cap settlement would equal the greenfield surface settlement, which it did. A similar result was seen for the A406 viaduct.

2.3.1.2 Crossrail running tunnels - Bishops Bridge Road, London

Williamson (2014) investigated the effects of the Crossrail running tunnels at 4-18 Bishops Bridge Road. The tunnels were twin bored 7.1m outer diameter using an EPB TBM. The piles were mostly driven vibro piles connected to pile caps supporting the building columns, and the pile toes were estimated to be within 3m of the tunnel crown.

It was found that the greenfield surface settlement was very similar to the building settlement. Williamson (2014) pointed out that this was unexpected: many of the piles lie directly above the tunnel centre-line, with factors of safety (FoS) between 1.73 and 2.16. Centrifuge tests on piles with similar FoS gave significantly greater displacements than the greenfield surface. Likewise, BEM and *t-z* analyses would also predict similar results for single piles. Williamson (2014) explains that this difference between actual and predicted behaviour could be due to a combination of the effect of the pile cap bearing capacity, a redistribution of load into piles away from the tunnel, tensile cracking of the piles, and an actual higher FoS.

Williamson (2014) observed another case on the Crossrail project at Kempton Court, where the tunnelling was sprayed concrete lining (SCL) 10.7m diameter tunnels. Similarly to Bishops Bridge Road, the building settlement at Kempton Court was observed to be very similar to the greenfield ground surface settlement, with very little difference in the shape, magnitude or width of the settlement trough.

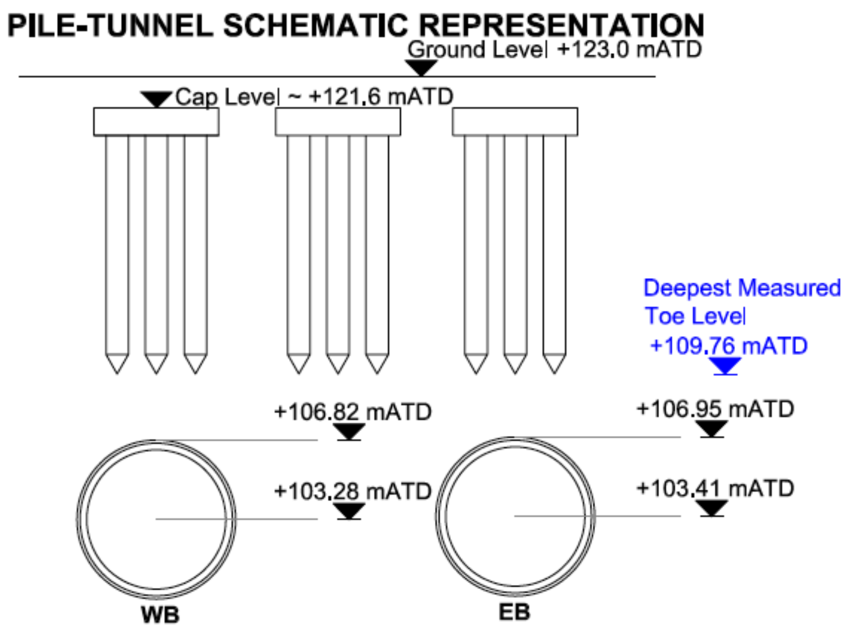


Figure 2.5: Cross section of the Crossrail running tunnels at Bishops Bridge Road, Williamson (2014)

2.3.2 Field Trials

2.3.2.1 Channel Tunnel Rail Link

Selemetas (2005) investigated the effects of tunnelling beneath piles in a field trial at a site above the CTRL. Twin tunnels were constructed using an EPB machine, with outer diameter 8.15m at a depth of 18.9m to tunnel axis. Four piles were constructed, all 480mm diameter, two of which were offset, and two of which were above the tunnel centreline. The results showed that the friction pile above the tunnel centreline settled more than the greenfield ground surface as the tunnel face passed through. The observations made on the piles offset from the tunnel centre-line were that there was virtually no difference between the ground movements and the pile settlement.

2.3.3 Centrifuge testing

A number of centrifuge tests have been carried out, to look at both effects of tunnelling beneath and adjacent to piles. Bezuijen & Van der Schrier (1994) and Hergarden et al. (1996) noted that large settlements occur when the pile toe is within 0.25 to 1.0 D_t (tunnel diameter) of the tunnel, and that piles show very little displacement when they are further than 2.0 D_t from the tunnel, based on a number of centrifuge investigations.

Jacobsz (2002) conducted centrifuge testing on tunnelling beneath piles and pile groups in fine sand. The greatest pile settlements were seen when directly above the tunnel centreline, although the absolute distance to the tunnel was found to be more important than the distance to the centreline. Pile groups were shown to have positive effects in reducing settlement and induced loading.

Williamson (2014) carried out centrifuge tests investigating bored piles in clay. The main conclusions were that a pile located above the tunnel centreline will settle more than the ground surface at the pile head, that the piles show relatively small changes in load, and that pile failure does not occur. Williamson (2014) also concluded that piles show an increase in skin friction when subjected to positive relative displacement and vise versa, and that there is no loss of capacity of the piles.

2.3.4 Numerical models

2.3.4.1 Finite Element Modelling

A number of finite element studies have been undertaken (Mroueh and Shahrour 2002; Lee and Ng 2005, Shahin et al. 2009; Lee 2012; Lee 2013; Yoo & Kim 2008). On the whole, work that compares the effects of single piles with pile groups has generally shown that pile groups have beneficial effects, i.e. the settlement and axial loads are reduced relative to that of single piles. However, Lee (2013) showed that pile groups located entirely within the tunnel zone of influence experience much greater displacement than single piles. The work suggests there is a difference in behaviour between pile groups that extend beyond the tunnel footprint and pile groups that are directly and solely above the tunnel.

2.3.4.2 Two-Step Approach Method

A common model for analysing tunnelling effects on a pile is a two-step approach method (TSAM) whereby the greenfield surface settlements are calculated and imposed onto the existing piles using a boundary element or *t-z* model. Chen et al. (1999), Loganathan et al. (2001) and Xu and Poulos (2001) have all undertaken research into the use of the BEM and generally shown greater pile settlements and induced loads compared to the *t-z* models used by Huang et al. (2009), and Zhang et al. (2011) for single piles. Single piles have also been shown to be a conservative ("worst case") estimate when compared to the behaviour of pile groups. Recent analysis by Basile (2012) took into account non-linearity of the soil, which produced a solution that reduced the loading compared to a linear elastic model, better modelling the behaviour.

2.3.5 Models used in practice

In practice, the tools for predicting pile head settlement are reasonably limited. The most common method used in practice is the "Assumed depth method", where the greenfield settlements at a particular depth (often the pile toe or pile head), is used for the first stage of analysis. This is a conservative approach.

2.4 Discussion

The effects of tunnelling on piled foundations are still relatively unknown, despite growing research. There have been numerous studies looking at the effects on piles in sands, but far less into the effects on piles in stiff clay, and in particular bored piles.

The studies outlined above all show that piles exhibit displacement and changes in axial force and bending moment when subjected to tunnelling effects, however, how much settlement is caused is still not fully known, since the magnitude of these effects depends on a number of factors.

Centrifuge tests and other studies have found that piles that have their toe located below the tunnel axis exhibit smaller settlements than those with their pile toe above the tunnel axis. Likewise, those that are laterally located further away from the tunnel centre line show less settlement than those piles located directly above the tunnel centreline.

The work on pile groups is limited, and the methods are conservative. The true effects of tunnelling beneath pile groups have not been properly modelled; finite element methods are computationally complex and expensive in terms of both cost and time. Simplified numerical modelling has been looked at for single piles, however, some of the boundary conditions have not been well modelled in the past, such as tensile cracking and shallow loading. A model has not been developed to investigate pile group behaviour, and the reduction in tunnelling-induced settlement due to the soil-stiffening effects of pile groups and large structures.

2.5 Aims

This project aims to investigate the behaviour of piled buildings above a tunnel construction to improve the understanding about which factors contribute to the difference in behaviour of single piles and piled buildings. There are three specific objectives:

1. To conduct a case study on three piled buildings at Farringdon, London, using data from Crossrail, to compare the building settlements to greenfield ground settlements.
2. To model the case in a *t-z* model, to investigate how closely the actual building movements can be predicted.
3. To investigate the contributing effects of soil stiffening, concrete cracking and shallow loading (ground beam effects).

Chapter 3 Case Study: Crossrail data at Farringdon, London

3.1 Background

Crossrail is a new, high frequency, high capacity railway for passengers travelling East-West across London. It is currently the biggest civil engineering project in Europe with an estimated cost of around £20bn. Through parts of central London the railway will run underground and thus new tunnels have been constructed. The vast amounts of data available from the project allow for a substantial amount of new research to be undertaken on the effect of tunnelling beneath piled buildings. A case study has been conducted on three piled buildings very close to Farringdon station, where available field data has been used to analyse both the building displacements and the greenfield surface settlements.

The buildings analysed were 78 Cowcross Street, 85 Cowcross Street and The Smokery, Greenhill Rents. These are all located to the east of Farringdon station, shown in Figure 3.1 below.

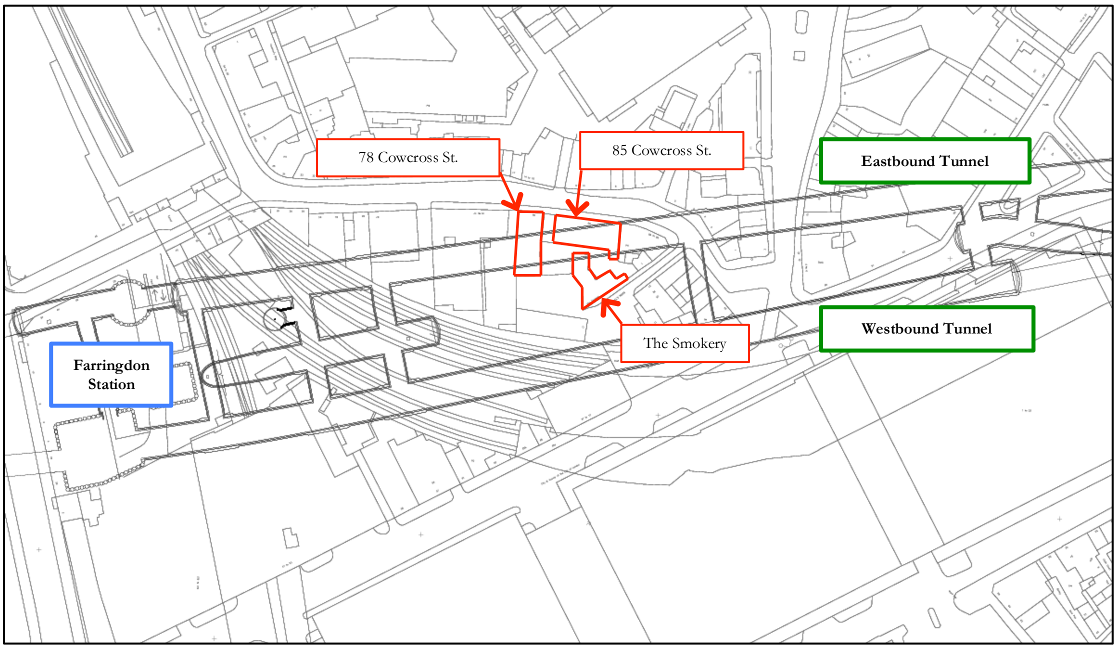


Figure 3.1: Location of piled buildings relative to Farringdon Station and Crossrail tunnels

3.2 Building details

3.2.1 85 Cowcross Street

85 Cowcross Street originally dates from the 1920s, and the four-storey superstructure consists of filler joist floor slabs spanning between an internal steel frame and load-bearing masonry front/rear walls, with a further internal load-bearing masonry wall diving the building into western and eastern halves. In the late 1980s a new lift shaft and staircase was constructed in the centre of the building, with the surrounding floor replaced with a new reinforced concrete slab.

At the ground floor, the western half of the building consists of the original suspended filler joist slab over a single story basement. The eastern half is entirely ground bearing.

3.2.1.1 Substructure

85 Cowcross Street is founded on shallow ground bearing footings with a local area of piling. The original perimeter masonry walls are assumed to be founded on shallow strip footings. A locally piled transfer structure exists beneath the eastern basement, underpinning the rear wall just below ground level. This consists of a steel transfer beam supported on three RC cross beams, spanning between three pairs of 450mm concrete piles. Three piles are cut-off at ground level whilst three piles are cut-off below the basement slab. The deepest pile toe is founded at 92.6 metres above the tunnel datum (mATD), equating to a maximum pile length of 19.4m.

Pile Number	As-built pile toe level (mATD)	Assumed pile cut-off level (mATD)	Pile length (m)	Design load estimates (kN)
P1	101.6	114.0	12.4	450
P2	97.8	114.0	16.2	625
P3	97.4	111.7	14.3	550
P4	99.8	114.0	14.2	550
P5	92.6	112.0	19.4	800
P6	95.9	112.0	16.1	625

Table 3.1: As-built pile toe levels and assumed cut-off levels at 85 Cowcross St

3.2.2 78 Cowcross Street

78 Cowcross street is a five-storey building and similarly to 85, it is steel framed with brick cladding and masonry load-bearing perimeter walls. The two buildings appear to be connected via a small part of the wall at the northeast region of the building, however, the majority of the buildings are detached and therefore it appears a limited amount of stiffness will be transferred between the buildings and thus they can be considered structurally independent.

3.2.2.1 Substructure

78 Cowcross Street has a less complicated substructure than 85. The entire building is founded on a ground-bearing slab into which piles extend a minimum of 600mm. There are 55 bored concrete piles distributed beneath the building. A selection of typical piles is listed in Table 3.2 below. All the piles are 450mm diameter, with 0.26% longitudinal steel reinforcement. The pile design lengths are approximate, based on information from a site investigation. The contractor was responsible for the design of piles based on the working loads and employing a factor of safety of 3.

Pile Number	Pile cut-off level (mATD)	Pile toe level (mATD)	Pile length (m)	Design load (kN)
P1	110.65	93.65	17.0	700
P3	110.65	97.15	13.5	500
P10	110.65	98.65	12.0	400
P22	110.65	98.65	12.0	400
P31	110.65	97.15	13.5	500
P48	110.65	97.15	13.5	500

Table 3.2: Typical pile properties at 78 Cowcross St

3.2.3 The Smokery

The Smokery is a three-storey load-bearing masonry building. As shown in Figure 3.1, it is located between both the Eastbound (EB) and Westbound (WB) Crossrail tunnels, whereas the buildings on Cowcross street lie more directly above the EB tunnel.

3.2.3.1 Substructure

Similarly to 78 Cowcross Street, the Smokery has a ground-bearing slab at the basement level into which the piles extend. There are 30 piles distributed beneath the Smokery. A selection of typical piles is listed in Table 3.3 below. The piles are 450mm diameter, with 0.26% longitudinal steel reinforcement, and designed with a FoS of 3.

Pile Number	Pile cut-off level (mATD)	Pile toe level (mATD)	Pile length (m)	Design load (kN)
P1	110.45	96.25	14.2	400
P5	110.45	93.75	16.7	400
P12	110.45	96.75	13.7	300
P17	110.45	88.55	21.9	725
P20	110.45	95.85	17.6	550
P26	110.45	92.25	18.2	475
P30	110.45	92.85	17.6	480

Table 3.3: Typical pile properties at The Smokery

Figure 3.2: Pile layout at 78 Cowcross, 85 Cowcross and The Smokery

3.3 Tunnel Construction

The tunnels beneath the buildings of interest are Sprayed Concrete Lining (SCL) platform tunnels. The construction of these tunnels occurred in two phases: a pilot tunnel, using an Earth Pressure Balance Tunnel Boring Machine (EPB TBM), where the two tunnels were constructed in sequence, followed by an enlargement using open face excavations and SCL, with both tunnels constructed simultaneously. The WB and EB pilot tunnels passed underneath the buildings between 28/09/13 and 30/09/13, and 12/01/14 and 13/01/14 respectively. The WB SCL enlargement passed the buildings between 21/06/14 and 11/07/14 and the EB SCL enlargement between 07/07/14 and 28/07/14. The tunnel axis depth varies between 28.1 – 28.6m beneath the ground surface, and the predicted volume loss for the platform tunnels was 1.25%.

Tunnel	Diameter, D_t (m)	Axis level (mATD)	Below ground level (m)
EB/WB Pilot	7.180	86.935	27.7 – 28.2
EB/WB SCL	10.195	86.582	28.1 – 28.6

Table 3.4: Tunnel properties

3.4 Instrumentation and monitoring

Figure 3.4 shows the instrumentation layout that was installed around the buildings of interest. Both manual and automatic monitoring were used. The manual monitoring consisted of precise levelling points (PLPs) mounted on the pavements at ground level, and BRE prisms mounted on the buildings close to ground level. Both these types of instrumentation were read approximately on a daily basis when the tunnels were in close proximity. Automatic monitoring consisted of automatically read prisms mounted on the buildings at height and were read remotely with automatic total stations. Readings were taken approximately every few hours. The data from the automatic monitoring broadly disagreed with the BRE data and therefore was not used in the analysis. There was also a nearby inclinometer and extensometer borehole; however, this data was also of poor quality and thus was not used.

