University of Cambridge

Improving engineering knowledge from Crossrail: Investigating the effect of building stiffness on pile displacement due to tunnelling works.

by

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Fourth-year undergraduate project in Group D,
2014/2015

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

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This report contains less than 12000 words and consists of 50 pages as required, not including the title page and technical abstract.

Abstract

Rapid urbanisation and an ever rising population has led to a demand for the construction of increasingly complex urban infrastructure. Dense urban environments lead to surface space constraints, resulting in new infrastructure becoming taller and also a movement to develop infrastructure underground. Tunnels of various uses have become omnipresent in the urban world of today with many major tunnelling projects having occurred in recent times.

Tunnelling effects cause the ground to be subject to stress relief which propagates through the soil to the ground surface. This has consequences for buildings and foundations that rest in the vicinity of tunnel constructions. It is important to understand the interactions and mechanisms involved between the tunnel, soil, foundation and building such that these effects may be better understood and steps can be made to alleviate them. Much research has already been conducted in this area, primarily focusing on building damage and tunnelling effects on shallow foundations. Less research has been reported on deep, piled foundations. Where such research exists, it tends to focus on tunnelling effects adjacent to piles and the effects of pile bending; although some research has also been conducted for tunnelling directly beneath piles. Research also has a propensity to ignore discussion of the actual mechanisms involved due to tunnelling that helps to improve understanding.

With the growing need for geotechnical infrastructure, congestion of subsurface space and increasingly deep, piled foundations to satisfy larger surface structures, tunnelling beneath piles is becoming more prevalent. Consequently, this project focuses on the tunnelling effects beneath group pile foundations in clay, with an attempt to model these effects in undrained conditions using finite element modelling that will ultimately improve understanding of this phenomenon.

Of particular focus in this investigation was in the effects of building stiffness on tunnelling-induced settlements, with further analysis into the material properties of the piles, the effects of pile cracking and the influence of horizontal tunnel position on the behaviour of group pile foundations. Finite element analysis was conducted in two-dimensions, modelling a building at Kempton Court, Whitechapel, that was subject to tunnelling effects during the construction of Crossrail, London. The aim was to attempt to replicate the observations made from extensive field monitoring of the building during the Crossrail project in the finite element models, in order to improve understanding of the tunnel-soil-building interaction mechanisms involved.

In the two-dimensional analysis, the settlement magnitudes for the case with a building present were observed to be larger than those seen under greenfield conditions. In

comparison, field observations showed that the buildings settlements closely followed the greenfield surface settlements. The higher predicted settlements from the finite element model were suggested to be caused by the effects of compensation grouting not being included in the model, where they were used extensively during tunnel construction.

Building stiffness was shown to have a profound effect on soil surface settlements, with smaller settlements observed for a stiffer building and foundation and the converse for a less stiff building. For high stiffness values, the building was shown to settle similar to that of a rigid punch solution; whereas for low stiffness values, a flexible raft settlement solution was observed. The effect of pile cracking was shown to be relatively small, with a change in the distribution of loads throughout the structure observed for the case when pile cracking occurred, rather than large changes in tunnelling induced settlements. Horizontal tunnel position appeared to have little effect on the building settlement response when the tunnel axis lay within the area of the building; outside this area, settlements tended rapidly towards greenfield values suggesting a flexible building response to tunnelling.

The three-dimensional analysis reinforced the settlement observations made in the two-dimensional analysis, with lower settlements modelled for a stiffer structure, which again settled as a rigid punch foundation. The deflection ratio induced in the building hardly differed for the different stiffness values analysed, suggesting building stiffness has a limited effect on the differential settlements of the building. Bending moment calculations in the piles suggested that pile cracking should not be an issue, with the highest bending moments induced in the piles directly above the centreline of the tunnel, or at the point of inflection of the settlement trough, depending on the direction of bending.

The results obtained from the finite element analysis reinforced many of the theoretical and field observations made in the literature and the case study of Kempton Court. Building stiffness was successfully modelled to show the effects of tunnelling induced settlements of Kempton Court, with the similarities observed between the different scenarios leading to an improved understanding of the tunnel-soil-building interaction problem.

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May 2015

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Chapter 1

Introduction

1.1 Background

The aim of the research is to investigate tunnelling effects on structures with piled foundations. Of particular focus is the influence of building stiffness on tunnelling induced settlements. As urban environments continue to expand and become increasingly complex, there is a need to develop infrastructure. Much of this infrastructure is being constructed underground due to the high demands for space above the ground surface. Construction has accelerated at such a pace, that there is an increasing need for geotechnical structures to be built in the vicinity of existing structures; whether the piled foundations of surface structures, or other geotechnical structures.

Tunnelling effects on existing structures represent an important area of understanding in geotechnical engineering. Tunnelling causes stress relief to the surrounding soil which propagates through the soil to the ground surface causing surface settlement troughs, both longitudinal and transverse to the tunnelling direction. Such movements must be fully understood so that settlements do not cause significant damage to existing structures. Much research has been conducted analysing tunnelling effects on buildings as well as extensive monitoring of field data and case histories of major tunnelling projects, providing data from which theories and models have been developed. However, there exists a problem that much of this data is poorly understood, which consequently leads to high factors of safety and intensive over-design of many geotechnical projects. It is increasingly becoming accepted that a better understanding of the mechanisms involved is needed, such as the tunnel-soil-pile interaction. Such understanding has, in the past, been gained through the use of computer modelling; notably finite element analysis in two or three dimensions. However, there exists a disparity and discrepancy between much of the major finite element research conducted to date, with few coherent or consistent observations made.

Current international tunnelling projects such as Crossrail, London, provide enormous quantities of data that can be used to help achieve an improved understanding. Extensive, high-technology monitoring systems utilised before, during, and after tunnelling projects can act as an impetuous for finite element analysis to be conducted, that accurately and reliably model the phenomena observed. The aim of this project was to use finite element modelling in both two and three dimensions to try to model observations made at Kempton Court, Whitechapel, which was subject to tunnelling effects during Crossrail tunnel construction, and to use the models to improve understanding of the mechanisms involved when considering tunnelling effects on piled structures.

1.2 Project Outline

The following outlines the layout of this report:

- Chapter 2 references and summarises key literature relating to tunnelling effects on piles.
- Chapter 3 describes the principles behind finite element analysis and provides insight into the theory used in the finite element software.
- Chapter 4 presents the analysis and results from two-dimensional finite element modelling in Safe, with comparisons drawn between the computer analysis and field observations.
- Chapter 5 presents the analysis and results from three-dimensional analysis in LS-Dyna.
- Chapter 6 summarises the mechanisms observed during the analysis and presents the conclusions drawn from the analysis conducted.

Chapter 2

Literature Review

2.1 Introduction

The effect of tunnelling in soil results in changes to the in-situ stress field around the tunnel excavation. The stress relief causes the soil to move towards the tunnel. This can occur in both transverse and longitudinal directions (perpendicular and parallel to the tunnel advancement respectively). The soil settlements around the tunnel propagate through the soil strata such that all soil within the tunnel influence zone is subject to movement. This has the most dramatic effect on the surface, where a surface settlement trough is experienced as shown in Figure 2.1.

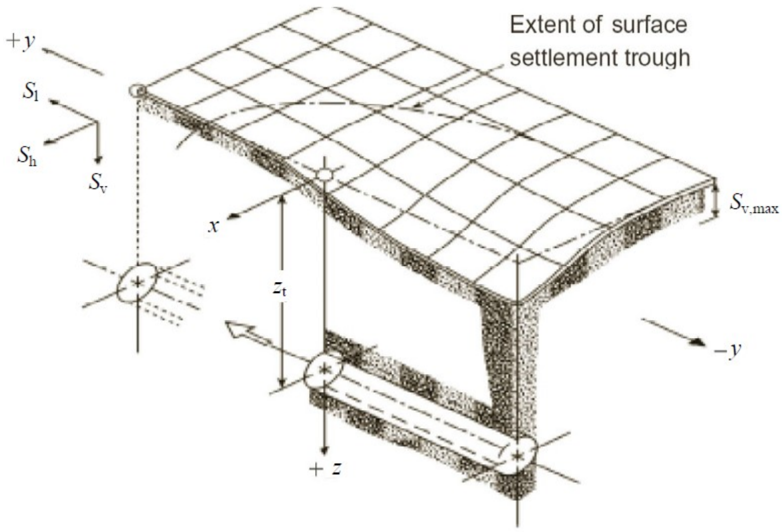


Figure 2.1: Assumed settlement surface profile, after Attewell et al. [1986]

Mair and Taylor [1997] stated that the vertical and horizontal ground movements associated with tunnelling can be attributed to five different components of tunnel construction:

1. Deformation of the ground towards the face resulting from stress relief

2. Passage of the shield: the presence of an over-cutting edge (bead) combined with any tendency of the machine to plough or yaw will lead to radial ground movements.
3. Tail void: Existence of a gap between the tailskin of the shield and the lining means that there will be a tendency for further radial ground movements into this gap. Tailskin grouting of the void is often employed to limit this effect.
4. Deflection of the lining as ground loading develops.
5. Consolidation: as pore water pressures in the ground change to their long-term equilibrium values the associated changes in effective stress lead to additional ground movements.

Consolidation effects are usually ignored to make settlement predictions more conservative. The exact nature of the stress relief is dependent on the soil material in which the tunnel is constructed. A majority of research in this area uses the assumed undrained constant volume behaviour of clays due to the complexity in modelling the non-constant volume behaviour of sands.

Greenfield settlements are taken as those occurring without interaction from nearby structures or foundations and such settlements form the basis of soil settlement analysis. The surface settlement trough can be analysed in more detail, revealing some standard behaviours. Vertical transverse surface movements in clay take the form of a Gaussian settlement trough, first proposed by Martos [1958]. In sands, the settlement trough has been shown to be much narrower than for clays; therefore, a modified Gaussian settlement profile was suggested by Vorster et al. [2005] which has been shown to provide a better approximation to tunnelling in sands. The modified shape is shown in Figure 2.2. The longitudinal settlement trough is assumed to be the shape of a cumulative probability curve, first proposed by Attewell and Woodman [1982], shown graphically in Figure 2.3.

When tunnelling in the vicinity of existing geotechnical structures, it is interesting to analyse the effect tunnelling has on the behaviour of the structures, especially when considering piled foundations. According to Tomlinson and Woodward [2014]: "piles are relatively long and slender members used to transmit foundation loads through soil strata of low bearing capacity to deeper soil or rock strata having a higher bearing capacity". It is therefore important to understand the behaviour of piles subject to tunnelling, in particular pile settlement, induced pile forces and pile failure. Piles tend to transfer loads to the ground through two principle methods (from Tomlinson and Woodward [2014]):

- End-bearing: if the bearing stratum for foundation piles is hard and relatively impenetrable material such as rock or a very dense sand and gravel, the piles derive most of their bearing capacity from the resistance of the stratum at the toe of the piles.

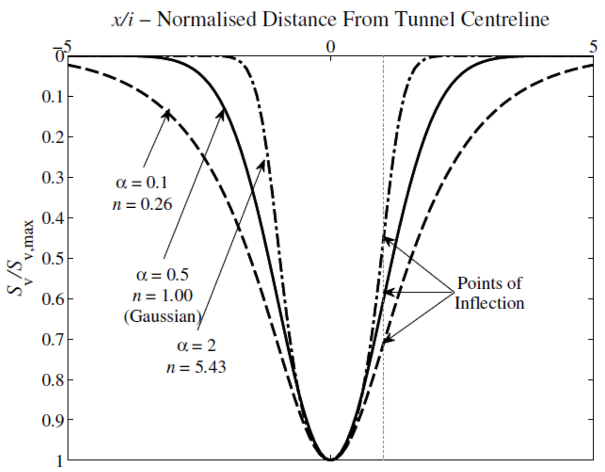


Figure 2.2: Modified Gaussian Settlement Trough, after Vorster et al. [2005]

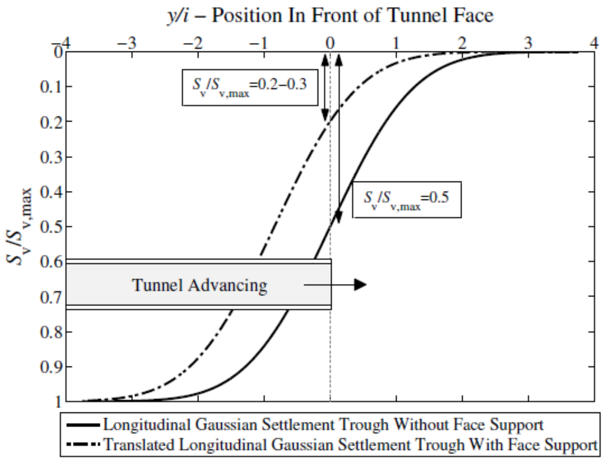


Figure 2.3: Longitudinal Settlement Trough, after Attewell and Woodman [1982], Mair and Taylor [1997]

- Friction piles: piles carrying capacity is derived partly from the end-bearing, and partly from the skin friction or adhesion between the embedded surface of the pile and the surrounding soil, which represents the greater part of their carrying capacity.

Previous work investigating the effects of tunnelling on piled foundations has been summarised by Mair and Williamson [2014] in seven categories. This literature review focuses on various Field Trials, Case Studies, Centrifuge Models and Finite Element Models on tunnelling effects on pile foundations.

2.2 Effect of Tunnelling on Piles

2.2.1 Field Trials

2.2.1.1 Second Heinenoordtunnel (Beneath)

Kaalberg et al. [2005] conducted full scale field trials on the Second Heinenoordtunnel. The field trials were conducted in strata consisting of *4m* of soft clay on top of a layer of fine sand. The tunnels had a diameter of *8.3m*, centre spacing of *16.3m* and a cover between *12-13m*. The research investigated two modes of deformation: settlement of the soil layer around the pile toe and settlement due to stress relief around the pile toe. Tests were carried out with single piles, pile pairs and pile groups, driven into the ground between $0.25D_t$ (diameter) and $2.5D_t$ distance from the tunnel. The significant conclusions were that: there were significant movements due to stress relief only at a pile toe distance within $0.25D_t$ of the central tunnel axis, the tunnel settlement trough was much steeper than expected and pile settlements were shown to be greatest when tunnelling occurred directly beneath the piles, rather than adjacent. Piles located outside the diameter of the tunnel with the pile toe cap above the tunnel crown showed settlements similar to the soil surface. Figure 2.4 shows the influence of pile toe location (both vertically and horizontally) on the induced settlement. A brief description of the zones referenced is as follows:

1. Zone A: piles will move equal to, or slightly more, than the surface level settlement.
2. Zone B: piles will move equal to the surface settlement.
3. Zone C: piles will move less than the surface level settlement.

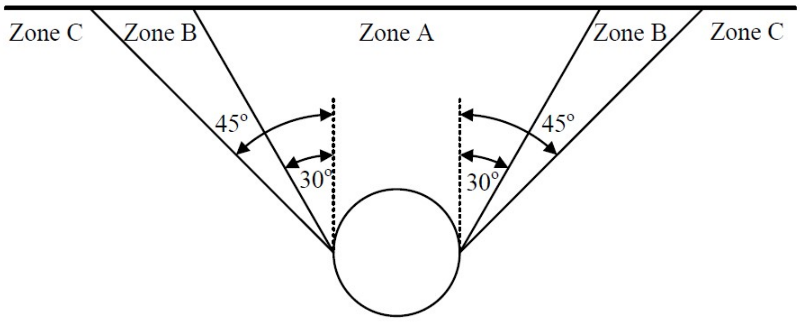


Figure 2.4: Dutch Field Trial Pile Toe Influence Zone, Kaalberg et al. [2005]

2.2.1.2 Channel Tunnel Rail Link (Beneath)

Effects of tunnelling beneath piles were investigated at a field test site, reported by Selemetas [2005] and Selemetas et al. [2005]. The tunnels investigated were of outside diameter, *OD=8.15m*, centre distance of *16.0m* and a depth of *18.9m* to tunnel axis. Two friction piles and two end-bearing piles were constructed. The most significant conclusions drawn were that piles directly above tunnel showed much greater displacement than those at a distance $1.0D_t$ and even more than at a distance $2.0D_t$ and that the effect of tunnelling on friction piles was less significant than end bearing piles. The results of the CTRL field trial are summarised in Figure 2.5, in a similar fashion to that of Kaalberg et al. [2005].

2.2.2 Case Studies

2.2.2.1 CTRL Bridges (Beneath)

Jacobsz et al. [2005] reported the effects of the CTRL tunnel construction on piled bridges along the tunnel alignment. The design was based on Jacobsz et al. [2004] which predicted that end-bearing piles with pile toes in the zone of influence would move roughly the same as the greenfield settlement in both sands and clays for small volume losses, up until the point at which maximum shaft friction was mobilised, in which case the piles would undergo large displacements.

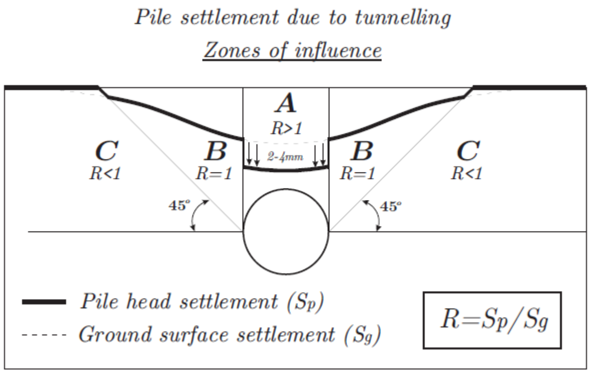


Figure 2.5: CTRL pile toe influence zone, Selemetas [2005]

For predominantly friction piles, Jacobsz et al. [2005] alludes that the shaft friction exhibits large stresses on the surrounding soil, thus affecting the soil stiffness. This induces subsurface movement profiles for tunnels which differs around piles within the tunnels' zone of influence. It is postulated that subsurface settlement would be more uniform with depth due to the stiffness increase close to the piles, with settlement nearly the same as the greenfield surface settlement. The tunnels in this scenario were constructed beneath three bridges, founded on piles. The friction pile cases are of particular interest in this review. At the Renwick Road Bridge (end-bearing piles) the pile base settlements were accurately estimated using New and Bowers [1994] and Jacobsz et al. [2005], which showed the pile bases following the greenfield settlements. The piles themselves were shown to settle less than the ground surface when not in the zone of influence, but similar to the pile bases within the zone of influence, as predicted.

At the Ripple Road Flyover (Shaft Friction Piles), the piles were situated directly above the tunnels. It was expected to see a large reduction in the pile base loads. The settlements due to both tunnels were similar to those of the greenfield settlement.

At the A406 Viaduct (Shaft Friction Piles), the piles were assumed to be shaft friction piles, with only a small base load being mobilised. The pile settlements were again similar to greenfield values, with the amount of settlement varying depending on the pile position relative to the tunnel.

The study showed that the effect of tunnelling beneath piles at all locations gave pile displacements similar to that of the greenfield displacements. For the friction controlled pile displacements, it may have been expected to see slightly larger displacements, however, the pile caps bear directly onto Terrace Gravel, which may have contributed to the relatively low pile displacements.

2.2.2.2 Crossrail running tunnels, UK

Twin running tunnels of *7.1m* OD were constructed in London Clay as reported in Mair and Williamson [2014]. Measurements were made of the greenfield ground surface settlements and settlements of the piled buildings. The greenfield measurements showed that the Westbound tunnel, constructed first, gave a volume loss of *0.46%*; the Eastbound tunnel, constructed subsequently, gave a volume loss of *1.02%*, with corresponding trough width parameter values, K, of 0.42 and *0.59*. The observed building settlement was similar to the greenfield ground surface settlement, as displayed in Figure 2.6.

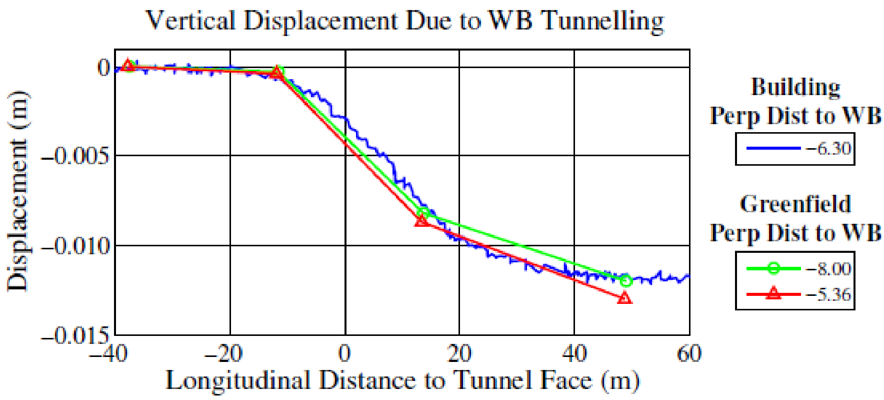


Figure 2.6: Comparison of observed settlement of 4-18 BBR piled building and greenfield ground surface settlements caused by westbound construction for Crossrail, Mair and Williamson [2014]

2.2.2.3 Crossrail station tunnels, UK

Station tunnels from Crossrail were constructed using sprayed concrete linings such as those beneath Kempton Court (KC). Figure 2.7 shows the layout of the KC piled building in relation to the tunnels. The building consists of two separate four storey structures constructed using load bearing masonry, highly susceptible to tensile cracking if large differential settlements were to occur. Figure 2.8 shows a cross-section through one of the *10.7m* diameter platform tunnels, constructed in London Clay, and the buildings' pile foundations.

The observed greenfield volume losses for the pilot tunnel and subsequent enlargement were *1.29%* and *1.22%* respectively. The observed building settlement profile was very similar to the greenfield ground surface settlement profile as shown in Figure 2.9, which also shows how there is little difference in the shape and magnitude of the settlement trough.

2.2.2.4 Further case studies

Teparaksa et al. [2006] reported on the Bangkok Flood Diversion Tunnel (Beneath) where

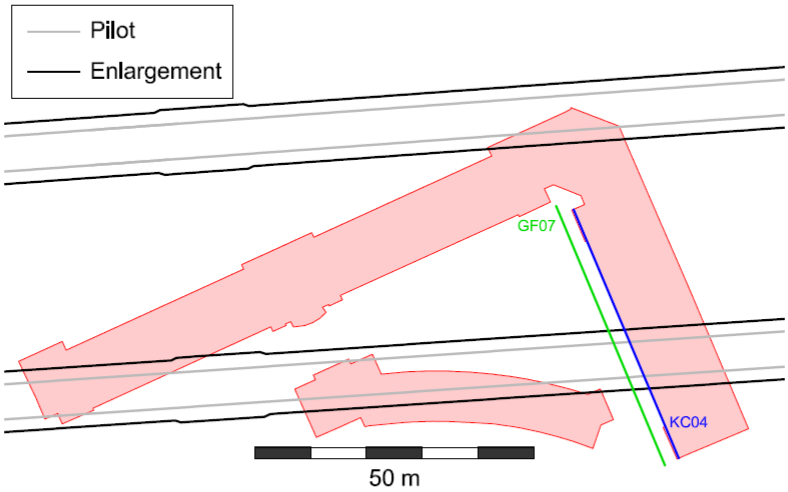


Figure 2.7: Layout of piled building KC in relation to station tunnels on Crossrail projects, Mair and Williamson [2014]

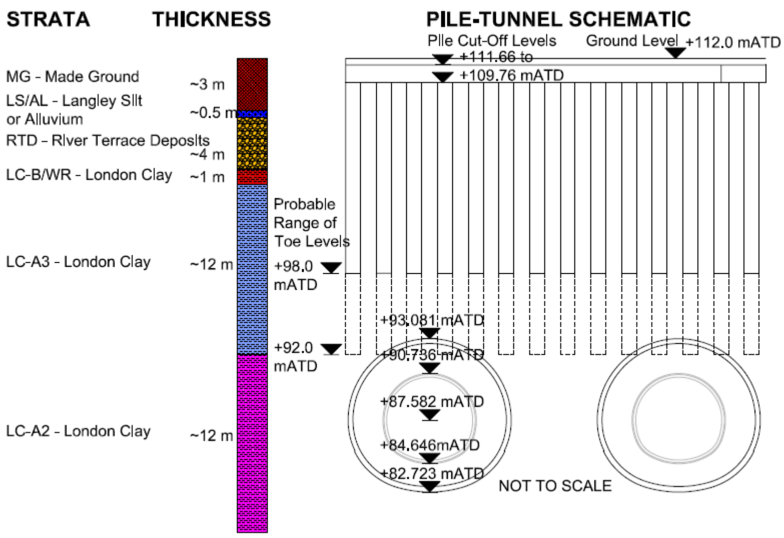


Figure 2.8: Tunnelling beneath KC piled buildings for Crossrail, London, Mair and Williamson [2014]

the largest settlements observed were directly above the tunnel centreline. Phienwej et al. [2006] reported on the construction of the First Bangkok Subway Line (Beneath and Adjacent), where displacements were found to drop off significantly with increasing distance from the tunnel centreline to practically zero displacement at 30m. The effect of the piled foundations is to distort the greenfield settlements and to bound the settlement trough within the piers.