3.5 Geology and soil parameters

The Crossrail report, Geotechnical Sectional Interpretative Report 1&2: Royal Oak to Liverpool Street Volumes 1-3 Report No. 1D0101-G0G00-00549, was used to establish the assumed geology and soil profile at the site of interest. The report contains a summary of a number of ground investigations, and extensive details of the geotechnical properties of the different layers of strata, including design parameters. The precise site geology is unknown however a good estimate can be made with interpolation between nearby boreholes. The assumed soil profile is as follows, from ground surface to depth:

1. Made Ground (MG): superficial ground of varying thickness, present at all borehole locations. Mainly consists of engineering fill, construction fill and previous building foundations.

2. River Terrace Deposits (RTD): a series of gravel sheets formed during colder climatic periods. The River Terrace Deposits typically comprise a mix of sand and gravel, and can contain clay and silty sediments. The specific form of RTD at Farringdon is Hackney gravel.

3. London Clay (LC): relatively homogenous in lithology, but has distinct lithological changes between regions in the London area. Therefore, different divisions are used to describe LC based on subtle changes in lithology due to coarseness. Only the lower part of the LC formation is present at Farringdon, and the only division in the relevant boreholes was A3, which is a stiff homogenous clay of thickness roughly 20m.

4. Lambeth Group (LG): comprises of three distinct formations, depending on the different depositional environments by which they were layed down. The only present formation in this location is the Upper Mottled Beds (UMB), which are "stiff to hard fissured and mottled multicolored clays, silty clays and clayey sand".

Other soil types were present, such as other formations of the Lambeth Group (Laminated Beds, Lower Mottled Beds), Upnor Formation and Thanet Sand Formation. However, these were below the depth of both the tunnel and the piles and thus were not of interest.

The assumed section is shown in Figure 3.3, with a typical building cross-section with reference to the EB tunnel.

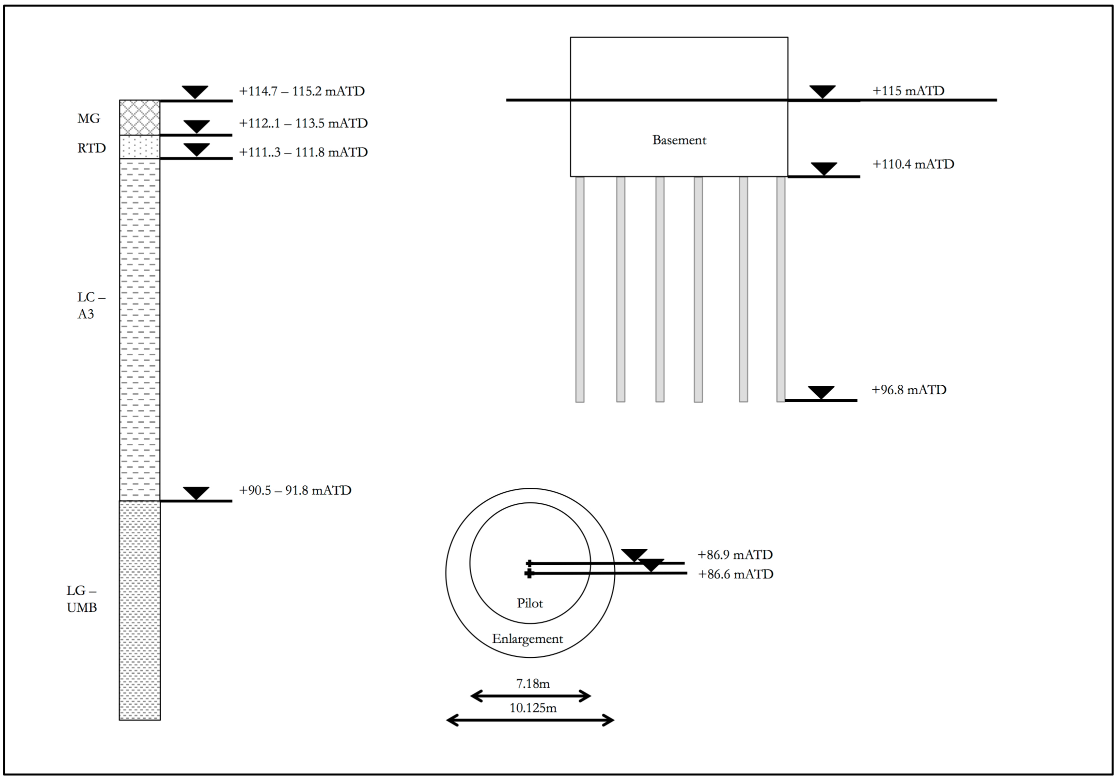


Figure 3.3: Farringdon representative site section

3.6 Greenfield Analysis

Greenfield settlements are the ground settlements that are not influenced by surrounding structures and foundations, and thus can be used to analyse the volume loss of the tunnel. Greenfield surface settlement analysis was carried out for both the WB and EB tunnels, across cross-sections of precise levelling points, shown in Figure 3.4. Transverse and longitudinal vertical settlements were analysed, for both the pilot TBM and the SCL enlargement.

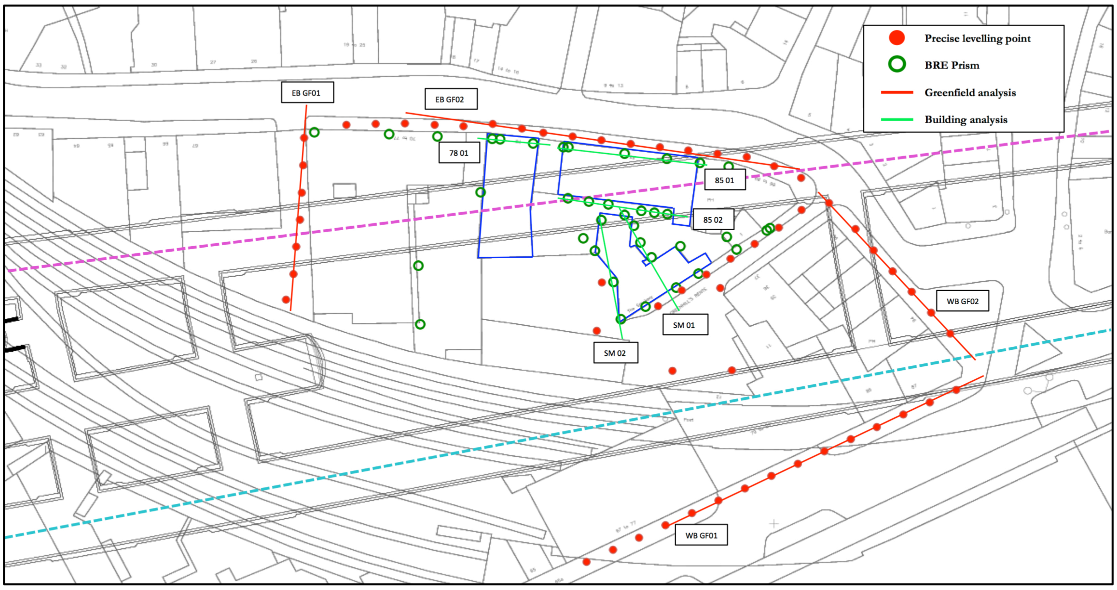


Figure 3.4: Instrumentation layout and cross sections used in both greenfield settlement analysis and building displacement analysis

3.6.1 Effect of compensation grouting

Compensation grouting (CG) is a protective measure technique whereby grout is injected into the ground between the tunnel crown and the building foundations via carefully selected tubes in order to control the building settlement. The layout of the CG tubes are shown in Figure 3.5; as can be seen, although CG does not occur directly beneath the three buildings, cross-sections EB GF01, WB GF01 and WB GF02 are all subject to the effects of CG. This complicated the analysis of the greenfield settlements since the CG beneath these sections had an effect of reversing the settlement, i.e. causing upwards ground movements. In order to take this effect into account, the CG records were correlated with the construction progress records and the monitoring data to determine the times at which CG took place. Any reversals of the settlement on the days in which CG took place were eliminated in order to negate the CG effect, and obtain a better estimate of the greenfield settlement. Cross-section EB GF02 was not subject to CG, and the settlement profile observed was similar to the CG-modified settlement profile of EB GF01, validating this method as a sensible approach.

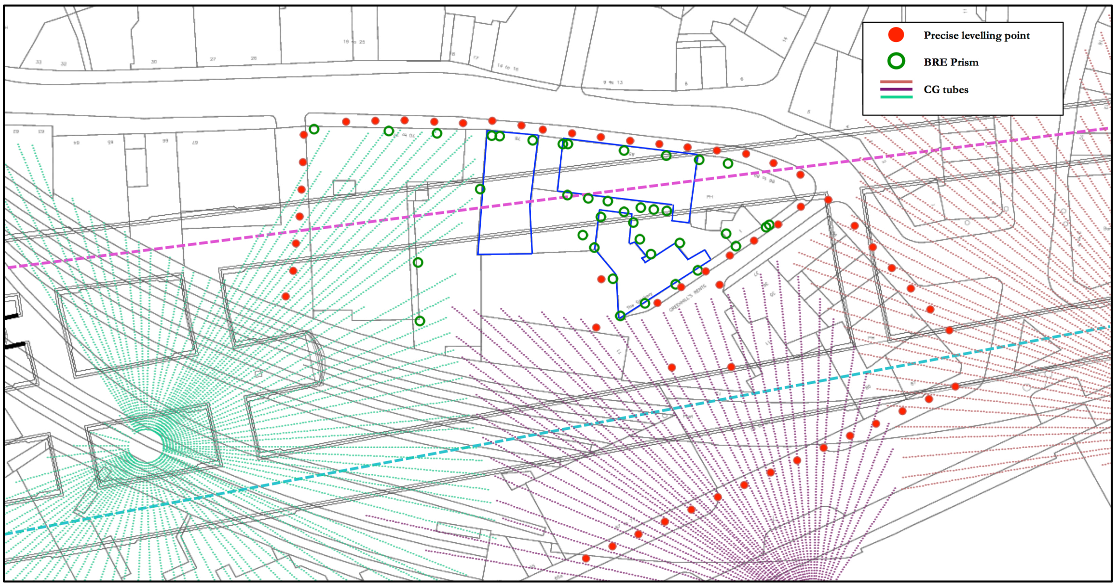


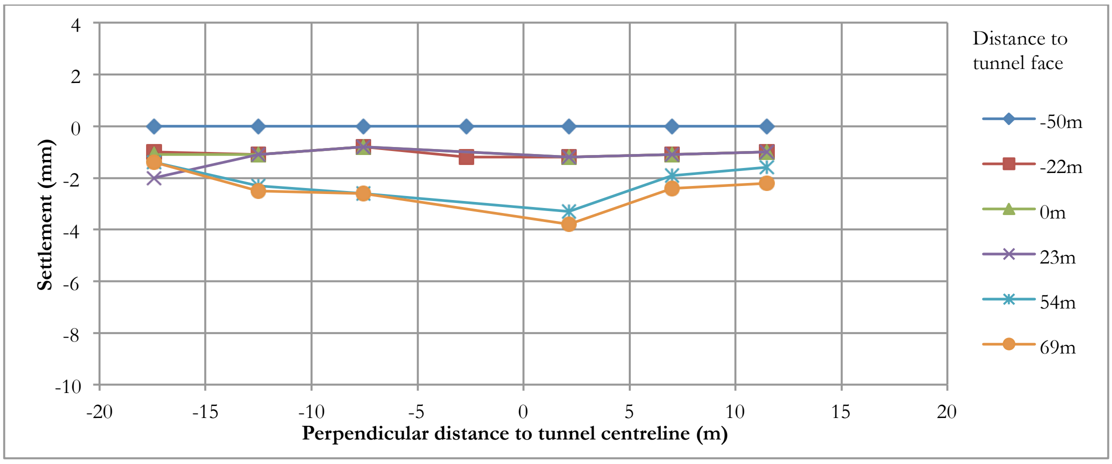
Figure 3.5: Compensational grouting layout at Farringdon

3.6.2 Transverse settlements

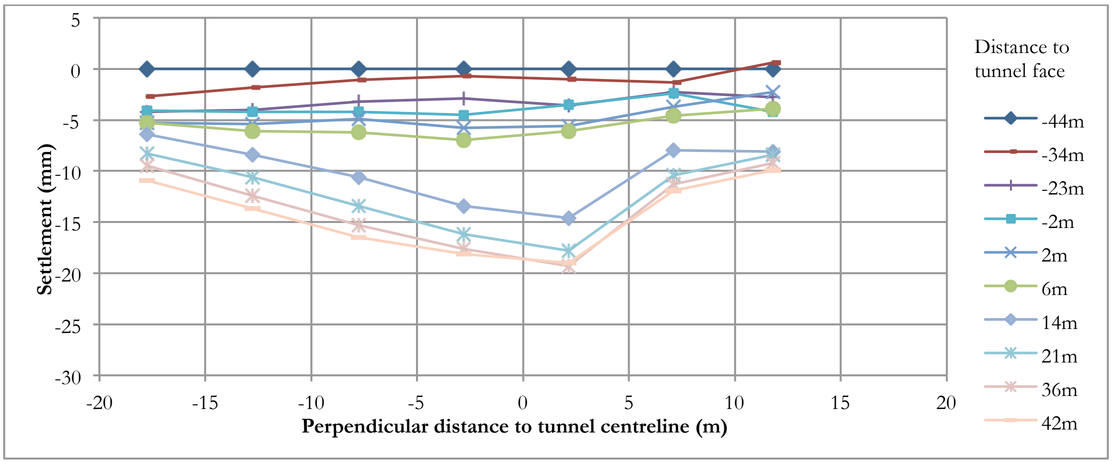
The monitoring data was zeroed for each levelling point when the tunnel face was a distance $z_t + 2d_t$ (~50m) from the point on the tunnel centreline perpendicular with that levelling point. This ensured that only the tunnelling effects were being taken into account. The data was analysed to a distance beyond the levelling points (i.e. once the tunnel had passed through) to a distance between $3d_t$ and $z_t + 2d_t$, depending on the frequency and availability of data recorded. Although further settlements were observed beyond this distance, the cause would have been due to consolidation effects and creep rather than pure tunnelling settlement effects and thus the settlement was only taken up to this point. This ensured a better calculation of the undrained volume loss.

The perpendicular distance from the tunnel centreline to each levelling point is plotted against the vertical settlement for the greenfield cross sections in Figure 3.6, for a number of different distances to the tunnel face. The data during the pilot tunnel construction was limited: since the tunnels advanced at as much as 28m per day and data was not captured every day during this period, there was a distinct lack of data to build an accurate, continuous profile of the pilot tunnel settlement profile. During the SCL enlargement, however, data was captured on a daily basis and combined with the fact the tunnels advanced approximately 2-3m per day, a far more continuous profile was able to be obtained.

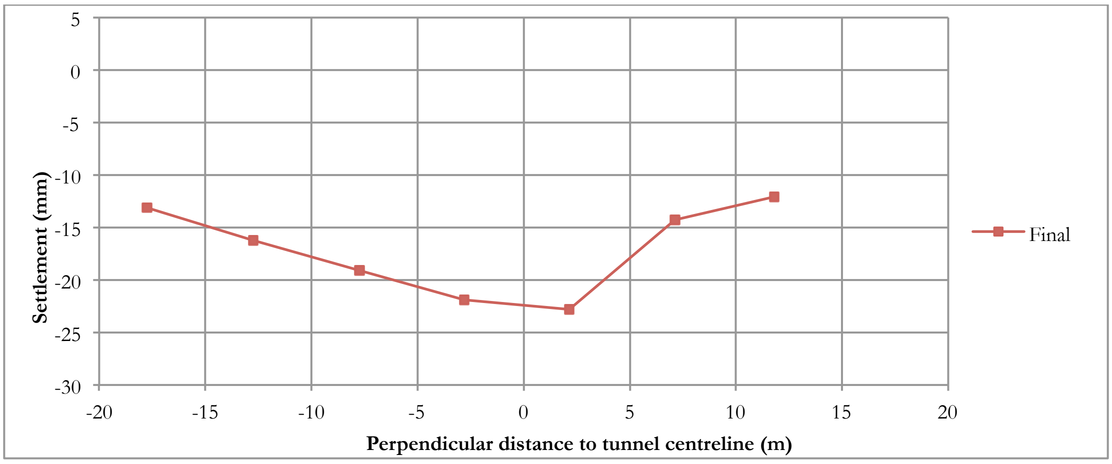
The WB and EB tunnels both show similar total settlements of ~25mm, with the maximum settlement at the tunnel centreline and the profile taking the form of a Gaussian trough, as would be expected.



Pilot only settlement



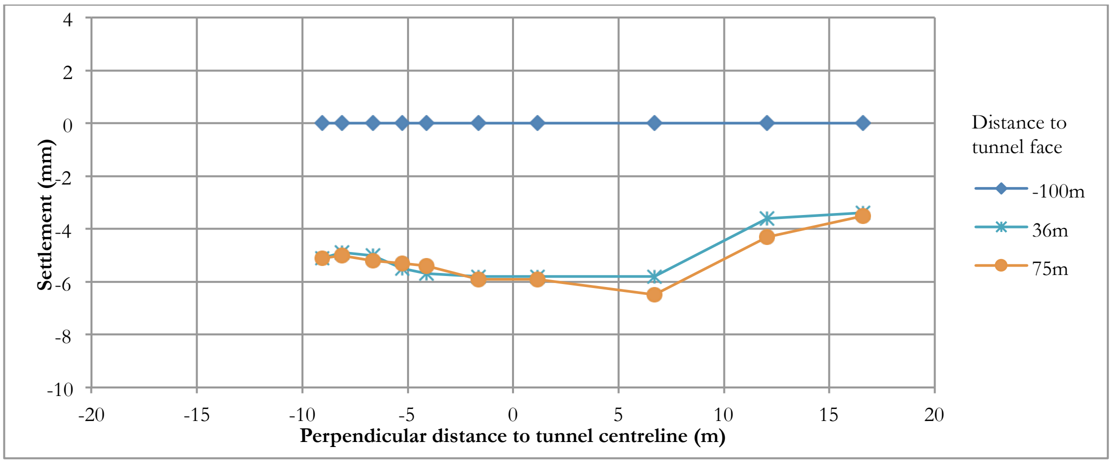
SCL enlargement only settlement



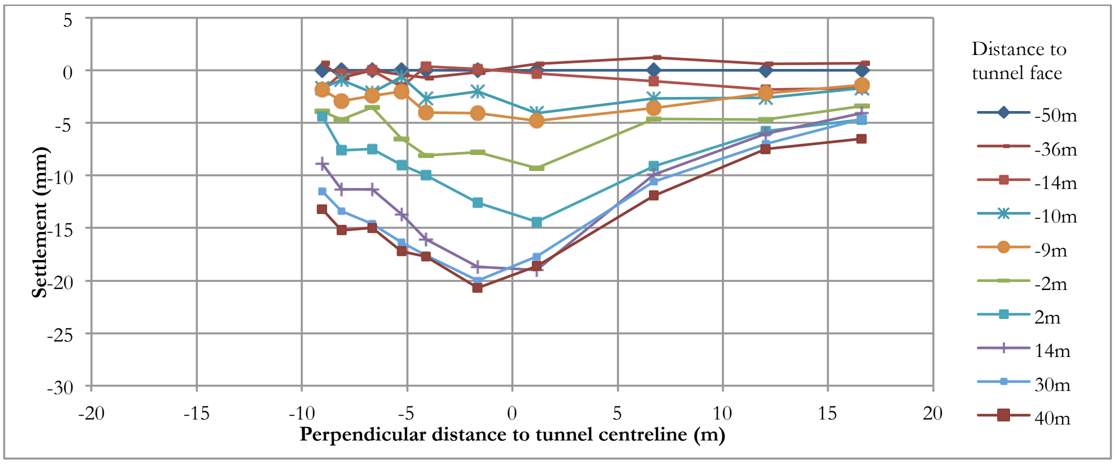
Pilot and SCL combined total settlement

a) Cross section EB GF01

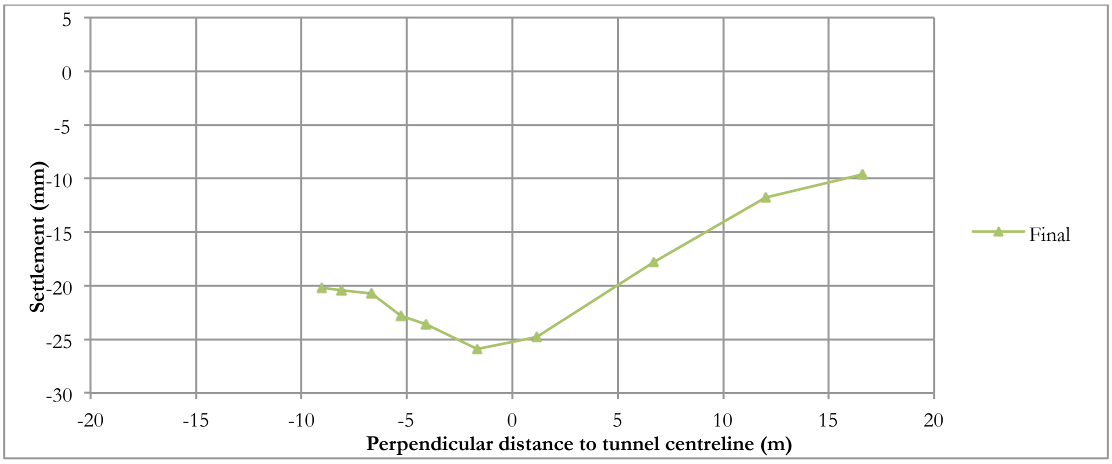
Figure 3.6: Greenfield transverse settlement profiles



Pilot only settlement



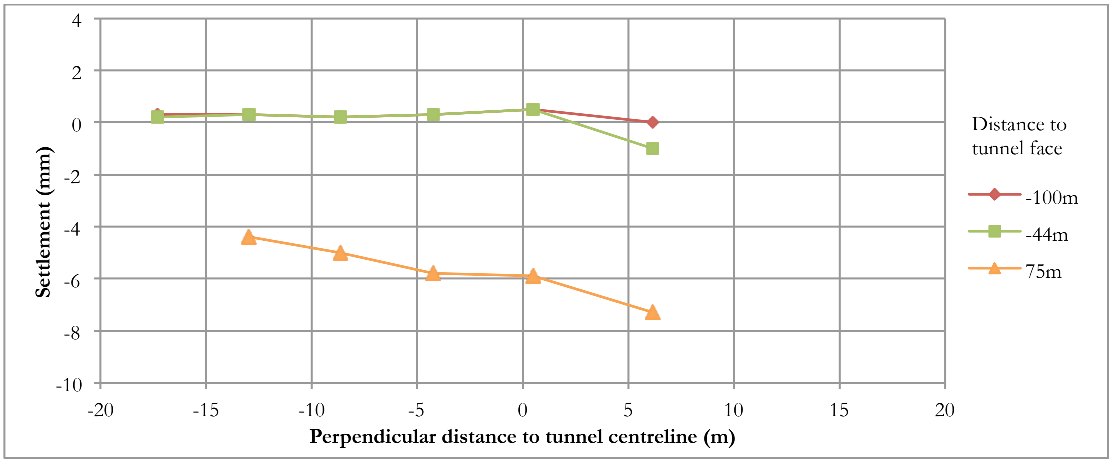
SCL enlargement only settlement



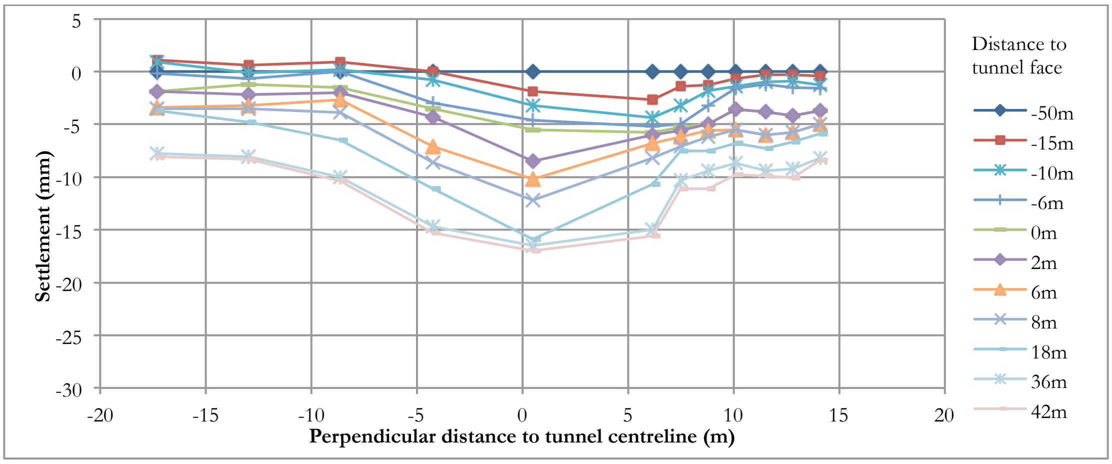
Pilot and SCL combined total settlement

b) Cross section EB GF02

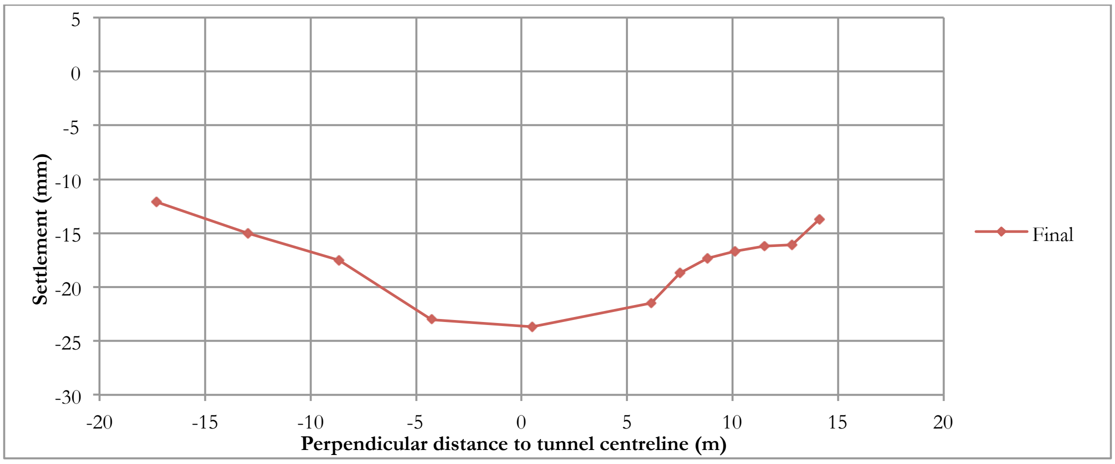
Figure 3.6: Greenfield transverse settlement profiles (continued)



Pilot only settlement



SCL enlargement only settlement



Pilot and SCL combined total settlement

c) **Cross section WB GF01/GF02**

Figure 3.6: Greenfield transverse settlement profiles (continued)

3.6.3 Longitudinal Settlements

Normalised longitudinal settlement profiles are shown in Figure 3.7, plotted against the parallel longitudinal distance from the monitoring point to the tunnel face. As explained previously, the absence of monitoring during the pilot construction meant the data lacked continuity and a clear profile was difficult to achieve. A far better plot was achieved for the SCL enlargement, which can be seen to be the shape of the cumulative probability curve as expected.

The best results are for the EB section that shows as the tunnel face passes through the section (at longitudinal distance = 0m), $S_v \sim 0.3S_{max}$ for the pilot and $S_v \sim 0.5S_{max}$ for the SCL, which is what would be expected for the closed face TBM and open face excavation respectively. The profile for the WB tunnel shows a similar shape to the EB, however, the value of $\sim 0.3S_{max}$ at the tunnel face is slightly below what would be expected.

3.6.4 Volume Loss

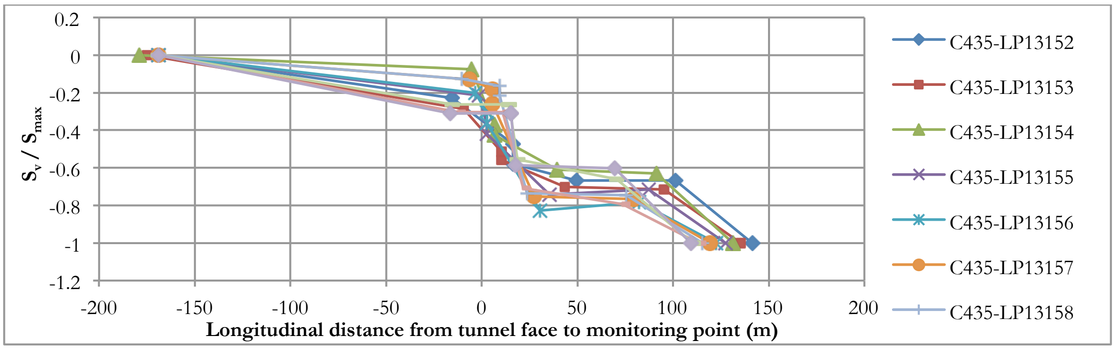
The undrained volume loss was calculated for both the WB and EB tunnels, for the pilot, enlargement and combined construction phases. Since the transverse profiles are Gaussian shaped, it is possible to apply a Gaussian fitting function to the data in order to calculate the point of inflection and thus estimate the volume loss, according to the method layed out in Mair et al. (1997).

The calculated volume losses for the two pilot tunnels showed the most discrepancy, which is to be expected given the quality of the data compared to that for the enlargement tunnels. The value of the volume loss for the combined construction phases was calculated using the final settlement trough, with the WB value of 1.01% being 13% greater than that of the EB tunnel of 0.88% (the WB tunnel was constructed ahead of the EB). Both values are lower than the Crossrail predicted value of 1.25%.

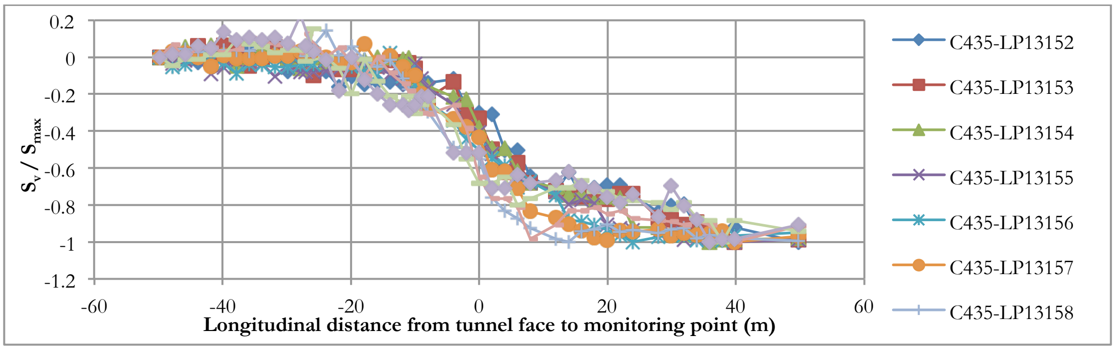
Values of *K* were also estimated based on the settlement data, using Mair et al. (1997):

$$i = K_1 z_1 + K_2 z_2 \tag{3.1}$$

Taking the value of *K* for the overlying river terrace deposits as 0.25, the *K* value of the London Clay could be calculated, using the points of inflection from the field data plots. The values are all close to 0.5, which is the assumed value for London Clay, with the WB enlargement and combined *K* value equalling 0.5 exactly.