2.2.2.5 Conclusions drawn from Field Trial and Case History Literature

- The effect of pile lateral position, x_p, and pile vertical position, C_p, in relation to the tunnel is shown to have a significant effect on the vertical pile displacements.

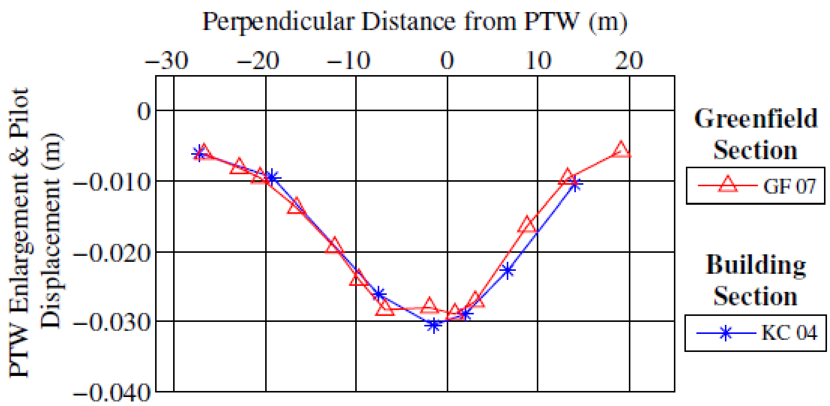


Figure 2.9: Comparison of observed settlement of KC piled building and greenfield ground surface settlements caused by tunnel construction for Crossrail station, London, Mair and Williamson [2014]

Piles with x_p=*0* are shown to generally exhibit the greatest vertical displacements; those at greater x_p exhibit lower displacements.

- Piles with bases beneath the tunnel axis generally show lower vertical pile head displacements than those with pile toes above the tunnel axis.
- The effect of pile head condition appears to have some effect on the vertical pile displacements. Piles with caps founded on less competent soils exhibit greater relative vertical pile displacement than those founded on more competent material.

2.2.3 Centrifuge Models

2.2.3.1 Delft Centrifuge Studies (Beneath)

Bezuijen and van der Schrier [1994] and Hergarden et al. [1996] describe centrifuge tests to investigate the effects of tunnelling beneath piles, also reported in Mair and Williamson [2014]. The following conclusions were summarised by Mair and Williamson [2014]: the stress reduction due to the tunnel cavity was the main cause of settlement and reduction in base capacity; generally large settlements occur when the pile toes are within *0.25* to *1.0* D_t of tunnel extrados in any geometric configuration and settlements of piles with toes greater than *2.0* D_t from the tunnel showed little displacement.

2.2.3.2 UWA/ University of Sydney Centrifuge Tests (Adjacent/ Beneath)

Loganathan et al. [2000] presents the results of three centrifuge tests carried out by Randolph et al. [1991]. The tests involved changing the location of the tunnel in relation to fixed pile locations. The tunnel was *60mm* diameter constructed in Specswhite Kaolin in undrained conditions, with the piles having a *9mm* OD. The main conclusions drawn were as follows: the maximum displacement occurring for the pile toe at the tunnel axis level is in agreement with Hergarden et al. [1996] and the effect of the pile group was generally to reduce settlement and induced loads, indicating that a single pile may be considered a worst case scenario for assessing the effects of tunnelling piles.

2.2.3.3 Williamson [2014]

Williamson [2014] reported on centrifuge tests modelling the effects of tunnelling beneath bored piles in clay. A greenfield package was initially used to investigate the detailed ground movement mechanisms for the tunnel construction in stiff clay, producing the following conclusions:

1. Piles above the tunnel centreline in response to tunnel volume loss show displacements greater than the soil surface settlement at the pile head.
2. Piles show small changes in load when subject to tunnelling induced settlements.
3. Pile failure does not occur, even at high volume losses, with piles experiencing increased settlement so that sufficient skin friction is generated to maintain equilibrium.

2.2.3.4 Discussion of centrifuge tests

Centrifuge tests have proven to replicate many of the results seen in both the Field Trials and Case Histories. Most notably, the fact that surface settlement is greatest when directly above the tunnel. However, there are some limitations to the centrifuge tests discussed, mainly the lack of testing on group piles. Therefore, the understanding of group piled structures is relatively under-researched in terms of centrifuge.

2.2.4 Finite Element Modelling (FEM)

2.2.4.1 Mroueh and Shahrour [2002] 3D FEM (Beneath/ Adjacent)

Mroueh and Shahrour [2002] used 3D FEM to investigate the interaction between tunnelling and piles. The model incorporated an elastic perfectly-plastic constitutive soil model based on the Mohr-Coulomb criterion with a non-associative flow rule, and analysed the behaviour of the pile during construction and loading, prior to the tunnel construction. It then ran the same analysis both during and after the tunnel construction phase. The main conclusions were that the worst position of the tunnel, relative to the pile for induced axial force and bending, was when the pile toe was close to the tunnel axis level. Investigations into the behaviour of group piles showed that they were less affected by axial load and bending in comparison to single piles, as the interaction between grouped piles caused an increase in stiffness of the ground around the group of piles.

2.2.4.2 Yoo and Kim FE Studies (Beneath)

Yoo and Kim [2008] used FEM to model the behaviour of a piled structure with a surcharge acting on the ground surface. The key results from their analysis were that the presence of piles reduces the soil settlement and causes the surface and subsurface to behave in a very similar manner. In addition, pile presence caused a reduction in the maximum settlement, along with the maximum slope of the settlement trough. There was also seen to be a reduction in settlement and a redistribution of loading when group piles were analysed. The model showed significant load reduction and loss of pile capacity for piles located directly above the tunnel axis with this load distributed to other piles.

2.2.4.3 Lee FE Studies (Beneath)

Lee [2012] and Lee [2013] used FEM to look at tunnelling beneath both single and grouped piles in London Clay. Both showed significantly larger displacements of the pile than the greenfield soil. There was also greater displacement for pile groups than for single piles.

2.2.4.3.1 Discussion The results obtained by Lee contradict many of the major FE studies conducted, showing increased settlement when piles are present and that group piles behave negatively in comparison to single piles. This contradiction with other studies could be linked to the fact that the entire pile series, even for grouped piles, lay within the tunnel footprint, suggesting that the grouped piles behaved in a block manner with regards to settlement and axial forces. There appears to be a variation in behaviour of pile groups above a tunnel, and those where the groups extend much further afield; however, further investigations are needed to fully understand these contradictions.

2.2.5 Estimating building stiffness

Elshafie [2008] summarised different approaches to evaluating building stiffness. The methods break down a building into individual structural components in a number of different ways. The stiffness of the components can be summed together to provide an approximation for the stiffness of the whole structure. Melis and Rodriguez Ortiz [2001] proposed a method by looking at the stiffness contribution from the floor, walls and basement, which are summed together to give the building stiffness as in Equation 2.1, where the moment of inertia of the floor slabs is relative to the structures' neutral axis.

$$(EI)_{building} = \sum (EI)_{floors} + \sum (EI)_{walls} + \sum (EI)_{basement} \tag{2.1}$$

2.2.6 Discussion of Literature

The studies from the respective Field Trails, Case Histories, Centrifuge Testing and FEM have all investigated the effects of tunnelling on piled foundations. Tunnelling works have shown to result in vertical and horizontal pile displacements and changes in axial bending in the piles. The Case Histories and Field Trials show that piles with their toes above the tunnel axis exhibit the greatest vertical displacements, an observation reinforced in the Centrifuge and FEM studies that have been conducted.

Similar results have been drawn for pile axial loads and bending moments which are greatest when the piles are located above the tunnel axis. Finally, it has been shown that pile groups tend to display positive behaviour in terms of displacements and loading when compared to single piles, with the exception being the analysis carried out by Lee [2013].

2.3 Problem Statement

There exists limitations in our understanding of the mechanisms of subsurface tunnel-soil-pile interactions and more research is needed to understand the exact benefits of pile groups. Case Histories and Field Trials are able to provide data for comparison with physical and numerical models, but it has proved challenging to fully replicate this data in centrifuge and computer models. It is important to be able to replicate the results seen in Field Trials, Case Histories and Centrifuge in FEM in order to try to improve the understanding of the mechanisms at work. Furthermore, more understanding is required of the significance of various parameters involved in pile behaviour due to tunnelling effects including, but not limited to: pile safety factors, pile stiffening, pile location, pile spacing, group pile behaviour, material properties and structural load redistribution.

Therefore, this project investigates the effect of building stiffness on pile settlement due to tunnelling works, using both 2D and 3D FEM in order to ty to replicate observed data from Crossrail and improve understanding of the data observed.

2.4 Aims

The aim of this project is to model the effects of tunnelling induced settlements on piled structures in undrained conditions using FEM, making comparisons with a previously conducted case study on Kempton Court in order to help improve understanding of the tunnel-soil-pile interaction and mechanisms due to tunnelling.

The aim will be satisfied by the following objectives detailed in the flowchart displayed in Figure 2.10, with particular focus on the following.

1. To replicate the greenfield data and building displacements observed in both 2D and 3D FEM, using case study data at Kempton Court.

2. Investigate how changing the building stiffness parameter affects pile behaviour, with particular focus in changes in observed settlements.

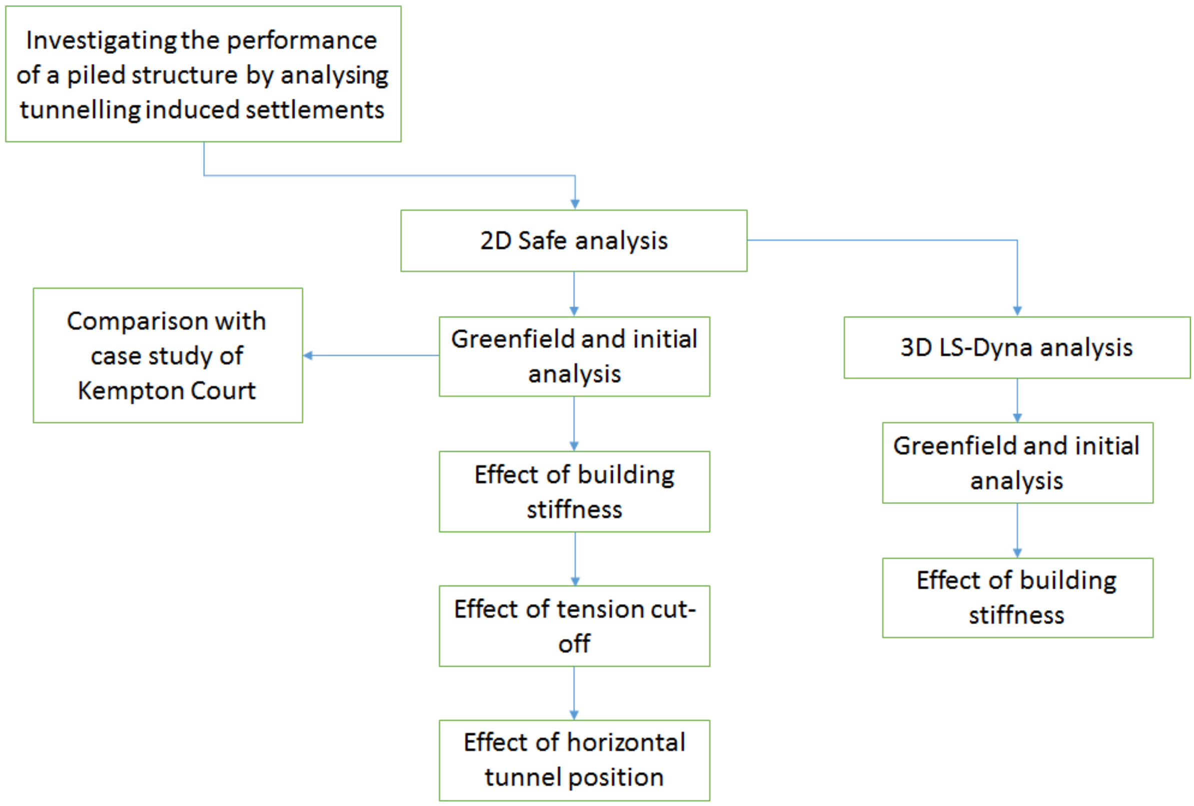


Figure 2.10: Flowchart of project research aims

Chapter 3

Finite Element Modelling Theory

3.1 Two-dimensional modelling: background model information

3.1.1 Modelling in Safe

Safe is a finite element (FE) software capable of performing two-dimensional FE computations for geotechnical problems. It is primarily used such that the user can study soil stresses, strains and deformations from plane stress, plane strain or axially symmetric problems.

Following FE theory, the problem area is discretised into a mesh of finite elements which create a modal for analysis. Each element is built up, and contributes, to an overall mesh comprised of many small elements. In Safe, each element is an 8-node "isoparametric" element; essentially a quadrilateral element with eight nodes; one at each corner and one at the centre-point of each edge. This is depicted in Figure 3.1. The stresses and strains are computed at Gauss points within the elements. Gauss points are the sampling points from which Safe integrates over the area of the element.

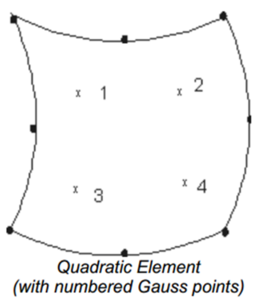


Figure 3.1: Quadratic element with number Gauss points, Oasys [2009]

The program uses a number of constitutive soil models for analysis purposes, including linear elastic, Mohr-Coulomb, modified Cam Clay and BRICK. For the KC model, analysis of plane strain is conducted, using the BRICK model introduced by Simpson [1992].

3.1.1.1 Constitutive Soil Model: 2D BRICK

BRICK is a plane strain model published by Simpson [1992] and Simpson [1993], initially developed to capture stress history effects in London Clay. A 3D version was subsequently developed and reported by Lehane and Simpson [2000]. The BRICK concept was developed based on three key observations of soil behaviour:

1. The stiffness of a soil decreases with accumulation of shear strain.
2. Plastic strains initially tend to continue in the same direction as the approach path after a change in stress path orientation.
3. A soils capacity to accept further normal and shear stresses will be extended as mean normal stress is increased.

Points 1 and 2 are tackled by Simpson through the introduction of an S-curve as shown in Figure 3.2. The curves attempt to model how soil stiffness decreases with increasing shear strain, whilst allowing the stress history of soil to be included in the model. Simpson described the formulation of the S-curve through an analogy of a man pulling bricks on strings. This analogy was paraphrased by Ellison [2012] and quoted here with the concept shown schematically in Figure 3.3:

> *"The person and bricks are situated in a transformed coordinate system comprised of a volumetric strain axis and multiple shear strain axes. The "bricks" represent different portions of the material behaviour and are connected to the current strain point by "strings" with different lengths. The direction and accumulation of plastic strains for each portion of the material is identical to the translation of its brick."*

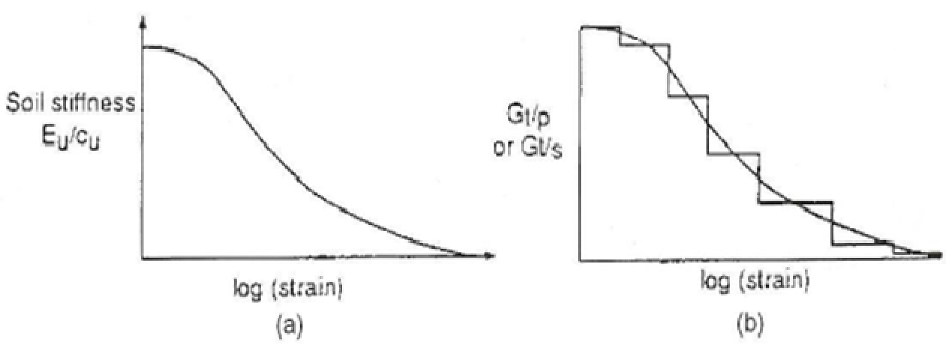


Figure 3.2: The S-shaped curve in terms of (a) ratio of Youngs modulus to undrained shear strength and (b) ratio of the tangent shear modulus to mean normal stress (Simpson [1993]

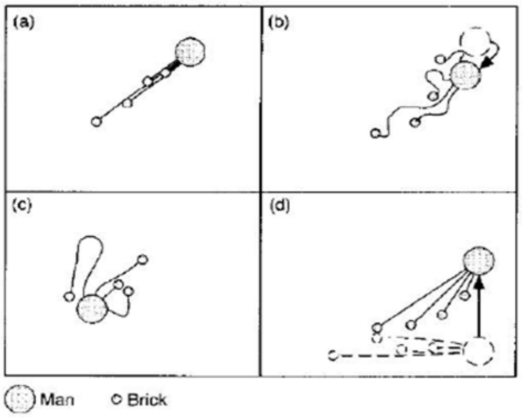


Figure 3.3: Possible scenarios for the analogue of a man pulling bricks around a room (Simpson [1992]

The analogy essentially states that when the man changes direction, the brick will also change direction. Simpson [1992] likened this to the change in strain vectors due to a change in stress path.

The BRICK coordinate system operates in pure strain space, unlike the majority of soil constitutive models that operate in some form of stress-strain space. The three axes are: volumetric strain, pure shear strain and simple engineering shear strain. Referring back to the “man” and “bricks” analogy, the location of the “man” represents the current soil strain state, with the “bricks” movement representing the change in plastic strains due to a change in stress load path. Therefore, for a given strain increment, summing the contribution of each “brick” produces the corresponding plastic strain increment. Essentially, this involves summing the product of the material proportion and the translation vector for each “brick”. Thus, when all the strings are loose, the material behaviour is entirely elastic; when all the strings are taut, plastic strains may develop.

3.1.1.2 Preliminary set-up of Safe model

The model is built up in four stages, denoted as Runs 1 to 4, in undrained conditions. The following section outlines the procedures involved in each Run. Although a 2D model, the mesh is multi-layered, with two layers superposed on top of each other, meaning there are a number of hidden elements not displayed to the user in the graphical view. A summary of all the different material types and their properties are summarised in Figure 3.4. These should be consulted in reference to input data described in Runs 1 to 4.

Run 1: Initialise sets up the initial conditions. Crossrail’s Geotechnical Interpretive Report (Group [2010]) showed the soil at KC to be Made Ground, overlying River Terrace Deposits, overlying London Clay, in which the tunnels are located. The ground level is at *111.9mATD*, although this taken to be *112mATD* in the FE model. Table 3.1 summarises how the soil stratum thicknesses were reflected in the mesh.

	Colour	Description	Type	Deform. mode	Drainage condition	Total weight	Isotropic	E1	v1
No		Material				kN/m^3		kPa	
1		MG	Elastic Mohr/Coulomb	PL.STRN	Drained	19	Yes	1.00E+04	0.2
2		RTD	Elastic Mohr/Coulomb	PL.STRN	Drained	20	Yes	2.50E+04	0.25
3		A3	Elastic Mohr/Coulomb	PL.STRN	Drained	20	Yes	1.80E+04	0.2
4		Void	Water or Void	PL.STRN	No water	0	Yes	n/a	n/a
5		MG Slip	Elastic Mohr/Coulomb	PL.STRN	Drained	19	Yes	1.00E+04	0.2
6		RTD Slip	Elastic Mohr/Coulomb	PL.STRN	Drained	20	Yes	2.50E+04	0.25
7		A3 Slip	Elastic Mohr/Coulomb	PL.STRN	Drained	20	Yes	1.80E+04	0.2
8		Pile	Linear Elastic	PL.STRN	No water	20	Yes	4.76E+06	0.2
9		Facade	Elastic Mohr/Coulomb	PL.STRN	No water	1.475	Yes	2.30E+05	0.2
10		Column	Linear Elastic	PL.STRN	No water	14.52	Yes	9.08E+06	0.2
11		Ground Beam	Linear Elastic	PL.STRN	No water	19.2	Yes	2.50E+06	0.2
12		Lintel	Linear Elastic	PL.STRN	No water	1.475	Yes	2.30E+05	0.2
13		LE MG	Linear Elastic	PL.STRN	Drained	0	Yes	1.00E+04	0.2
14		LE RTD	Linear Elastic	PL.STRN	Drained	0	Yes	2.50E+04	0.25
15		LE A3	Linear Elastic	PL.STRN	Drained	0	Yes	1.80E+04	0.2
16		Joint	Elastic Mohr/Coulomb	PL.STRN	No water	0.1	Yes	3000	0.2

Figure 3.4: Materials used in Safe and their key properties

Stratum	**Level at top, mATD (thickness, m)**
Made Ground	112 (3)
River Terrace Deposits	109 (6)
London Clay	104 (14)

Table 3.1: Soil stratum adopted in Safe FE mesh

Figure 3.5 displays a screenshot of the completed mesh at the end of Run 1 with the coordinate system shown. Along the sides of the mesh horizontal restraints are applied, in conjunction with standard practice in FE programmes with pinned restraints applied at the base of the mesh. The building façade and ground beam is left as void space.

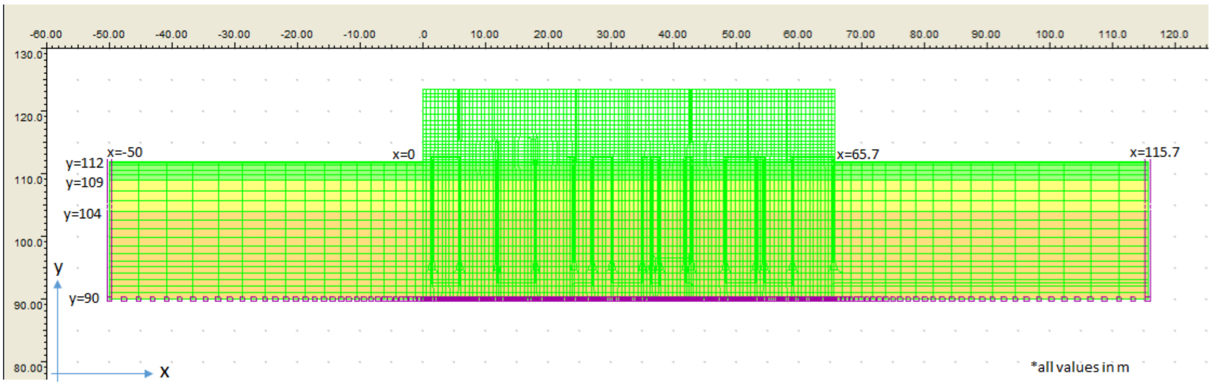


Figure 3.5: FE mesh for initial conditions in Run 1

Run 2: Install Piles involves installing the piles by changing the pile material properties from the soil conditions applied in Run 1 to a linear-elastic material. Slip elements are input either side of the pile, modelling the pile-soil interaction.

Run 3: Construct Building involves inputting the building materials for the building façade as shown in Figure 3.6. This represents the completed mesh used for analysis.

Run 4: Construct Tunnels inputs the ground movements into the model which were generated in *Arup Oasys* software, *XDisp*, which is used to generate a series of nodal displacements that can be applied to a FE mesh. Nodal displacements are applied at chosen nodes within the mesh. These are graphically displayed in Figure 3.7. Displacements were also applied at the sides of the mesh. The centre-lines of the tunnels are at *x=19.4* (westbound) and *x=67.5* (eastbound), seen by the fact the maximum settlement occurs at these points.

Safe uses an iterative method to calculate the displacements by converging towards a solution, with the number of iterations required increasing with calculation complexity. The nodal displacements due to tunnelling are applied incrementally in 10 stages in the Safe model. This is better explained using an example in section 4.1.