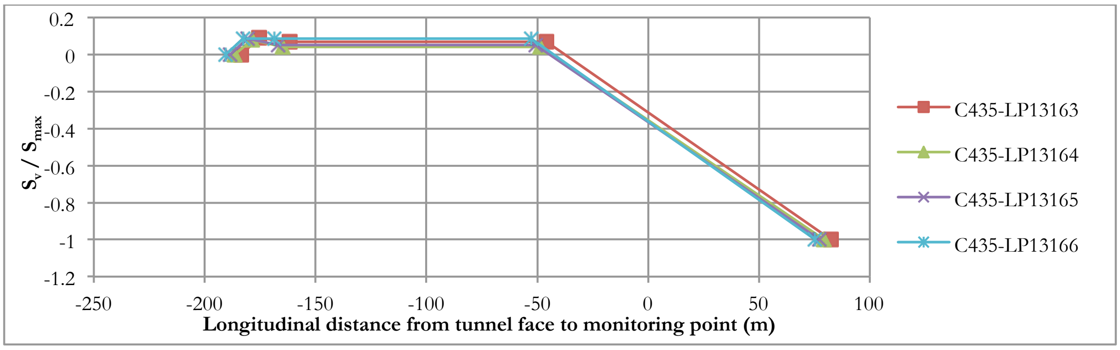


Pilot only longitudinal settlement

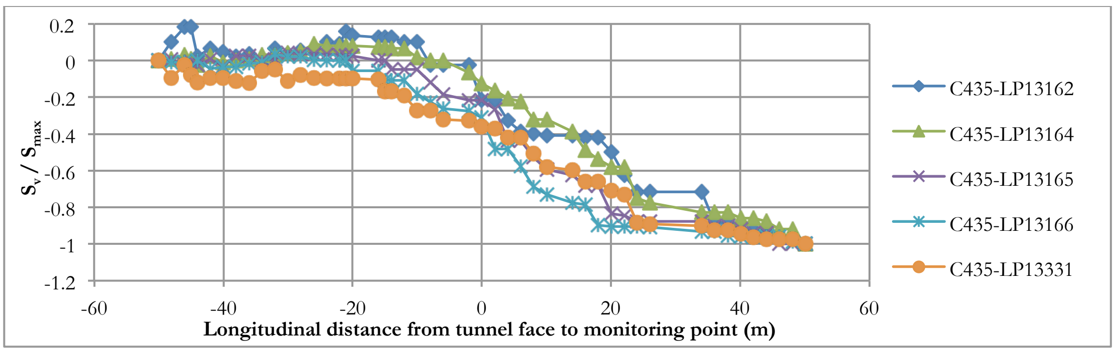


SCL only longitudinal settlement

a) Cross section EB GF02



Pilot only longitudinal settlement



SCL only longitudinal settlement

b) Cross section WB GF02

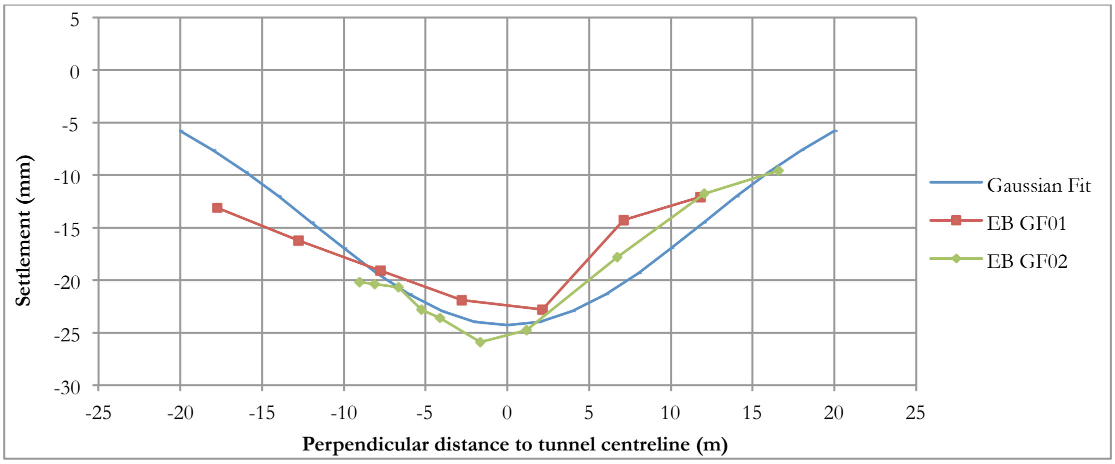
Figure 3.7: Greenfield longitudinal settlement profiles

Tunnel	D_t (m)	i (m)	$S_{v,max}$ (mm)	V_L (%)	K
EB pilot	7.18	13.6	4.8	0.40	0.53
WB pilot	7.18	16.4	5.9	0.60	0.60
EB enlargement[1]	10.195	11.0	19.5	1.31	0.42
WB enlargement[1]	10.195	13.8	17.8	1.49	0.50
EB pilot & enlargement	10.195	11.8	24.3	0.88	0.45
WB pilot & enlargement	10.195	13.9	23.7	1.01	0.50

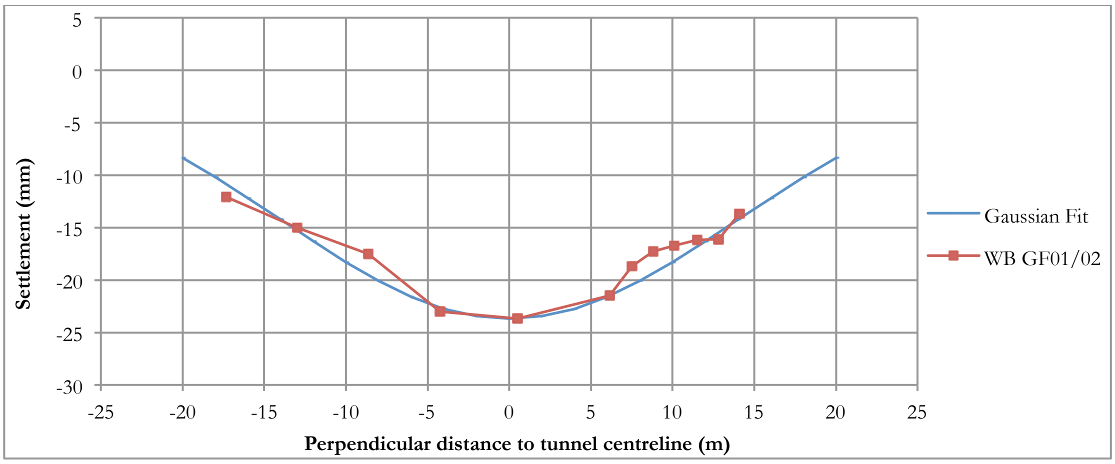
1: calculation of volume loss using only addition area excavated, not total tunnel diameter

Note: EB calculations used an average of two cross sections

Table 3.5: EB and WB volume losses for different construction phases



a) Eastbound



b) Westbound

Figure 3.8: Gaussian fits to transverse settlement troughs

3.7 Building displacements

Figure 3.4 shows the sections that were used in the analysis of the building displacements. Automated 3D prism data was analysed, however, the data contained a considerable amount of noise and the readings showed far lower displacements than that of the BRE data, thus was not used in the analysis since it was considered inaccurate and of poor quality. This resulted in only BRE prisms used for the building displacement analysis, which limited the number of points and cross-sections that could be analysed.

3.7.1 Transverse displacements

It is difficult to accurately fit a Gaussian curve to the building displacement plots due to a small number of data points, however, there are still a number of interesting observations:

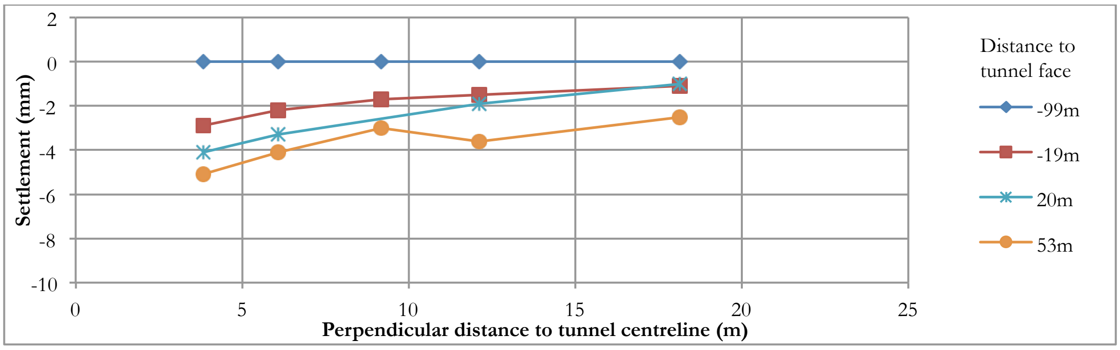
1) The Smokery has two cross-sections analysed, one that exhibits reasonably rigid behaviour (SM 01) and one that exhibits slightly more flexible behaviour (SM 02). This can be explained by the fact that at SM 01 there will be a higher building stiffness due to the shape of the building, as well as the presence of a lift shaft on that section. This difference in rigid and flexible behaviour will result in a degree of torsion in the building. This rigid profile also suggests that the behaviour is controlled by the pile group and ground beam acting together, rather than individual piles behaving independently which would lead to a flexible response.

2) 78 Cowcross shows reasonable rigidity, again indicating that rigid pile group behaviour controls, similarly to The Smokery. The displacement values also closely match those of the Smokery sections. This is hardly surprising given the building substructures are extremely similar. A large difference in displacement can be observed at the join between 78 and 85 Cowcross where the foundation type changes from piled to strip footing.

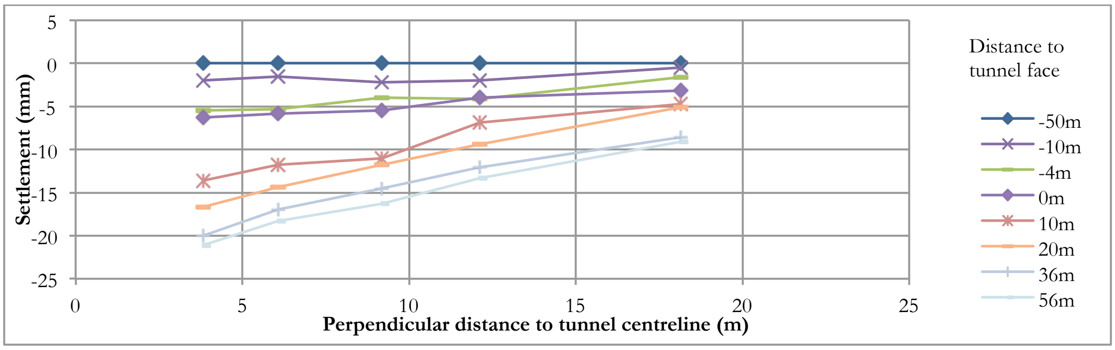
3) 85 Cowcross behaves relatively flexibly. One of the cross-sections shows much larger displacements than the other, despite being further from the tunnel centreline on the whole. This is because the other cross section is above the area of local piling, which will have an effect of limiting the displacement compared to a shallow foundation. The cross-section that is not above the piles shows a much larger displacement than all the other sections.

3.7.2 Longitudinal displacements

The longitudinal displacements profiles are shown in Figure 3.10 for the pilot and enlargement tunnels. All three buildings show flexible behaviour in the longitudinal direction, with $S_v \sim 0.3\text{-}0.4S_{max}$ at the tunnel face for the enlargement, which, as discussed