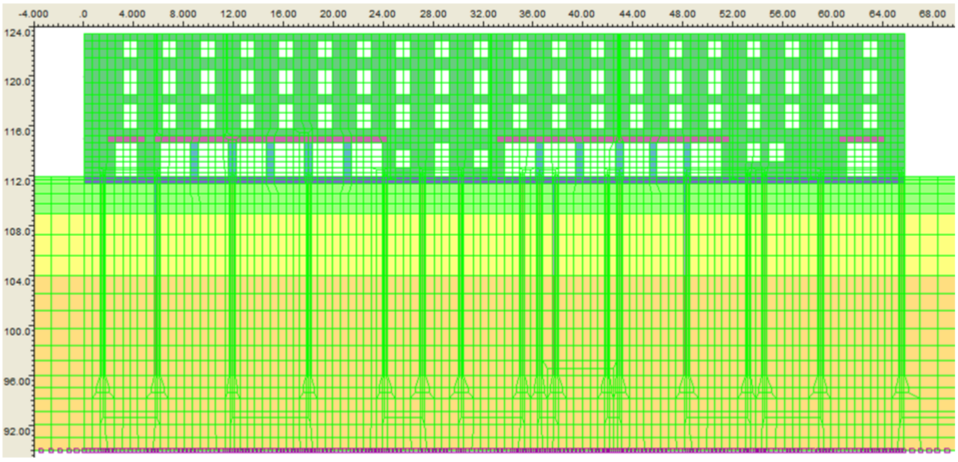


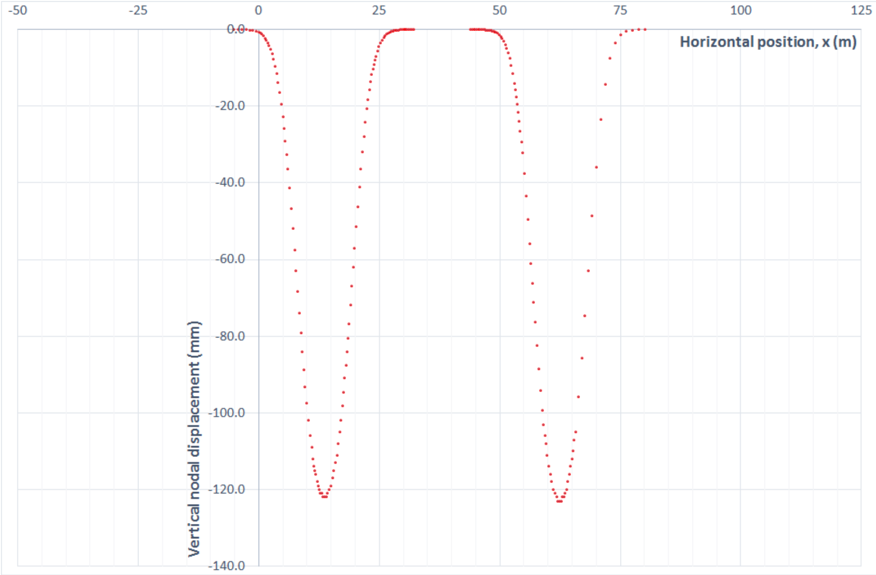
Figure 3.6: Zoomed-in mesh for Run 3 showing buildings (and piles from Run 2)

3.2 Three-dimensional modelling: background information

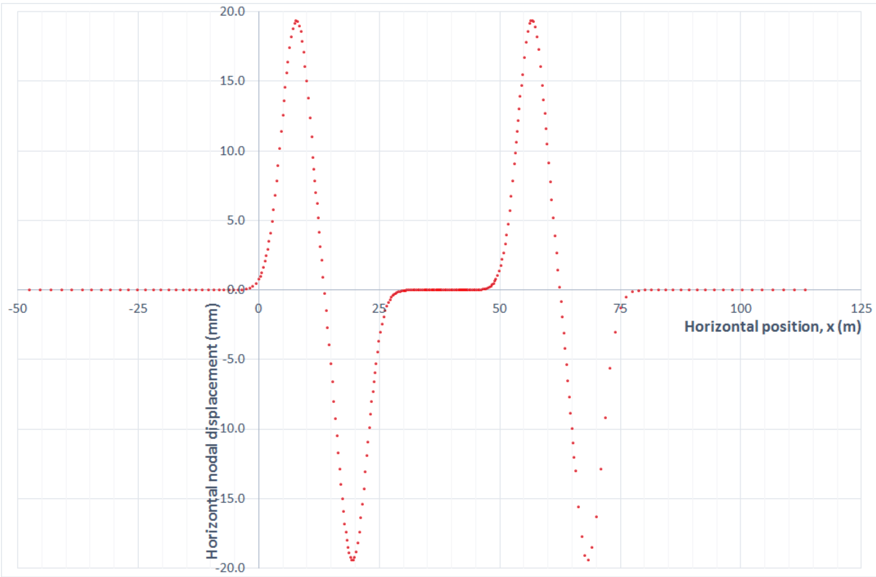
LS Dyna is a 3D FEM software developed by *Livermore Software Technology Corporation* with contributions from *Oasys Arup*. It represents a powerful FE analysis tool and is used extensively in modelling geotechnical scenarios.

3.2.1 Three-dimensional finite element modelling in LS-Dyna

LS-Dyna operates using many of the principles used in 2D FEM, but with extra complexity. As with Safe, Dyna discretises the problem area into a mesh of finite elements that are built up to render a model for analysis. The major difference is that the elements are 3D, with example shapes shown in Figure 3.8 depending on the type and complexity of analysis required. Solid elements have four-to-eight nodes, usually at each vertex, with computations made by integrating through the whole volume of the element. The interaction between different elements uses a three-dimensional contact-impact algorithm as defined by the LS-Dyna User Manual. This defines one surface as the "master" surface



(a) Vertical displacements at y=90



(b) Horizontal displacements at y=90

Figure 3.7: Input nodal displacements for Safe model at various positions in mesh

and the other as "slave". Each surface is defined by three or four node quadrilateral segments on which the nodes of the "slave" and master surfaces must slide.

3.2.1.1 Constitutive soil model: 3D BRICK

The BRICK concept in 3D uses the same principals as that in 2D described in subsection 3.1.1. The changes are referred to here, with full explanations provided in the literature cited. The 3D BRICK model was developed by Lehane and Simpson [2000] describing similarities between Simpson [1992], along with a number of modifications, which primarily involve transforming the 2D strain space into a 3D strain space.

(a) Eight-node hexahedron element

(b) Four-node tetrahedron

Figure 3.8: Possible 3D element shapes

3.2.2 Model set-up

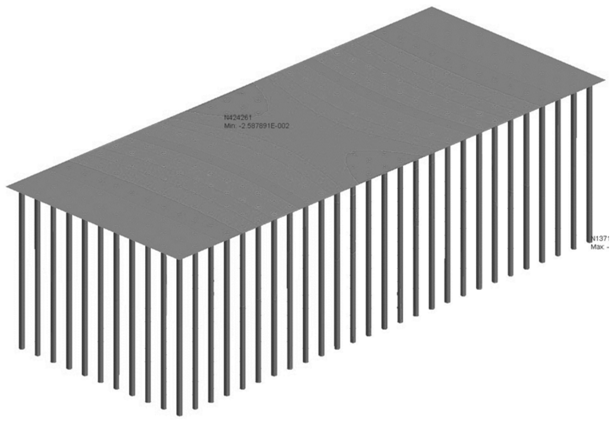


Figure 3.9: Simplified model of piles and ground beam connection

The original plan set out at the commencement of this project was to attempt to develop a 3D model of KC and to repeat similar analysis in LS-Dyna that replicate the results from the 2D analysis. Unfortunately, this had to be streamlined with the eventual model being a simplified model of a group of piles connected by a stiff ground beam as shown in Figure 3.9, with Figure 3.10 showing a plan view of the completed mesh. Due to the complexity of 3D modelling, the construction sequence in LS-Dyna involved thirteen stages consisting of setting up initial conditions, constructing the piles and pile cap and modelling the tunnelling effects. The exact nature of each stage is excluded for brevity. Figure 3.11 shows the dimensions and key features of mesh.

Initially, a nodal displacement method was used, as in Safe, though this proved unsuccessful, with the quantity of nodes causing too complex-a-computation. Therefore, tunnelling effects were input by applying a volume loss at the base of the mesh according to the standard Gaussian profile outlined in section 2.1. One final difference is that only one tunnel was modelled, running through the centre on the group piles along the y-y axis. Unfortunately, the tension cut-off analysis and horizontal position of the tunnel variation could not be replicated in 3D due to the time constraints and the complexity of such analysis.

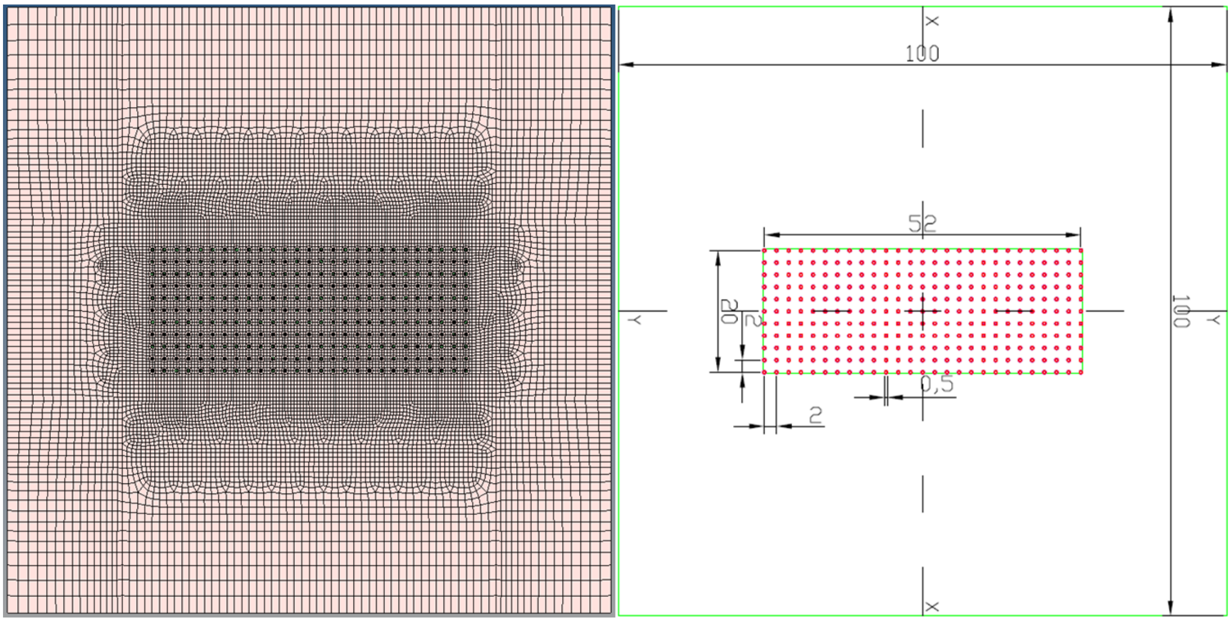
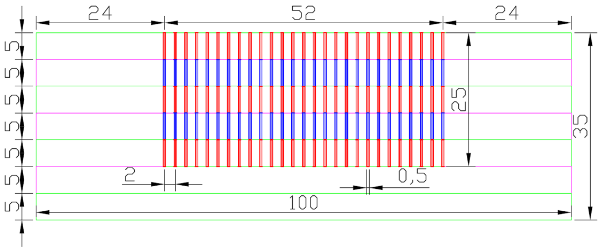
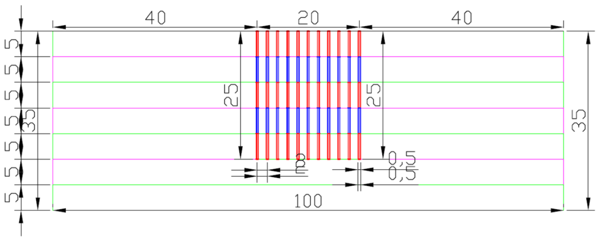


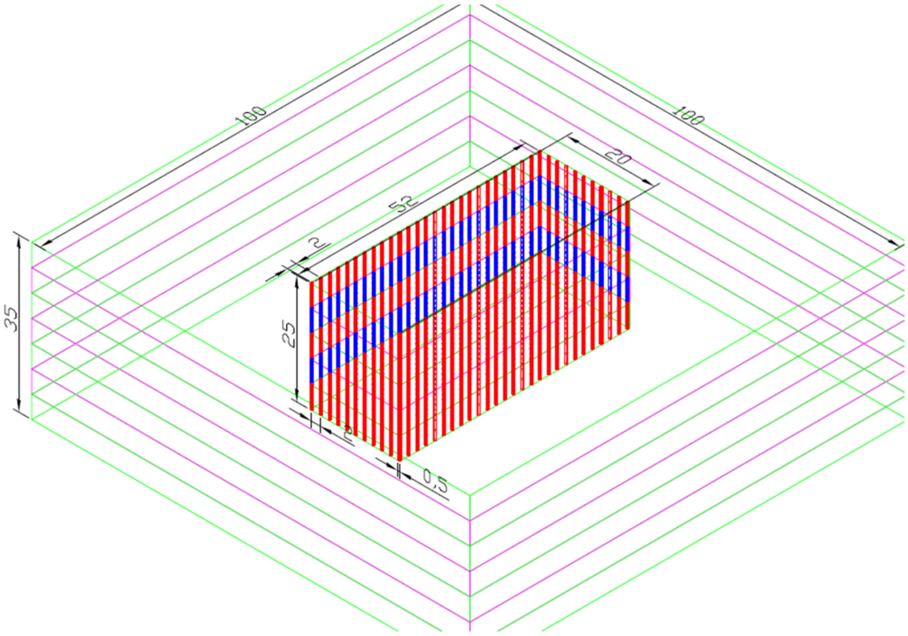
Figure 3.10: Plan views of 3D FE mesh



(a) Side view y-y plane



(b) Side view x-x plane



(c) Isometric view

Figure 3.11: Mesh construction from LS-Dyna

Chapter 4

Two-dimensional Finite Element Analysis

This chapter concerns the 2D analyses conducted in *Arup Oasys* finite element software Safe. An FE model was provided, courtesy of *Arup*, from which a back analysis of Kempton Court was conducted. The areas of analysis and results are presented in this chapter.

4.1 Initial Analysis and Model Compatibility

4.1.1 Greenfield analysis

With any FE analysis for modelling geotechnical scenarios, it is important to be able to generate acceptable greenfield outputs in order to develop confidence that the model will output accurate results once further analyses are conducted. It is expected that a greenfield analysis of the KC mesh would render displacement results similar to those predicted in Figure 2.2.

Greenfield analysis requires the tunnelling effects to be input into the model immediately after the initial set-up phase; Run 4 following Run 1, by applying the nodal displacements to the initial conditions. As mentioned in subsubsection 3.1.1.2, computations are made incrementally in 10 stages in Safe. Figure 4.1 displays the cumulative displacement after each increment to highlight how the solution is calculated. The theoretical greenfield plots are derived using the data for greenfield volume losses and settlement trough widths in subsubsection 2.2.2.3 presented by Mair and Williamson [2014]. The curves were derived using the same equations for deriving the volume loss in subsubsection 4.1.1.1, with the added expression $i = Kz_0$ used to find the i value.

Figure 4.1a displays the vertical greenfield settlement at ground level due to tunnelling effects. The horizontal axis is relative to the FE mesh coordinates, as in Figure 3.5. The expected Gaussian displacement trough for vertical soil settlements is observed, with the presence of two tunnels having the effect of superposing two Gaussian solutions, as suggested by Mair et al. [1996]. There is a close relationship between the theoretical and model greenfield plots. The nature of Figure 4.1a suggests that the FE model is accurate and reliable, with the slightly increased settlement for the westbound tunnel due to it being slightly deeper and the fact it was constructed first. Figure 4.1b displays the horizontal displacements at ground level. It is surprising that the westbound tunnel gives displacements much greater than the eastbound tunnel, as theoretically, the profiles for each tunnel should be symmetrical, as shown by the theoretical plot. Possible reasons for this are investigated in subsubsection 4.1.2.2.

4.1.1.1 Volume Loss

It is useful to compare volume losses observed from field measurements to those predicted by the FE analysis. Safe does not calculate volume losses, therefore numerical calculations are required. As referenced in section 2.1, transverse movements in clay take the form of a Gaussian settlement trough according to Equation 4.1,

$$S_v = S_{v,max} \exp\left(\frac{-y^2}{2i^2}\right) \tag{4.1}$$

where S_v is the vertical settlement at transverse distance x from the tunnel centreline, $S_{v,max}$ is the maximum settlement of the trough assumed to be at the tunnel centreline and i is the distance to the point of inflection. A note must be made on nomenclature to avoid confusion; y in Equation 4.1 represents the horizontal x coordinate in the Safe model, and must not be confused for vertical position. Plotting -ln($S_v/S_{v,max}$) against y^2 can be used to provide the value of the gradient $2i^2$, from which the value of i may be deduced. It should be noted that the theoretical model assumes the tunnel centre line at a value of x=*0*; therefore, the values of x output from the model need to be shifted by *19.4* and *67.5* for the westbound and eastbound tunnel respectively.

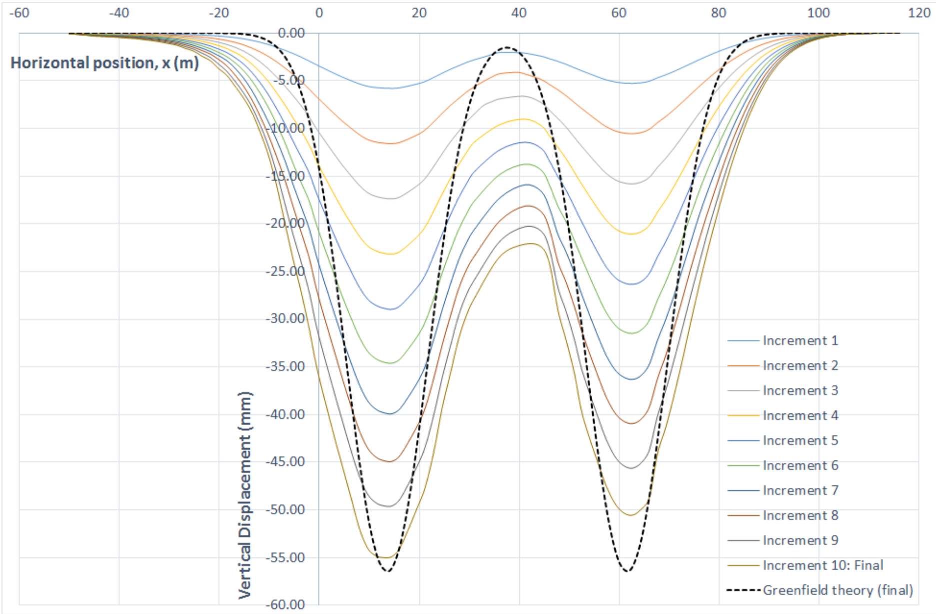
Mair et al. [1993] stated that the settlement trough, V_s, can be evaluated by integrating Equation 4.1, giving:

$$V_s = \sqrt{2\pi} i S_{max} \tag{4.2}$$

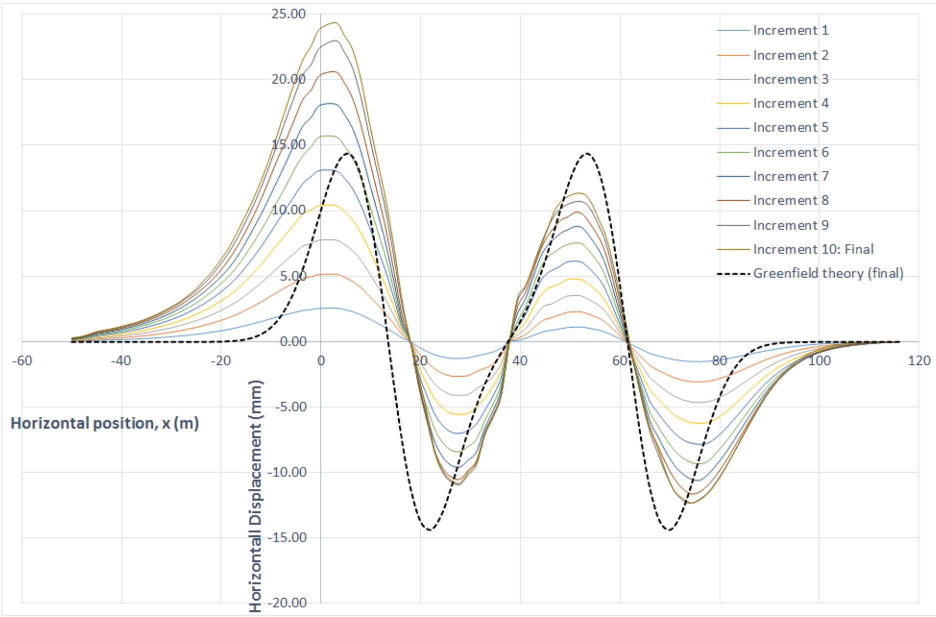
and that the volume loss, V_l, can be expressed by:

$$V_s = V_l \frac{\pi D^2}{4} \tag{4.3}$$

By rearranging Equation 4.2 and Equation 4.3, an approximation of the volume loss may be obtained, with the volume loss due to westbound and eastbound tunnels calculated to be *1.26%* and *0.85%* respectively. The calculated values match well with those predicted by Williamson [2014], subsubsection 2.2.2.3 and Mair et al. [1996]. Considering



(a) Vertical settlement at ground level under greenfield conditions



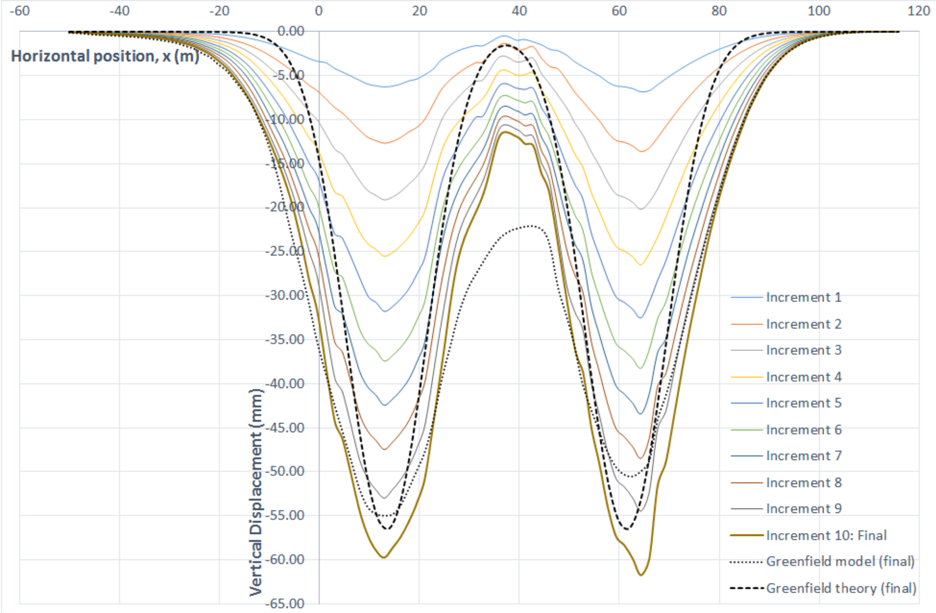
(b) Horizontal displacement at ground level under greenfield conditions

Figure 4.1: Settlements and horizontal displacements under greenfield conditions

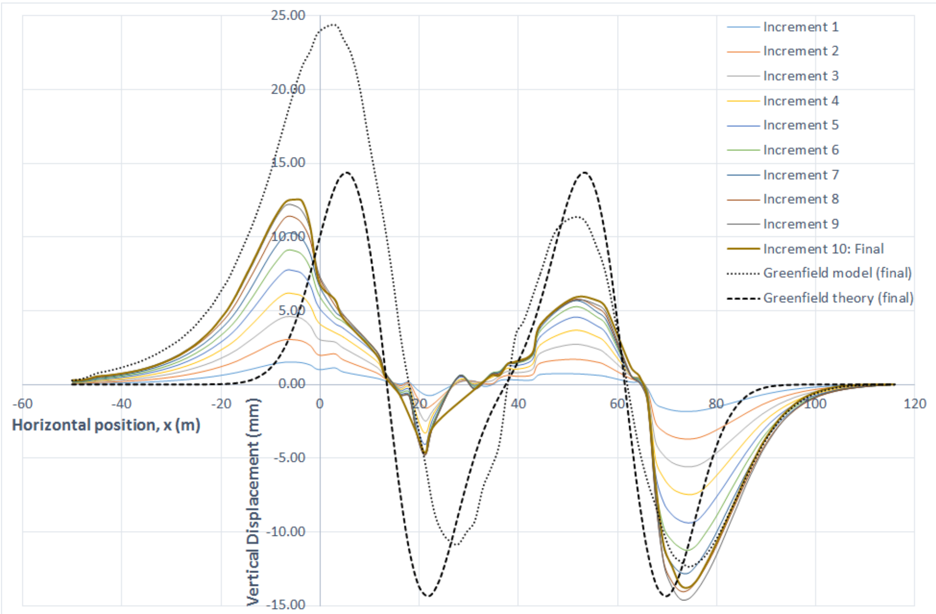
the inaccuracies in the above method, the calculated volume losses appear reasonable, adding credibility to the model.