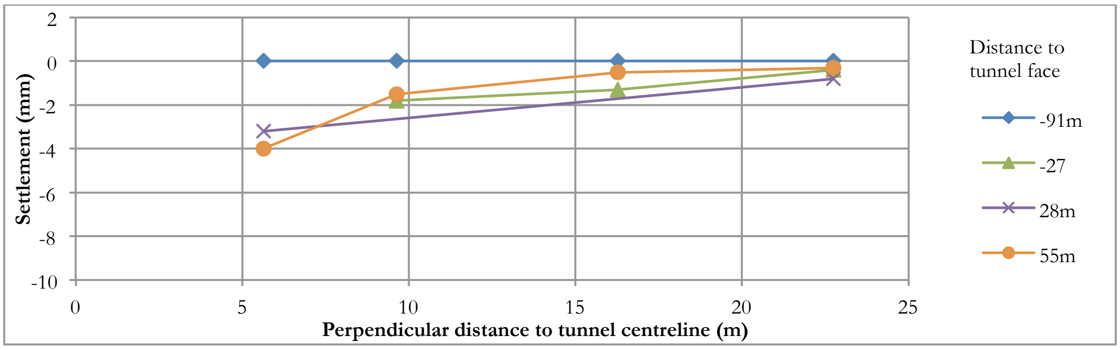


Pilot only displacement

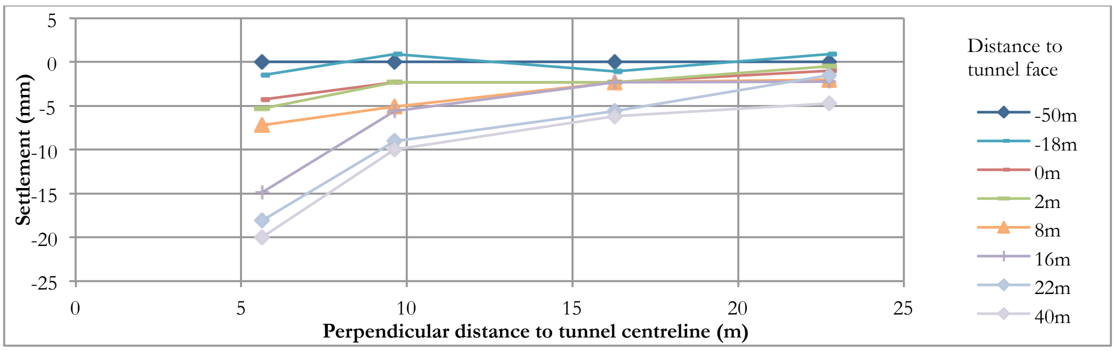


SCL enlargement only displacement

a) Cross-section SM 01



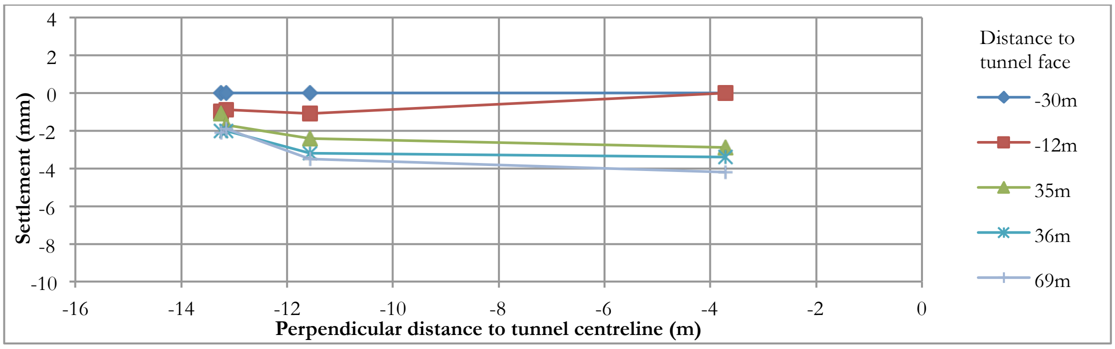
Pilot only displacement



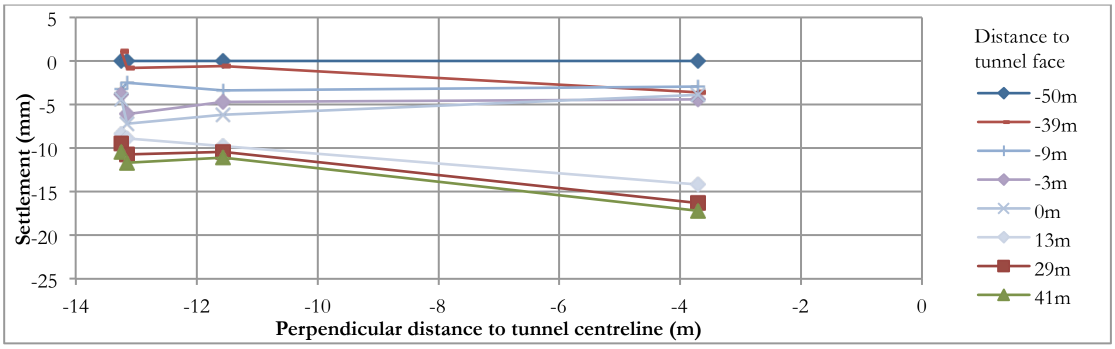
SCL enlargement only displacement

b) Cross-section SM 02

Figure 3.9: Building transverse displacements

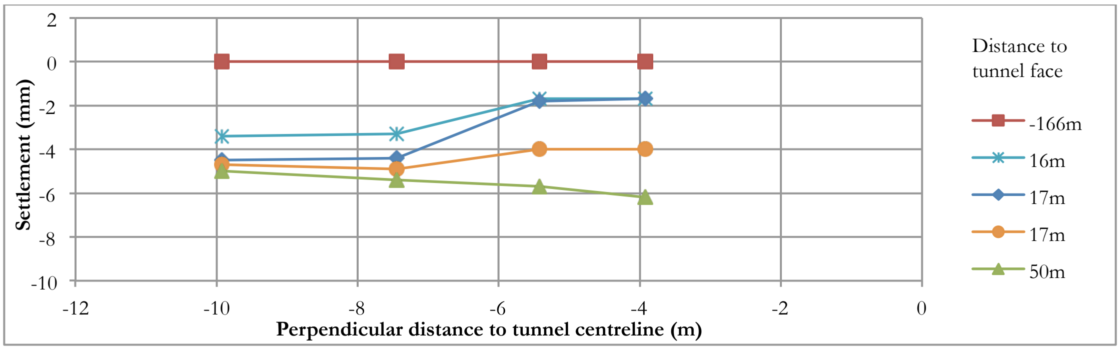


Pilot only displacement

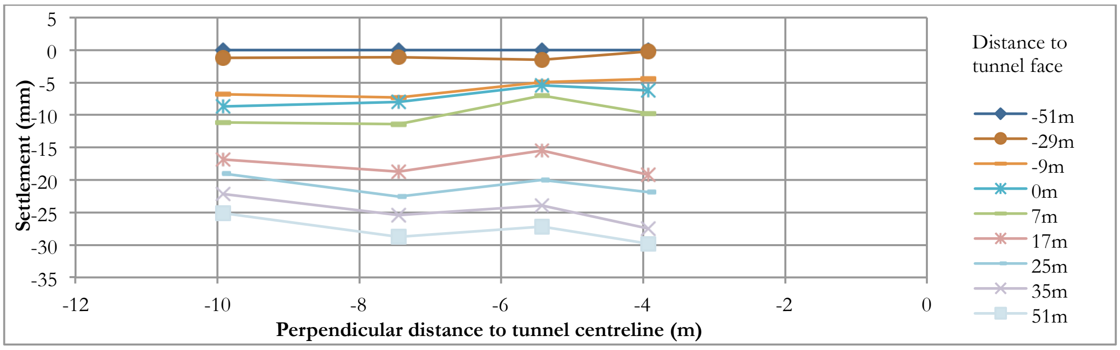


SCL enlargement only displacement

c) Cross-section 78 01



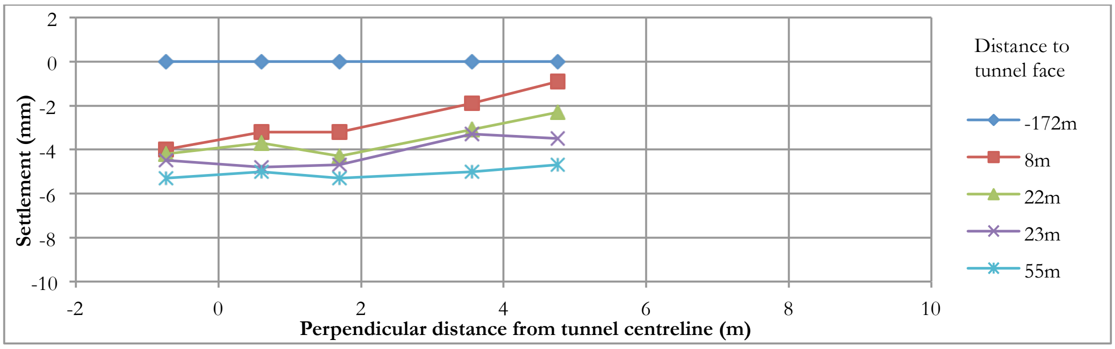
Pilot only displacement



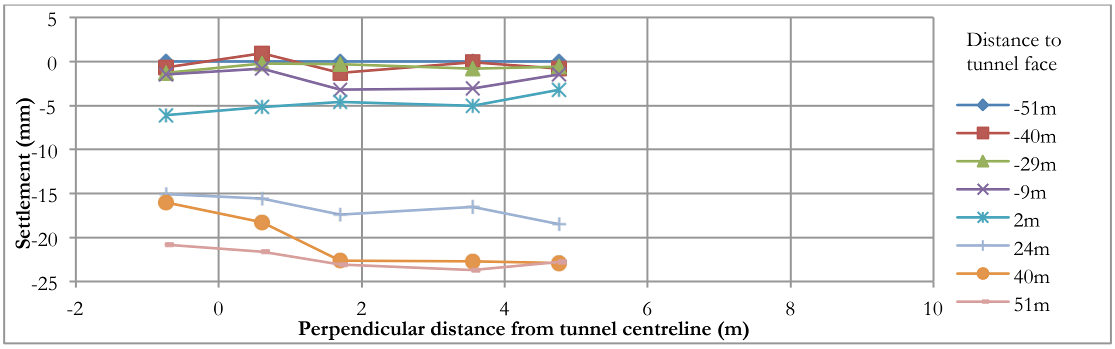
SCL enlargement only displacement

d) Cross-section 85 01

Figure 3.9: Building transverse displacements (continued)

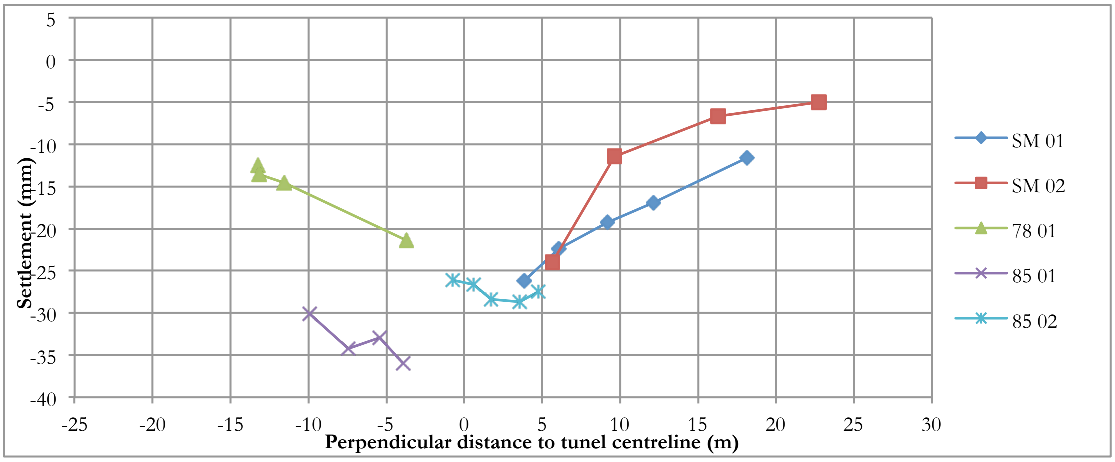


Pilot only displacement



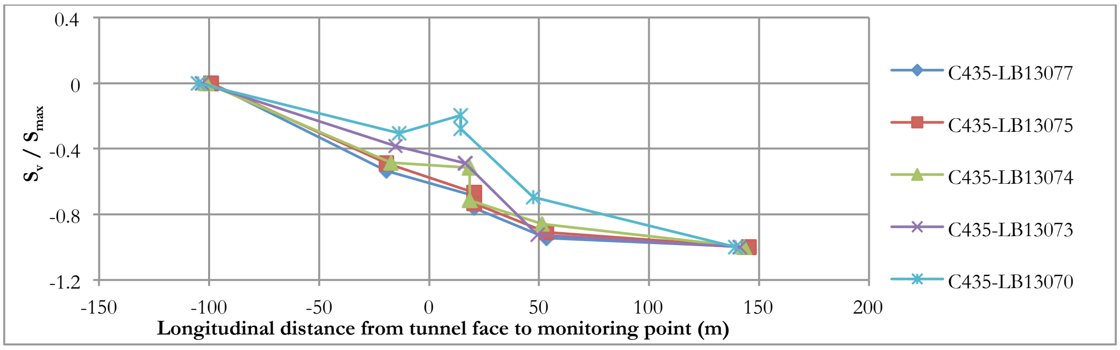
SCL enlargement only displacement

e) Cross-section 85 02

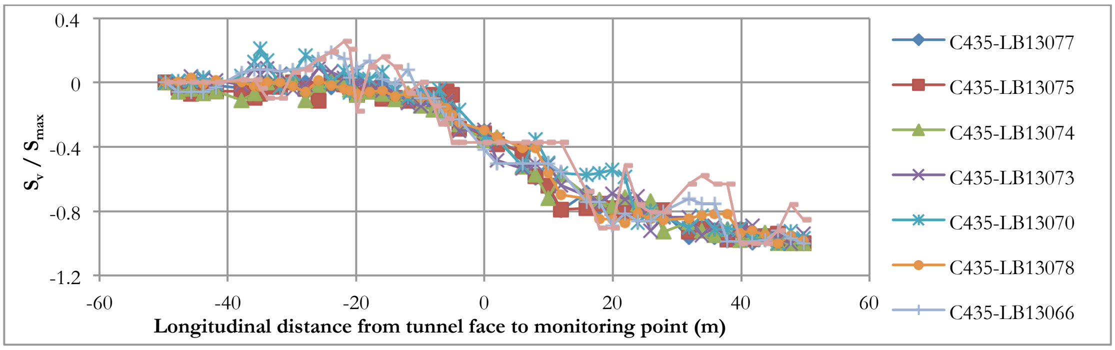


f) Combined pilot and SCL enlargement total settlement for all sections

Figure 3.9: Building transverse displacements (continued)

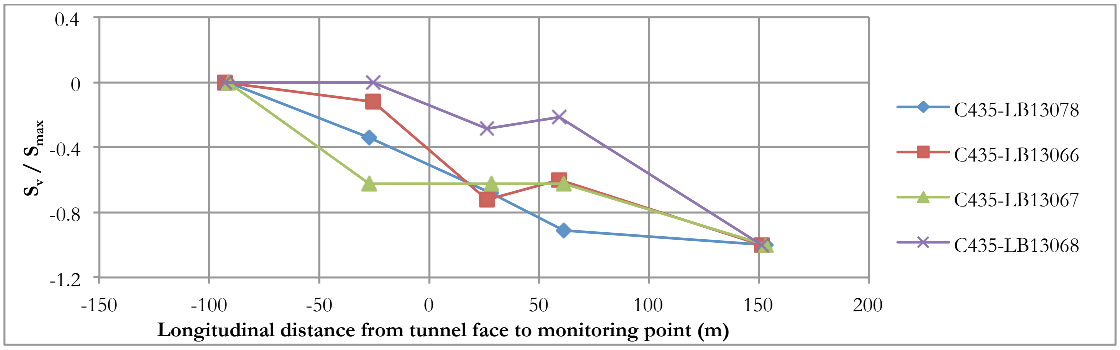


Pilot only displacement

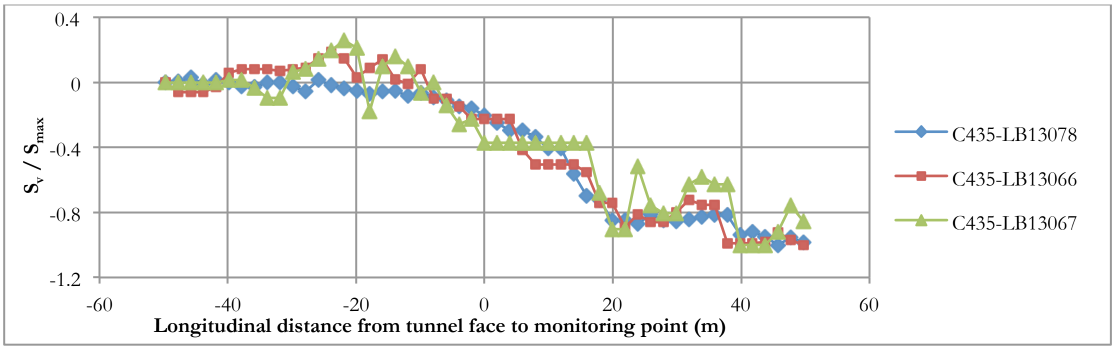


SCL enlargement only displacement

a) Cross-section SM 01



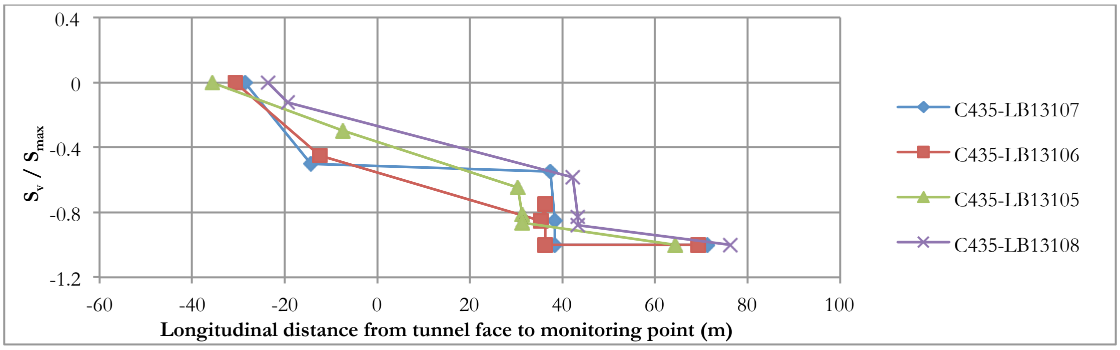
Pilot only displacement



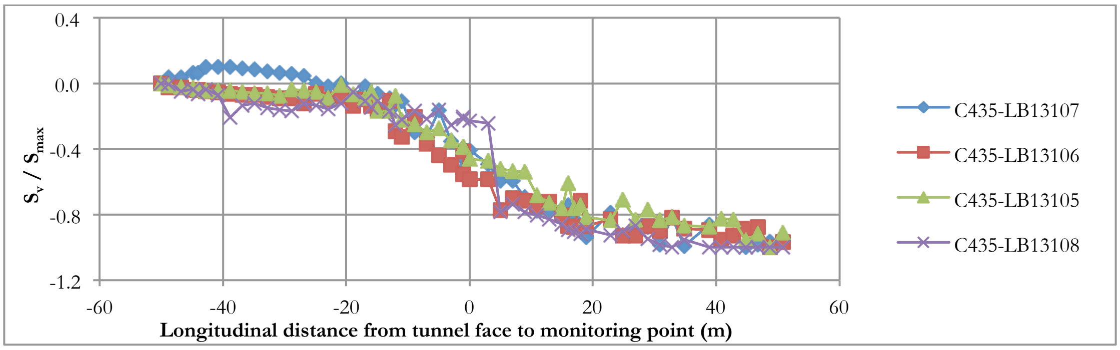
SCL enlargement only displacement

b) Cross-section SM 02

Figure 3.10: Longitudinal building displacements

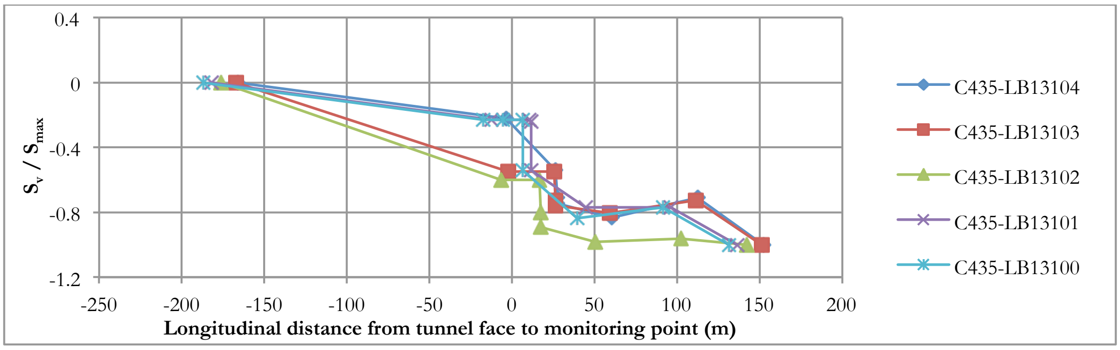


Pilot only displacement

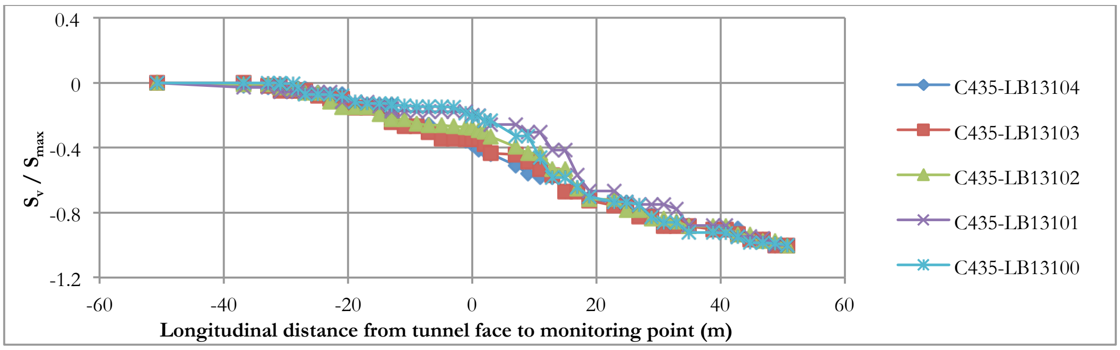


SCL enlargement only displacement

c) Cross-section 78 01



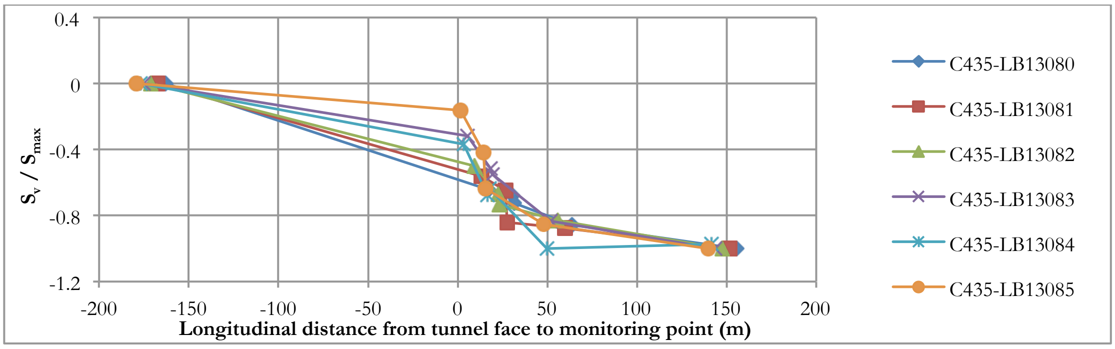
Pilot only displacement



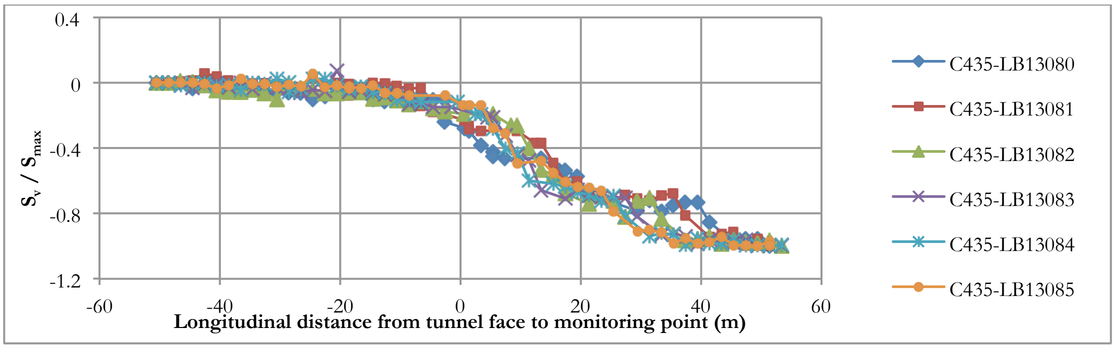
SCL enlargement only displacement

d) Cross-section 85 01

Figure 3.10: Longitudinal building displacements (continued)



Pilot only displacement



SCL enlargement only displacement

e) Cross-section 85 02

Figure 3.10: Longitudinal building displacements (continued)

earlier, is slightly below the expected value of 0.5 and below the EB greenfield value of approximately 0.5. This is likely to be due to the ground beam stiffness not allowing fully flexible behaviour and therefore leading to a value of settlement at the excavation face that is lower than the greenfield value.