4.1.2 Initial Run: Kempton Court Back Analysis

The next set of analyses involved observing the output of the original model with a building present. As previously, it is useful to observe each incremental calculation; hence, Figure 4.2 displays the cumulative settlements and horizontal displacements after each increment to show how they are generated in Safe.



(a) Vertical settlement at ground level under initial conditions



(b) Horizontal displacement at ground level under initial conditions

Figure 4.2: Settlements and horizontal displacements under initial conditions

Figure 4.3 shows the settlement profile at various levels through the model, displaying how the settlements propagate through the subsurface. The magnitude of settlements increase the further from ground level, with the subsurface experiencing greater settlement than the surface. Figure 4.4 shows the equivalent for horizontal displacements.

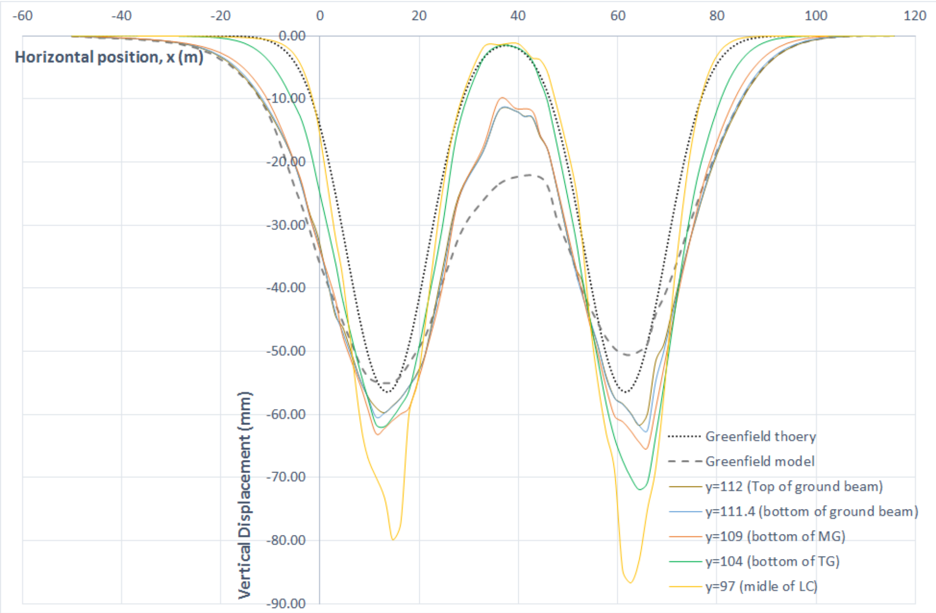


Figure 4.3: Vertical settlements propagating through the model under initial conditions

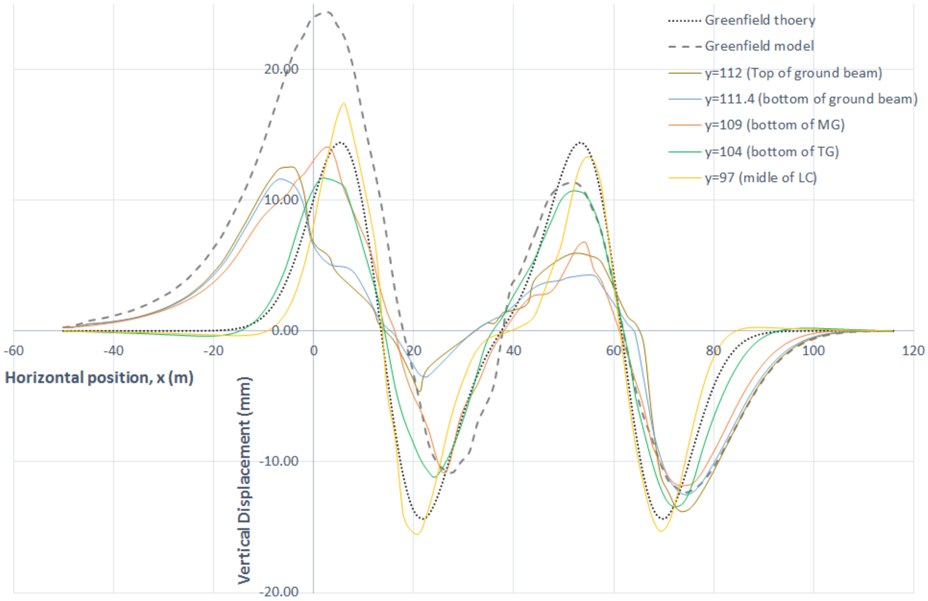


Figure 4.4: Horizontal displacements propagating through the model under initial conditions

4.1.2.1 Volume loss

It is useful to compare the volume losses observed in field observations with those predicted by the model. Using the same method described for greenfield volume loss, the volume losses predicted numerically for the westbound and eastbound tunnel were *1.65%* and *1.58%* respectively, again comparing favourably with subsubsection 2.2.2.3.

4.1.2.2 Comparison between greenfield and initial conditions

Figure 4.1a, Figure 4.2a and Figure 4.3 seem to conform well to theoretical predictions. The shapes of the curves agree with that predicted by Martos [1958] and the superposition of two tunnels suggested by Mair et al. [1996]. The slightly larger settlements for the westbound tunnel is also expected, as it is deeper and constructed first. The settlement profiles also agree with the literature, notably Figure 2.4, (Kaalberg et al. [2005]) and Figure 2.5 (Selemetas [2005]), in that piles directly above the tunnel settle greater than greenfield values, but piles further afield settle less than greenfield predictions.

There remains some uncertainty with the horizontal displacements output by the model in Figure 4.1b, Figure 4.2b and Figure 4.4. In Figure 4.1b, a non-symmetric horizontal displacement response is observed, which is surprising considering the horizontal inputs are symmetric, shown in Figure 3.7b. There is also discrepancy between the theoretical and model greenfield plots. An investigation into reasons behind this was conducted; the process undergone in Figure 4.1b suggests this issue is not due to the construction (iteration) process in Safe. Figure 4.2b shows the same process with a building present. The results are more symmetric, with similar maximum displacements for each tunnel. This could be explained by the fact the building stiffness is affecting the horizontal displacements of the soil. The horizontal displacements propagating through the soil are shown in Figure 4.4; these appear reasonable, giving fairly symmetrical solutions.

The three graphs show a sensitivity analysis for the model to try to understand the horizontal displacements by looking at the construction sequence, incremental computations, the effect of a building and a profile of soil displacements with depth. However, there remains uncertainty in the horizontal outputs produced by the model. Possible explanations are that, for horizontal displacement, it is not possible to linearly superpose solutions, as done by Mair et al. [1996] for settlements; there may be a non-linear relationship that is not taken account of. Alternatively, there could be unaccounted shear effects between the different soil layers within the soil stratum that transfer horizontal loads in a different way than expected. However, both these hypotheses would require further testing for them to be validated.

4.1.2.3 Discussion

For vertical settlements the results show a close fit to the theoretical predictions made by Martos [1958], with the greenfield and initial analysis giving expected results. Therefore, one can accept confidence in the model for the purposes of further settlement analysis. One consideration, is that the volume losses obtained from the analysis are larger than those observed in both the greenfield analysis and the for the building response to tunnelling. An explanation of this is, that during Crossrail construction, compensation grouting was used, which would reduce the observed settlements. Compensation grouting is not modelled in Safe, which could explain the discrepancy in the values. The increased settlement in the presence of a piled foundation could also be due to onset of pile cracking, investigated in subsection 4.1.4. There appears to be a corresponding decrease in horizontal displacements when group pile foundations are present.

4.1.3 Effect of building stiffness on tunnelling effects

The preliminary analysis involved investigating the effect of varying building stiffness on tunnelling effects, with the primary focus being analysing settlements. It is often assumed that the stiffness contribution from the building façade is negligible compared to that of the ground beam. Therefore, building stiffness analysis was conducted in three stages to compare results and to determine whether the building stiffness may be ignored.

Stage 1 involved removing the façade of the building and modelling KC as simply a group pile foundation, connected by a ground beam of a given stiffness. The stiffness of the ground beam is taken to be the same as the original model. ***Stage 2*** involved running the original model as in section 4.1, but varying the stiffness parameter of the ground beam, whilst keeping the façade stiffness constant. ***Stage 3*** involved varying the façade stiffness and ground beam stiffness simultaneously.

Stage 1: Model as group pile foundation connected by a ground beam of varying stiffness. As a preliminary analysis, five different stiffness values were applied to the ground beam. Increasing ground beam stiffness has the effect of reducing settlements, whilst reducing ground beam stiffness appears to have little effect on the predicted settlements.

Stage 2: Vary ground beam stiffness with façade present. The next stage of the analysis involved changing the ground beam stiffness whilst keeping the façade material properties constant. The graphical results for vertical settlements are displayed in Figure 4.6 for the different stiffness values tested.

Stage 3: Vary ground beam and façade stiffness concurrently. Figure 4.7 shows the variation in ground settlements for the case when ground beam stiffness and façade stiffness are altered. The observed trends are the same as those seen in stages 1 and 2.

4.1.3.1 Discussion

Comparing all three stages, it can be seen that the possibility of ignoring building façade stiffness would not be appropriate. With the effect of the building stiffness included, the settlements at the equivalent stiffness of the ground beam are reduced. This suggests that façade stiffness makes a small, yet significant, contribution to the overall building stiffness.

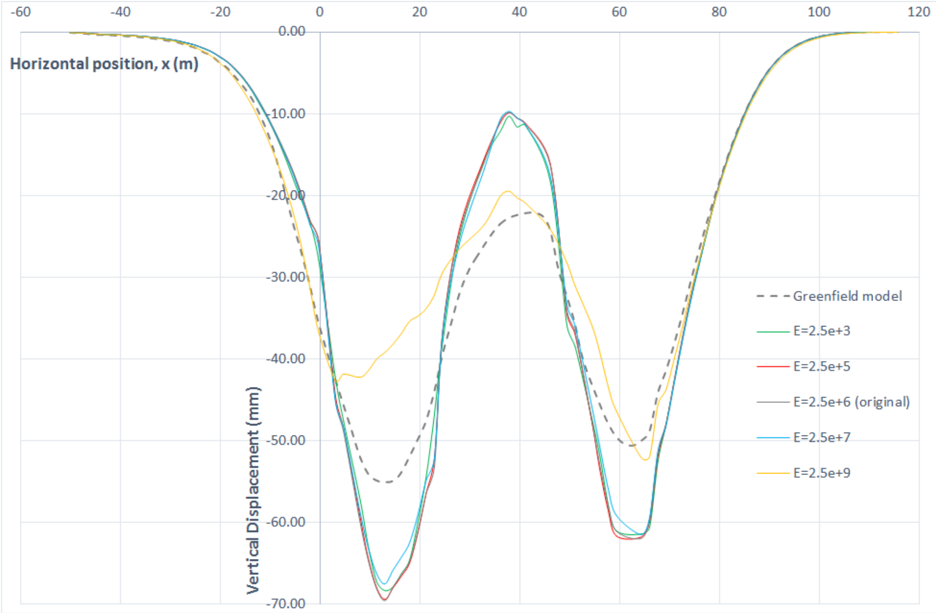


Figure 4.5: Vertical settlements for different ground beam stiffness values at ground level for the situation where just a ground beam is present

Further stiffness variants were conducted using the approach adopted in Stage 2 to investigate the effect of salient values of stiffness on settlements. The results are displayed in Figure 4.6. The situation for maximum stiffness, *E=1e+12kPa*, shows that the foundation settles like a rigid raft foundation, which is the expected observation. For decreasing stiffness, there appears to be less of an effect on the settlement. The largest settlements were observed for the case of minimum stiffness, *E=1000kPa*, where the foundation behaves as a fully-flexible raft foundation.