3.8 Overall combined greenfield and building settlements

In order to establish an overall picture of the settlement profile across the entire region, settlement contours were plotted with tunnel progress. The monitoring points were interpolated between using a fitting function. Monitoring points were zeroed when the parallel distance from the tunnel face to the specific monitoring point was $z_d + 2d_t$, and this was matched to the time sequence to show the progress as the tunnels pass the buildings of interest, shown in Figure 3.12.

It is clear that the buildings with a high density of piles, The Smokery and 78 Cowcross, appear to experience less displacement than the predominantly shallow founded 85 Cowcross. The region of local piling at 85 Cowcross does not appear to cause a noticeable reduction in the displacement, with the magnitude at that point just as great as the majority of the rest of the building. As mentioned before, The Smokery can be seen to experience some torsion due to its more complex and non- symmetric shape.

It should be noted that a true profile of the WB settlement trough was not really possible in these plots across the entire tunnel length due to the limited data points; the contours were interpolated between points a significant distance either side of the tunnel centreline and so "miss out" the trough.

3.9 Comparison of greenfield settlements vs. building displacements

In order to make comparisons between the greenfield and building settlements, a subsurface greenfield settlement profile was estimated at the foundation level, based on the greenfield surface results. Because the buildings are founded at the basement level, approximately 3.5-4m below the ground surface, this results in a better comparison of greenfield vs. building settlement. Based on the interpolated soil stratigraphy at the different locations, and applying the volume loss calculated in Section 3.6.4, a subsurface profile was estimated for each building at the basement level using Mair et al. (1997). These subsurface estimates are plotted with the building displacements for comparison in Figure 3.11.

There is a degree of variation between the different building sections and the greenfield profiles. Section 78 01 appears to match the greenfield profile well, particularly at the points further from the tunnel centreline. SM 02 and SM 01 are above and below the greenfield line respectively, with SM02 showing slightly greater discrepancy. As

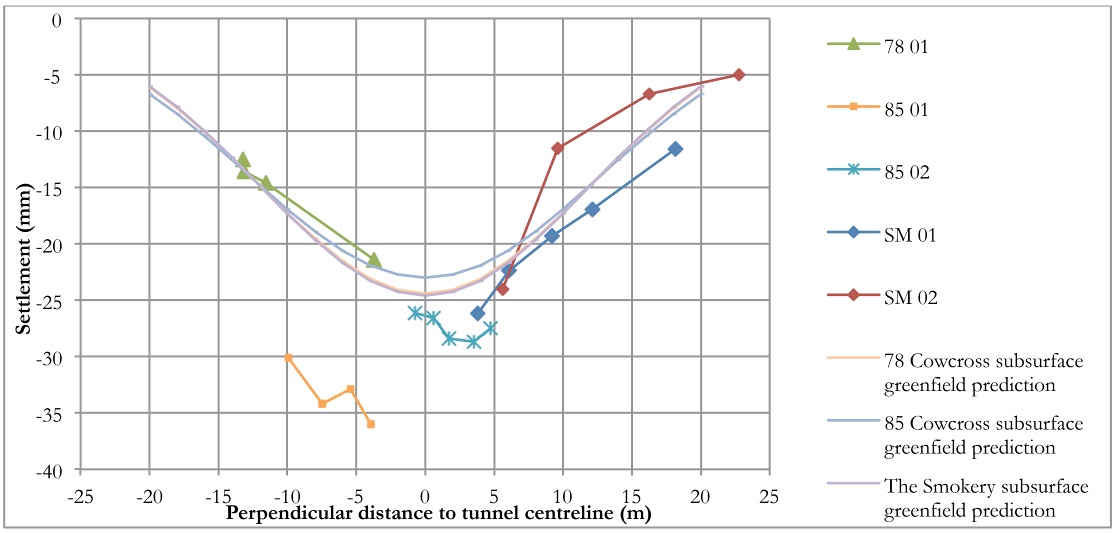


Figure 3.11: Total building displacements vs. subsurface greenfield settlement

discussed previously, SM 01 is behaving more rigidly (presumably due to the higher building stiffness at that section) and the non uniform shape of The Smokery may be a factor in the difference in settlements observed at the two sections. 85 Cowcross can clearly be seen to displace a greater distance than the greenfield settlement, however, section 85 01 is not founded on piles and thus the settlement of strip foundations will control. Section 85 02 is predominantly founded on strip foundations but is also above the area of local piling, which appears to have an effect of reducing the overall total displacement of the section, although not quite enough to be equal to the greenfield settlement.

3.9.1 Discussion

In general, the piled sections can be observed to settle less than or close to the greenfield settlement further away from the tunnel, and greater than the greenfield settlement close to the tunnel centreline. As discussed in Chapter 2, both centrifuge and numerical modelling would predict piles with a factor of safety of ~3 (the FoS used in the design of the three buildings) to displace a greater distance than the greenfield behaviour close to the tunnel centreline, and to displace a similar distance to the greenfield behaviour away from the tunnel centreline. As such, qualitatively the buildings seem to be behaving similarly to these predictions (although the magnitude by which the buildings displace compared to the greenfield settlements for piles above the centreline is lower than would be expected, as discussed in Chapter 4). These observations, however, are not completely in line with those made on similar case studies at Bishops Bridge Road and Kempton Court by Williamson (2014), where the building displacements closely matched the greenfield settlements across the whole settlement trough. A number of reasons were proposed for this, the primary ones being pile tensile cracking and the effect of the ground beam bearing capacity.

- Tensile cracking: the piles closest to the tunnel centreline will be subject to tension in the lower part of the pile as the pile is unloaded by the tunnel construction, and the pile is effectively pulled vertically downwards. If the pile cracks, which is likely given the steel reinforcement does not extend the full length of the pile, the settlement at the pile head will be reduced.
- Ground beam effect: it is possible that the buildings may shed significant load at shallow depth through the bearing capacity of the continuous ground beam slab, as it acts to restrain the pile head movement. Factors of

safety are likely to also mean that the actual loads will be lower than the design loads and thus the ground bearing slab is able to carry a greater proportion of the load.

Considering both The Smokery and 78 Cowcross, both of these effects would be expected; both buildings have piles that may be subject to tensile cracking based on their position relative to the tunnel, the magnitude of the tunnel settlements and the length of the pile reinforcement. They are also both founded on ground bearing slabs, at the surface of the London Clay layer, which may result in increased bearing capacity beneath the slab as the piles are pulled down. As mentioned above, the building displacements, despite being greater than the greenfield settlements, are still not as large in magnitude as would be expected and thus these factors are likely to be responsible. However, it would appear that these effects are perhaps not as prominent as for the buildings analysed by Williamson (2014). In addition, there is likely to be a degree of error in the data and because only a limited number of points could be analysed, the overall picture is slightly limited. In order to try and determine the magnitude of the effects mentioned above, the field data was compared to a *t-z* model developed in Cambridge, the results of which are discussed in Chapter 4.

3.10 Case Study Summary

- The greenfield transverse settlements closely fit a Gaussian trough and the greenfield longitudinal settlements are the shape of a cumulative probability curve.
- The building displacements vary depending on foundation type, with the piled buildings exhibiting lower displacements than the shallow footings.
- The building displacements were of greater magnitude than the greenfield settlements close to the tunnel centreline and of similar magnitude away from the tunnel centreline.
- On the whole, the buildings behaved with a degree of rigidity in both the longitudinal and transverse directions, suggesting pile group effects controlled the behaviour.
- The magnitude of the effects of tensile cracking and ground beam stiffness is still unknown, and needs to be investigated further.

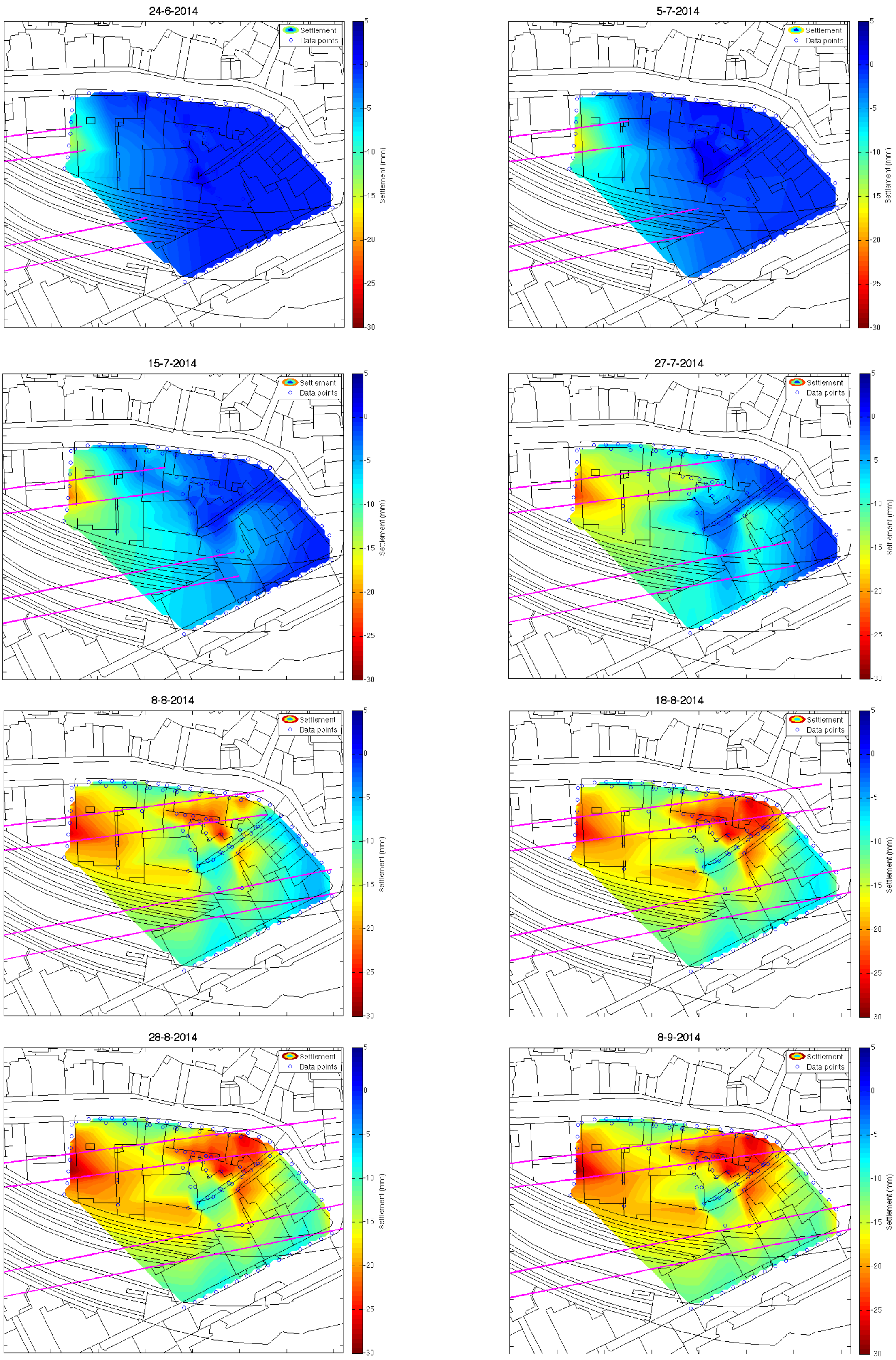
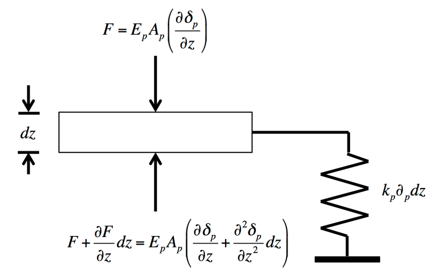


Figure 3.12: Settlement contour plots at Farringdon during tunnel construction progress

Chapter 4 *t-z* modelling

4.1 Background

t-z models provide a good method for analysing pile settlement behaviour. As discussed in Chapter 2, the pile is split into elements and all influence factors taken to be zero except at the location of the element being analysed. Each element is modelled as being connected to a Winkler spring, as shown in Figure 4.1.



$$E_p A_p \frac{\partial^2 \delta_p(z)}{\partial z^2} - k_z \delta_p(z) = 0$$

Figure 4.1: Pile element Winkler spring model

The standard Winkler equation shown above is altered to account for the ground settlement, δ_{ff}, (Equation 4.1). This was solved analytically by Klar and Soga (2005).

$$E_p A_p \frac{\partial^2 \delta_p(z)}{\partial z^2} - k_z \left(\delta_p(z) - \delta_{ff}(z)\right) = 0 \tag{4.1}$$

The equation can also be solved numerally in a finite difference formulation, giving the advantage of allowing variation in soil stiffness along the pile. Expressing the equation in matrix form and rearranging, the pile element displacement can be found,

$$\{\delta_p\} = [K_{pz}]^{-1}([K_{sz}]\{\delta_{ff}\} + \{F_z\}) \tag{4.2}$$

where K_{pz} and K_{sz} are the pile and soil stiffness matrices respectively.

Loads are applied at the pile head and via equilibrium and compatibility, the displacement can be found for each element. During tunnelling the soil movements induce changes in the loading in the pile, which results in settlement to remobilise shaft friction and maintain equilibrium. For the t-z analysis the head load is assumed to be constant.

This chapter aims to model the Farringdon case, in order to see how closely the true behaviour can be predicted and how various factors effect the settlement.

4.2 Model theory

The model used in this analysis is for a single pile, and is based on linear elastic – perfectly plastic pile-soil interaction. It works in two stages: the first stage is an initial loading stage, to analyse the pile prior to tunnel construction. A load is applied at the head and the initial base displacement of the pile is estimated based on Vardanega et al. (2012), by assuming a rigid pile displacement. The load at the base element can then be found based on the displacement of the pile relative to the soil. This is used to set the value of the strain, dW/dz, where W is the pile displacement and z is the element length, and thus τ can be found. The Huen Solver numerical method (2^{nd} order Taylor Series) is then used to solve the displacement and load at each element, using a forward and backward projection, which continues up the pile element by element and an estimate of the head load is made. The base displacement is then varied iteratively until the correct head load is found.

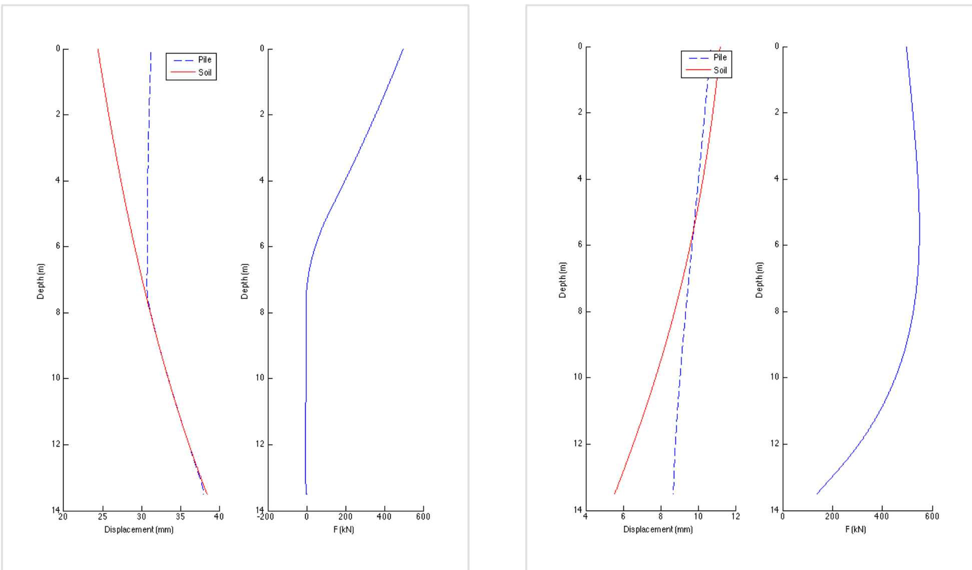
The second phase models the tunnelling influence. The starting values for this phase are taken as the end values for phase one. A volume loss is then assumed which creates a soil displacement profile along the pile length. In this case, the soil displacement is used to initially estimate the base displacement. Similarly to phase one, δW (pile displacement) and δSz (soil displacement) are used to calculate the strain and the value of τ. The Huen solver is used as above, with the base displacement again varied iteratively until equilibrium of the head load is found.

4.2.1 Tension cut-off

As the tunnel construction takes place and the pile is pulled downwards, part of the pile may go into tension. Since the tensile strength of concrete is very low, this is likely to result in pile cracking if the steel does not extend to the full pile depth (which is often the case). The soil displacement profile varies with the distance away from the tunnel centreline. Above the tunnel centreline, the soil close to the tunnel crown will displace further than the soil near the ground surface. Further from the tunnel centreline, the opposite is the case. This means that pile cracking is likely to occur close to the centreline and unlikely away from it. The model assumes that any tensile force in the pile cannot be carried in regions where there is no steel reinforcement and sets it to zero, and the part of the pile in this region of zero force will follow the displacement profile of the soil.

Figure 4.2 shows a typical soil and pile displacement profile for a pile above the centreline and a pile away from the centreline. The pile axial force is also shown. The pile modelled is 450mm diameter, 13.5m in length, with a 6m depth of 0.26% steel reinforcement by area, and a design factor of safety equal to 3, i.e. a typical pile at 78 Cowcross/The Smokery. A volume loss of 0.9% was applied.

For the pile above the tunnel centreline (Figure 4.2a), tensile cracking has occurred. At a depth of roughly 7m the pile axial force reaches zero, and thus remains zero for the remainder of the depth since it is not able to carry tension. This results in the pile displacement equalling the soil displacement in this region, as the cracked pile is pulled downwards by the soil. In the non-tensile region and at the pile head, the pile displaces a greater distance than the soil. Away from the tunnel centreline, the pile is not subject to tension because of the different soil profile, which can be seen in Figure 4.2b. Thus, pile cracking does not occur and the pile behaves more rigidly with the displacement profile varying very little with depth and force carried along the entire length. The difference between the soil displacement and the pile head displacement is much smaller.



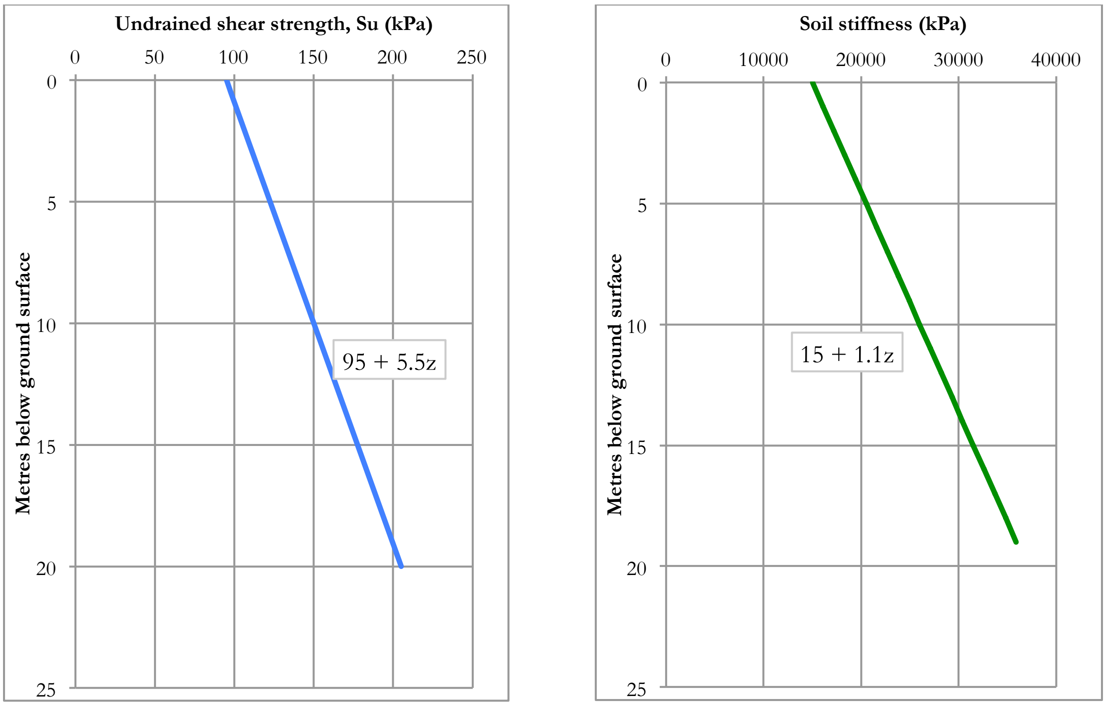
a) Pile above tunnel centreline **b) Pile 15m from tunnel centreline**

Figure 4.2: Typical pile and soil displacement profiles

4.3 Results: Comparison with Farringdon case study

The model has been observed to generally over predict the true building settlement when compared to the field data, particularly close to the tunnel centreline. To better understand the contributing factors to this difference in settlement and more closely predict the true behaviour, certain scenarios were considered and are presented below.

78 Cowcross St. and The Smokery were chosen to model due to their high number of piles and less complex substructure compared to 85 Cowcross, as discussed in Section 3.2. The piles chosen to model were directly below or close to the BRE monitoring points used for the analysis in Chapter 3, to get as close a comparison as possible. The pile parameters used were based on the design values from Section 3.2 and the stratigraphy as described in Section 3.5. The soil strength and stiffness profiles are shown in Figure 4.3 for the London Clay, into which the piles were entirely founded and therefore these profiles were used in the model. The adhesion factor, α (the correlation between skin friction and undrained shear strength), was taken to be 0.5 which is the typical value for London Clay.



a) Soil strength **b) Soil stiffness**

Figure 4.3: Soil strength and stiffness properties used in model for London Clay

4.3.1 Effect of tension cut-off

In order to assess the impact that pile cracking would have on the building settlement, the model was used both in the case where tension was allowed in the pile (i.e. no tension cut-off), and in the case where no tension was allowed (i.e. tension cut-off). The predicted pile head displacements from the *t-z* model for the two cases are plotted against the field data for the various building cross sections in Figure 4.4. A volume loss of 0.9% was applied (as calculated in Chapter 3) and a pile factor of safety of 3 used.

Tensile forces are only experienced in piles near the tunnel centreline, as discussed in the previous section; as such, there is only a difference in predicted profiles for the piles nearest the centreline. In addition, since the majority of the building sections lie outside the area in which tensile cracking would be expected to occur, in this case the tension cut-off does not have a particularly large impact. That said, for the piles that are subject to tension a small reduction in settlement can be observed and therefore this is likely to be contributing factor. It is clear that the effect of this will be larger for buildings directly above the centreline.

4.3.2 Effect of pile safety factor

It is possible that the piles were built with a different factor of safety to the design records, and as such this may be a contributing factor to the difference in predicted and actual behaviour. The *t-z* predicted profiles for varying factors of safety are plotted in Figure 4.5. A volume loss of 0.9% was applied, and the tension cut-off was used.

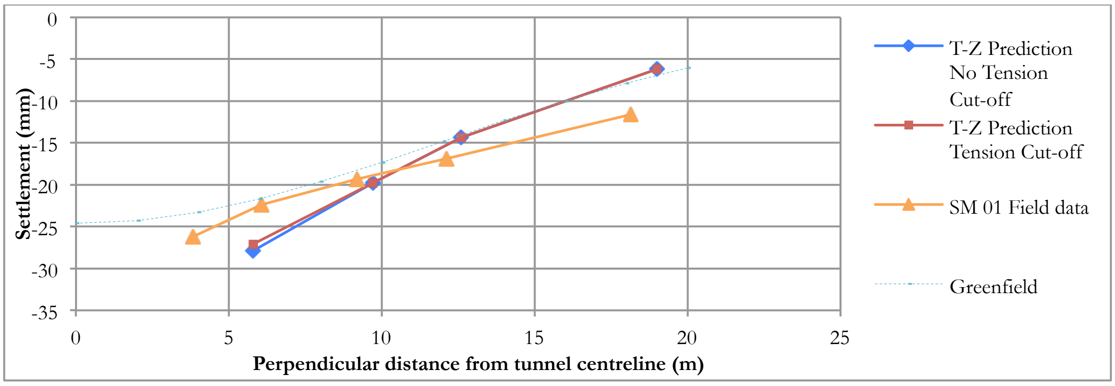
The discrepancy between different FoS is relatively small, particularly between FoS = 3 and FoS = 4. Since it is unlikely the piles were built with a FoS lower than 3, the difference between whether they were built as stated in the design criteria with FoS = 3, or whether they were built with a greater FoS, is unlikely to have a major impact on the difference in predicted and actual behaviour.

4.3.3 Effect of soil stiffening

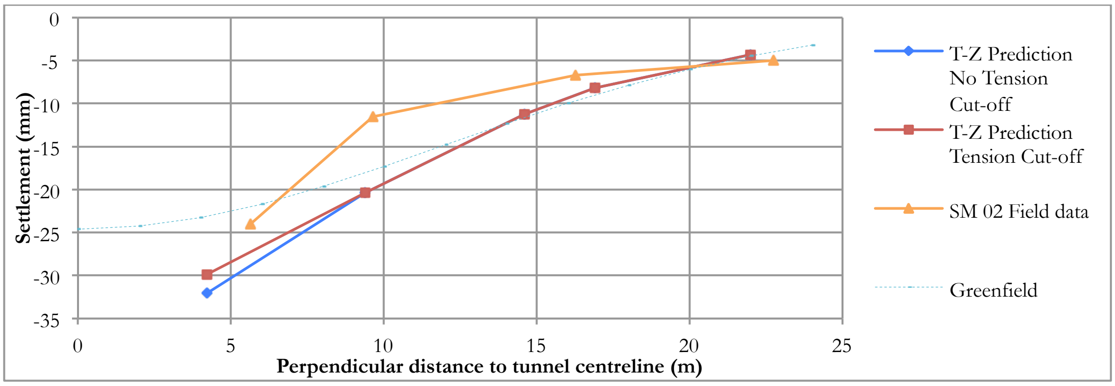
A high density of piles interacting with each other may well lead to soil stiffening effects, and this would be expected to lower the total settlement. This is a highly complex effect, and would be an extremely useful area of future research, however, was considered unachievable within the practical constraints of this project due to its complexity. Factors such as the number of piles, the pile spacing and pile dimensions will all contribute to the magnitude of this effect. As a very basic trial, piles were modelled with varying values of soil stiffness and only at very large factors compared to the actual assumed stiffness were there noticeable differences in the displacement, and even then the contribution to total displacement reduction was relatively small. Although this is an extremely basic approach, it suggests there is a more prominent effect that is the main contributor to the lower than expected settlement.

4.3.4 Effect of ground bearing slab

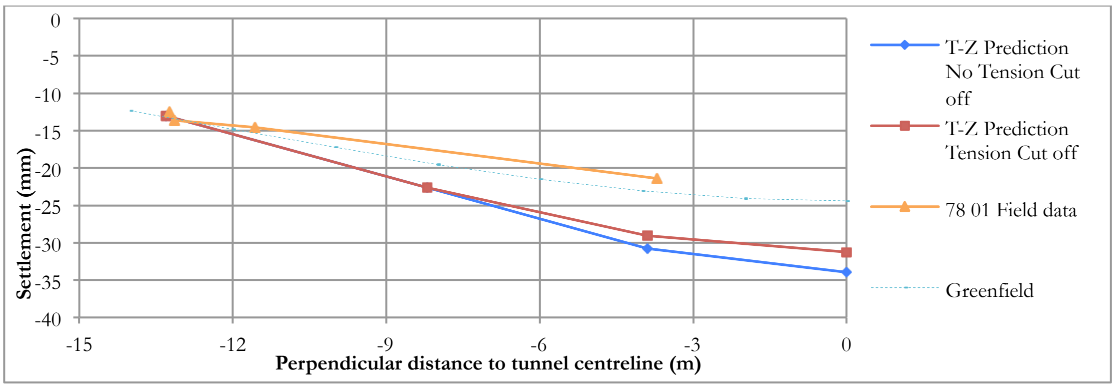
Both 78 Cowcross and The Smokery have a continuous ground bearing slab into which the piles are cast, as discussed in the substructure details in Chapter 3. The effects discussed above have been shown to have a noticeable but relatively minor contribution to the difference in predicted and actual building behaviour. This implies the effects of the ground bearing slab are likely to be mostly responsible. Part of this contribution may come from the bearing capacity of the slab. As the tunnelling occurs and the pile



a) SM 01 *t-z* prediction



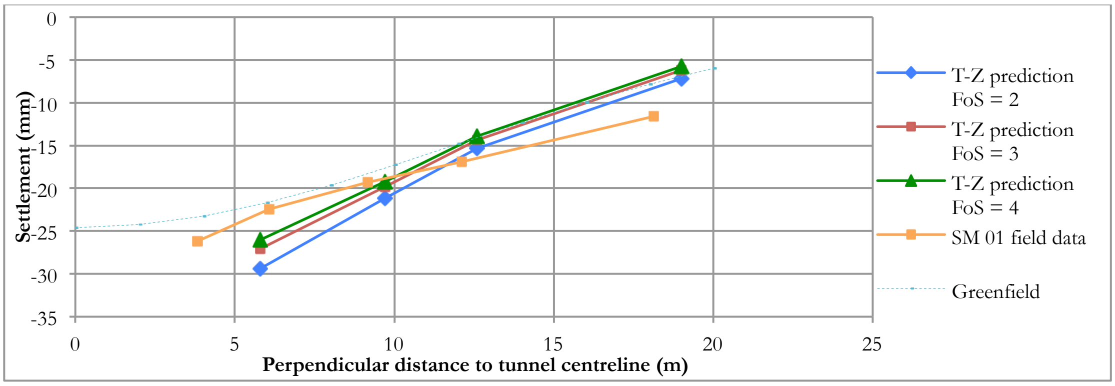
b) SM 02 *t-z* prediction



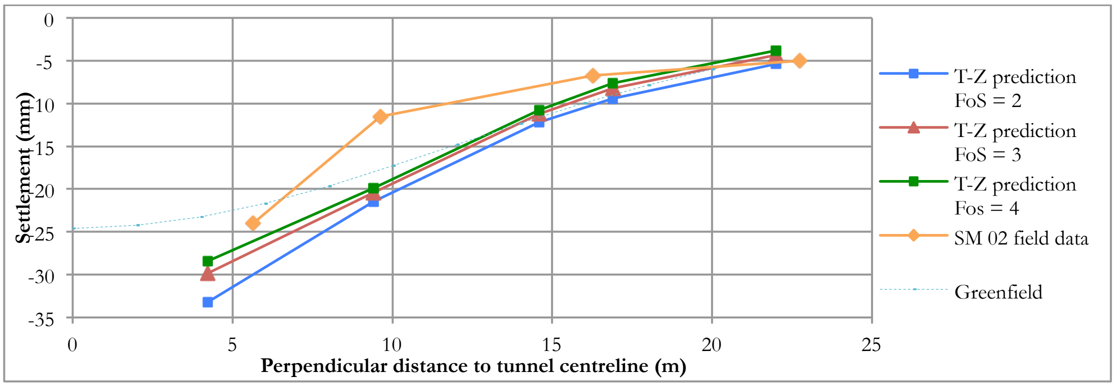
c) 78 01 *t-z* prediction

Figure 4.4: Effect of the tension cut-off

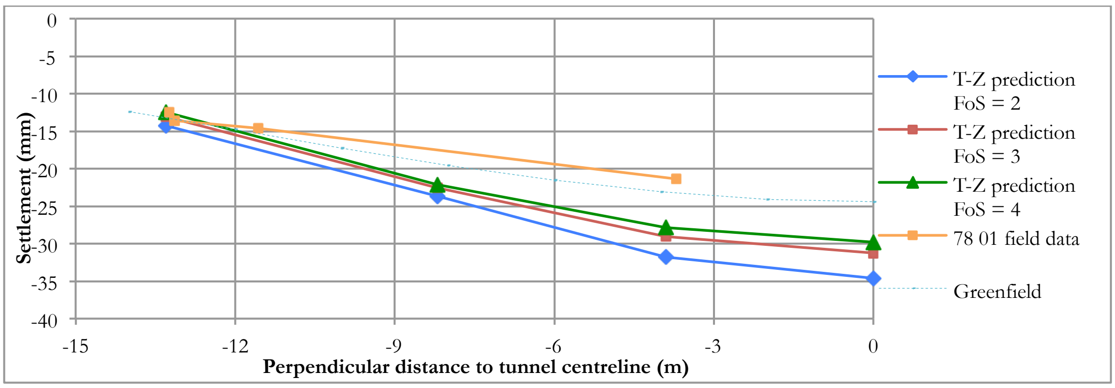
displaces vertically downwards, it would pull down on the slab, leading to an increase in bearing pressure until a new equilibrium state is established. If the bearing capacity of the slab is sufficient to resist this pressure, the overall displacement will be limited by the slab displacement. It is likely that the actual loads will be less than the design loads due to incorporated factors of safety, in which case the slab contribution will be more prominent and more load will be carried at shallow depth. This bearing capacity effect is likely to be of greater significance for the tunnels above and in close proximity to the tunnel centreline that are subject to greater displacements and high tension forces.



a) SM 01 *t-z* prediction



b) SM 02 *t-z* prediction



c) 78 01 *t-z* prediction

Figure 4.5: Effect of varying factor of safety (using tension cut-off)

In conjunction with this, the presence of a ground beam with reasonable structural stiffness allows for a redistribution of loads away from the pile centreline. This would result in a reduction in head load in the piles above and close to the pile centreline, which displace further with depth, and an increase in head load in the piles further from the tunnel centreline, which displace less with depth. As a result, the displacement close to the tunnel centreline would be less than expected, and greater than expected further from the centreline. This would explain the observations in Figures 4.4a and 4.5a for the section SM 01, where this exact behaviour appears to be taking place. The stiffness at this section is higher due to the building geometry, resulting in a more rigid response and supporting the likelihood of load redistribution. The extent of the load distribution is difficult to model in the single pile analysis; however, Yoo and Kim (2008) used finite element analysis to show this effect is feasible for stiff buildings.

4.3.5 Other factors

There are some other factors to also consider, which may contribute to the discrepancy between actual and predicted behaviour. Inherently, field data has a degree of error. Inaccuracies in the data could have resulted in a different volume loss being calculated to the true value and then being used in the *t-z* model. Reading errors could also result in direct discrepancy with the predicted results, however, the frequency of readings meant anomalies were easily identified and thus this is likely to have had little effect.

Since the tunnel was constructed in two phases, the possibility was considered that the predicted settlement modelled as a single phase construction with a volume loss of 0.9% may have been reduced compared to a two phase pilot and enlargement with volume losses 0.4% and 1.3%, respectively. In light of this, a number of piles were modelled in two phases and the combined overall settlement showed very little difference to the one phase settlement, thus it was deduced that this effect was negligible.

The true values of the parameters such as undrained shear strength may differ slightly to the design values used in the model. The shear strength profile was taken from the Crossrail Geotechnical Sectional Interpretative Report, and the design values are conservative. However, the results mentioned in Section 4.3.3 show that a slightly different soil strength profile is unlikely to cause any major changes to the predicted settlement, and thus this will not have significant impact.

In this analysis the pile-soil interface was modelled as being linear elastic - perfectly plastic (LEPP), whereas in reality, soil is non-linear. The LEPP approach provides a good estimate, however, the behaviour is likely to be better modelled with a non-linear approach, although it will be more computationally complex.

4.3.6 Comparison with Crossrail predictions

The values of the maximum pile head settlement predicted in the Crossrail Building Damage Assessment reports are displayed in Table 4.1. These values are based on a volume loss of 1.25% and are clearly very conservative. Although the *t-z* model over-predicts the settlement compared to the field data, it provides a far closer estimate; this modelling could provide a good initial building damage assessment due to its computational simplicity and speed, before more complex finite element is used for a thorough analysis.

	Crossrail predicted max. pile head settlement (mm)	*t-z* predicted max. pile head settlement (mm)	Field data max. settlement (mm)
The Smokery	37.0	29.9	26.2
78 Cowcross	36.0	29.1	21.4

Table 4.1: Crossrail predictions vs *t-z* predictions vs field data

4.4 Summary of *t-z* modelling

- The *t-z* model used in the previous analysis generally over predicts the pile displacement, particularly close to the centreline.
- The predominant factor in this difference in predicted and actual behaviour is thought to be the effect of the ground bearing slab, which provides extra bearing capacity and the ability to redistribute loads. These factors were not a feature of the single pile model and thus led to the over prediction of settlement.
- Other factors such as the tension cut-off and pile factor of safety have been shown to have noteworthy yet relatively small effects. The effect of soil stiffening is unknown and difficult to quantify, but simple analysis suggests the effect is not as prominent as that of the ground beam.
- The model provides a much closer estimate to the true behaviour than the conservative methods used by Crossrail. As such, this type of modelling would be very feasible for use in future when carrying out initial building damage assessments.

Chapter 5 Conclusions

The following conclusions can be drawn based on the previous chapters:

1. The interaction between tunnel construction and existing piles is complex and there are a number of factors that influence the behaviour. Traditionally, very conservative methods have been used to take account of this uncertainty when assessing building damage risks, resulting in protective measures that may have been unnecessary.
2. Field data analysed from the Crossrail project at Farringdon showed that piled buildings displace a distance similar to the greenfield settlement profile on the whole, and a slightly greater distance than the greenfield settlement close to the tunnel centreline. The buildings with a ground-bearing slab behaved with a degree of rigidity in both the longitudinal and transverse directions, suggesting pile group effects in conjunction with a higher stiffness were responsible for controlling the behaviour.
3. The analysis carried out was undrained, and therefore the load distributions are likely to change with drainage as the pore pressures change from those induced by tunnelling. This is likely to have the effect of reducing the slope of the trough therefore reducing the effect of differential settlement.
4. The *t-z* model for single piles generally over predicts the settlement compared to the field data, implying that the mechanism for pile groups is more complex than that for single piles, which provides a too simplistic approach.
5. The effect of tensile cracking was shown to have a relatively small influence on this difference in behaviour, although it is likely that the contribution is present. Tensile cracking is very likely in the piles close to the tunnel centreline given the magnitude of the soil displacements and the length of the tensile reinforcement in the piles. Pile factor of safety was shown to have a minimal influence.
6. The presence of the ground bearing slab was considered to be the main contributor in reducing the overall building settlement. This was due to both the effect of the slab bearing capacity limiting the displacement, in conjugation with pile load redistribution away from the tunnel centreline, indicated by a reasonably rigid building response.
7. The effect of soil stiffening is complex and the contribution of this effect is still relatively unknown; the greenfield settlements imposed on the pile are likely to be

smaller with soil stiffening effects, although it is thought the effect is less prominent than that of the ground beam.

8. The *t-z* model provides a suitable analysis for initial building damage assessments since it is computationally fast and produces a conservative estimate of the settlement. However, even 5-10mm difference in displacement will still make a reasonable difference to the deflection ratio and induced strains in the building, and could be the difference between damage categories and therefore the closer the true behaviour can be predicted, the less likely the need for over designed remedial measures will be. Ultimately, the more accurate the models are, the more confident the designers can be.

5.1 Suggestions for future research

- A thorough investigation into the soil stiffening effects of pile group interaction would be extremely beneficial, and being able to quantify the effects for different pile group scenarios would greatly increase the understanding.
- Incorporating the ground beam effects into the *t-z* model would allow the building behaviour to be modelled even more accurately. The author began work on this by incorporating a high stiffness value into the top nodes of the pile, and although not fully robust, early signs suggest this leads to a significant reduction in settlement.
- Using a non-linear approach rather than the linear elastic - perfectly plastic approach for the pile-soil interaction is likely to better model the behaviour.
- Implementing into the *t-z* model the cumulative distribution function to model the tunnel-induced settlement ahead of the tunnel face would allow pile settlement to be modelled at different stages of the tunnel advancement, in order to assess the building damage in the longitudinal direction and discover the critical construction stage. The model is capable of this with very little variation in the code.

Acknowledgements

I would like to thank my MEng supervisor Dr. C. Kechavarzi for his support and supervision throughout the year. I would also like to thank Dr. M. Williamson for his continual advice, expert knowledge and constant willingness to help, and for helping me drive the direction of this project. I am also grateful to Dr. M. Elshafie for his assistance, advice and guidance.

Appendix

Risk assessment review

A risk assessment was carried out at the beginning of the project, with the main risks being identified as computer based since the entirety of the project was data analysis and numerical modelling. The risks were considered throughout the project, and measures put in place that resulted in none being encountered.

References

Attewell, P.B. and Woodman, J.P. Predicting the dynamics of ground settlement and its derivatives caused by tunnelling in soil. *Ground Engineering*, 15(8): 13-36, 1982.

Attewell, P.B., Yeates, J. and Selby, A.R. *Soil movements induced by tunnelling and their effects on pipelines and structures*, 1986

Basile, F. Pile-group response due to tunnelling. In *Proceedings 7th International Symposium on Geotechnical Aspects of Underground Construction in Soft Ground*, pages 781-790, Rome, 2012.

Bezuijen, A and van der Schrier, J.S. The influence of a bored tunnel on pile foundations. In *Proceedings of the International Conference on Geotechnical Centrifuge Modelling*, pages 681-686, Singapore, 1994.

Chen, L.T., Poulos, H.G., and Loganathan, N. Pile responses caused by tunneling. *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, 125(3):207-215, 1999.

Chow, Y.K. Analysis of vertically loaded pile groups. *International Journal for Numerical and Analytical Methods in Geomechanics,* 10(1):59-72, 1986.

Coyle, H.M. and Reese, L.C. Load transfer for axially loaded piles in clay. *ASCE Journal of Soil Mechanics and Foundation Division*, 92(SM2):1-26, 1966.

Grant, R.J. and Taylor, R.N. Tunnelling-induced ground movements in clay. *Proceedings of the Institution of Civil Engineers: Geotechnical Engineering,* 143(1):43-55, 2000.

Hergarden, H.J.A.M., van der Poel, J.T., and van der Schrier, J.S. Ground movements due to tunneling: Influence on pile foundations. In *Proceedings of the 2nd International Symposium on Geotechncial Aspects of Underground Construction in Soft Ground*, pages 519-524, London, 1996.

Huang, M., Zhang, C., and Li, Z. A simplified analysis method for the influence of tunnelling on grouped piles. *Tunnelling and Underground Space Technology*, 24(4):410-422, 2009.

Jacobsz, S.W. *The effects of tunnelling on piled foundations*. PhD thesis, University of

Cambridge, 2002.

Jacobsz, S.W., Bowers, K.H., Moss, N.A., and Zanardo, G. The effects of tunnelling on piled foundations on the CTRL. In *Proceedings of the 5th International Symposium on Geotechncial Aspects of Underground Construction in Soft Ground,* pages 115-121, Amsterdam, 2005.

Klar, A. and Soga, K. The effect of ground settlements on the axial response of piles: some closed from solutions. Technical Report CUED/D-SOILS/TR341, Cambridge University, 2005.

Konda, T. Tunneling and excavation in soils. In *Proceeding of the 8th Asian Regional Conference on Soil Mechanics and Foundation Enginering, volume 2,* pages 259-283, Kyoto, 1987.

Kraft Jr, L.M., Ray, R.P., and Kagawa, T. Theoretical t -z curves. *ASCE Journal of the Geotechnical Engineering Division*, 107(11):1543-1561, 1981.

Lee, G.T.K. and Ng, C.W.W. Three-dimensional numerical simulation of tunnelling effects on an existing pile. In *Proceedings of the 5th International Symposium on Geotechncial Aspects of Underground Construction in Soft Ground*, pages 139-144, Amsterdam, 2005b.

Lee, C.J. Numerical analysis of the interface shear transfer mechanism of a single pile to tunnelling in weathered residual soil. *Computers and Geotechnics*, 42:193-203, 2012a.

Lee, C.J. Three-dimensional numerical analyses of the response of a single pile and pile groups to tunnelling in weak weathered rock. *Tunnelling and Underground Space Technology*, 32:132-142, 2012b.

Lee, C.J. Numerical analysis of pile response to open face tunnelling in stiff clay. *Computers and Geotechnics*, 51:116-127, 2013.

Loganathan, N., Poulos, H.G., and Xu, K.J. Ground and pile-group responses due to tunnelling. *Soils and Foundations*, 41(1):57-67, 2001.

Mair, R. and Taylor, R. Bored tunnelling in the urban environment. *In 14th int. Conf. SMFE, Hamburg*. 1997.

Mair, R.J. and Williamson, M.G. The influence of tunnelling and deep excavations on piled foundations, 2014

Martos, F. Concerning an approximate equation of the subsidence trough and its time factors, 1958

Mindlin, R.D. Force at a point in the interior of a semi-infinite solid. *Journal of Applied Physics*, 7(5):195-202, 1936.

Mroueh, H. and Shahrour, I. Three-dimensional finite element analysis of the interaction between tunneling and pile foundations. *International Journal for Numerical and Analytical Methods in Geomechanics,* 26(3):217-230, 2002.

Ng, C.W.W., Lu, H., and Peng, S.Y. Three-dimensional centrifuge modelling of the effects of twin tunnelling on an existing pile. *Tunnelling and Underground Space Technology*, 35:189-199, 2013.

O'Reilly, M.P. and New, B.M. Settlements above tunnels in the United Kingdom - their magnitude and prediction. In *M.J. Jones, editor, Tunnelling '82: Proceedings of the 3rd International Symposium*, pages 173-181, London, 1982.

Peck, R.B. Deep excavations and tunnelling in soft ground. *In Proc. 7th Int. Conf. on SMFE*, pages 225-290, 1969

Poulos, H.G., and Davis, E.H. Settlement behaviour of single axially loaded incompressible piles and piers. *Geotechnique*, 18(3):351-371, 1968.

Poulos, H.G. Analysis of settlement of pile groups. *Geotechnique*, 18(4):449-471, 1968.

Poulos, H.G. and Davis, E.H. *Pile foundation analysis and design*. John Wiley and Sons, 1980.

Poulos, H.G. Pile behaviour - theory and application. *Geotechnique,* 39(3):365-415, 1989.

Selemetas, D., Standing, J.R., and Mair, R.J. The response of full-scale piles to tunnelling. In *Proceedings of the 5th International Symposium on Geotechnical Aspects of Underground Construction in Soft Ground*, pages 763-769, Amsterdam, 2005.

Shahin, H.M., Nakahara, E., and Nagata, M. Behaviors of ground and existing structures due to circular tunneling. In *Proceedings of the 17th International Conference on Soil Mechanics and Geotechnical Engineering*, pages 1786-1789, Alexandria, Egypt, 2009.

Williamson, M.G, Elshafie, M.Z.E.B., and Mair, R.J. Centrifuge modelling of bored piles in sands. In *Proceedings of the 18th International Conference on SMGE, volume 2*, pages 981-984, 2013.

Williamson, M.G. *The effects of tunnelling on bored piles*. PhD thesis, University of Cambridge, 2014

Xu, K.J. and Poulos, H.G. 3-D elastic analysis of vertical piles subjected to passive loadings. *Computers and Geotechnics*, 28(5):349-375, 2001.

Yoo, C. and Kim, S.B. Three-dimensional numerical investigation of multifaced tunneling in water-bearing soft ground. *Canadian Geotechnical Journal,* 45(10):1467-1486, 2008.

Zhang, R.J., Zheng, J.J., Zhang, L.M, and Pu, H-F. An analysis method for the influence of tunneling on adjacent loaded pile groups with rigid elevated caps. *International Journal for Numerical & Analytical Methods in Geomechanics*, 35:1949-1971, 2011.