The results show some recurring trends, with settlement decreasing with increasing building stiffness. For decreasing stiffness, an increase in settlement is observed; however, this occurs at a slower rate, with a smaller difference between maximum settlements when comparing low stiffness values. For higher stiffness values, wider settlement troughs are experienced due to the more rigid behaviour of the group pile foundation exhibiting a more influential response on the surrounding soil.

4.1.4 Effect of pile cracking: tension cut

A further area of analysis looked at the effect applying a tension cut to the concrete pile material. A tension cut effectively limits the tensile capacity of a material, by setting a limiting value of tension that a material can carry in the model, which can be used to look at the effects of cracking in the piles. Pile cracking could provide an explanation as to why the observed settlements from data monitoring are lower than those predicted by the model.

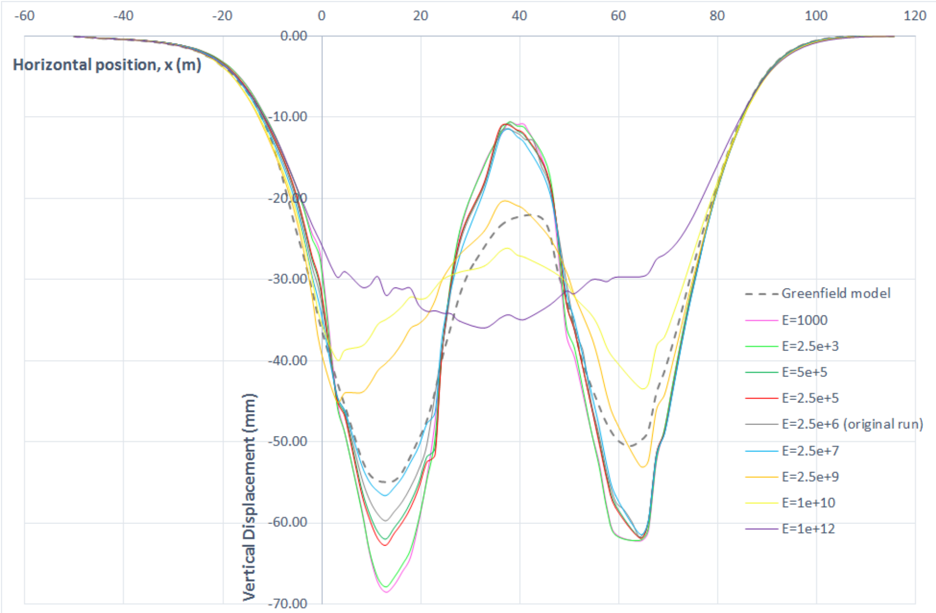


Figure 4.6: Vertical settlements for different ground beam stiffness values at ground level for the situation with building faade included, but with the stiffness of the building façade kept constant

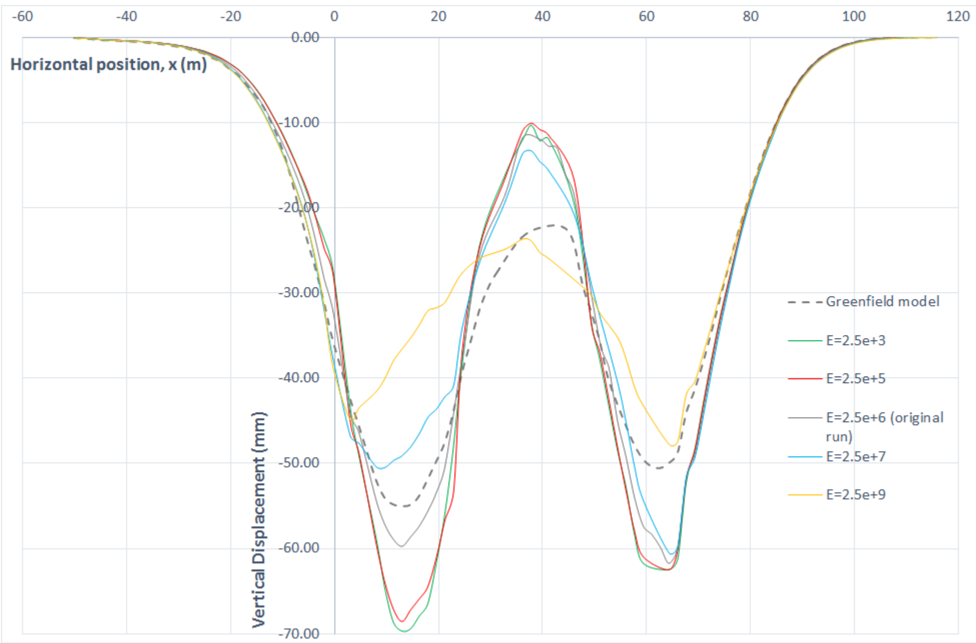


Figure 4.7: Vertical settlements for different ground beam stiffness values at ground level for the situation with building façade included and varied with ground beam stiffness

Tension cut off parameters can only be applied to Mohr-Coulomb materials. In the original model, the pile material is modelled as Linear-Elastic; therefore, the material model must be changed. It is important to ensure that these changes do not affect the output. A preliminary analysis was conducted where the material parameters were changed, but no cut off applied, to verify that this had no effect on the model. The result of this comparison showed the Mohr-Coulomb curve directly superposed on top of the Linear-Elastic curve, showing there is no effect in changing the materials.

Subsequently, a series of tension cut off values were applied to the model. Note that compression is positive. Figure 4.8 shows the results for different cut off values and their effect on settlements. It shows that the effect of applying a tension cut, even in the most extreme case of zero tension, has little overall effect on the total displacement, although a slight reduction may be seen when compared to the results when no tension cut is applied.

Looking at contour maps for the whole mesh provides a better understand of the effect of tension cuts. Figure 4.9 and Figure 4.10 show contour maps for the case of a *-100kPa* tension cut and no tension cut. As expected from Figure 4.8, the contour plots hardly differ when looking at displacements. However, the stress contour plots in Figure 4.10 show some deviation from one another. Note that colour scales on the plots are different due to the fact that a different range of stresses are experienced throughout the model.

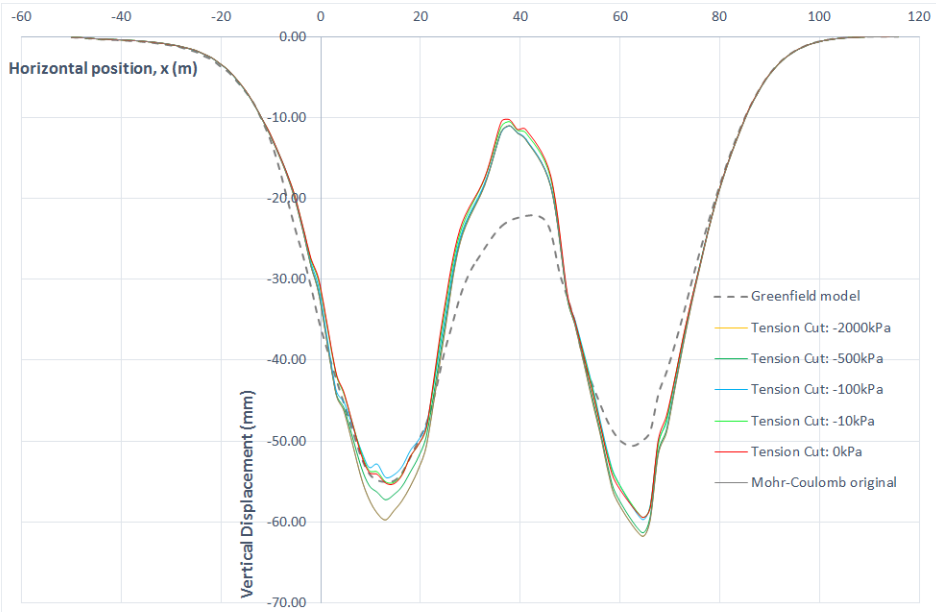
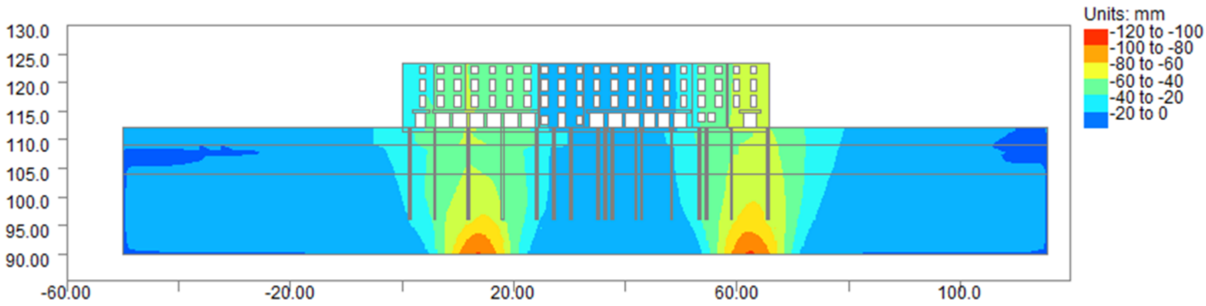


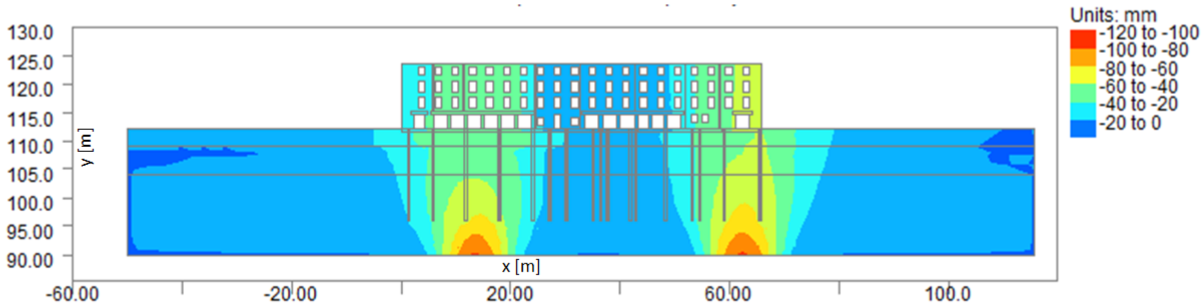
Figure 4.8: Vertical settlements at ground level for different values of tension cut off

Figure 4.10 shows zoomed-in contour plots of the piles, where the red sections denote zones in compression, and the blue zones denote tension. The original model clearly has piles in tension (piles 2-4). These piles are no longer in tension once the tension cut of *-100kPa* is applied suggesting the piles no longer take any capacity (as expected). The removal of any tensile capacity in some piles, causes an increase in the compressive force in the piles elsewhere in the foundation; shown by the maximum tensile force in the original model being *300-350kPa*, yet *400-450kPa* once the tension cut is applied.

A tension cut effectively models a pile cracking under tension, applying the assumption that concrete has limited tensile capacity. These results from Figure 4.8 to Figure 4.10 suggest pile cracking does hugely influence the structure as the force path is simply moved from the cracked piles to other areas of the foundation. The same results were observed for the other values of tension cut applied.



(a) Contour map for vertical settlement under initial conditions

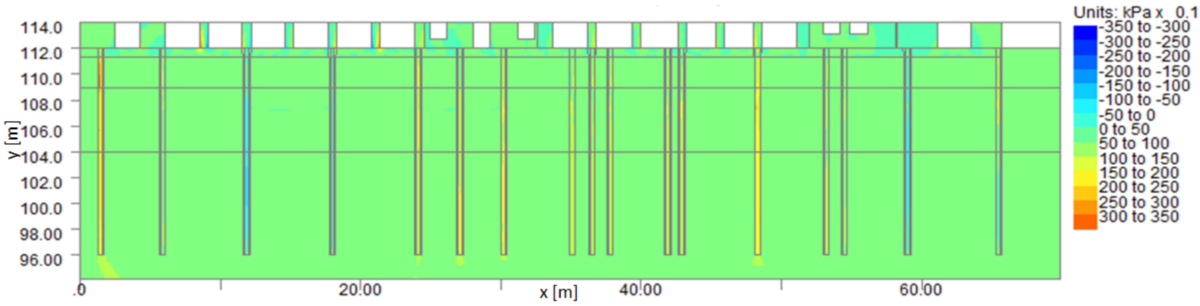


(b) Contour map for vertical settlement with a tension cut of -100kPa

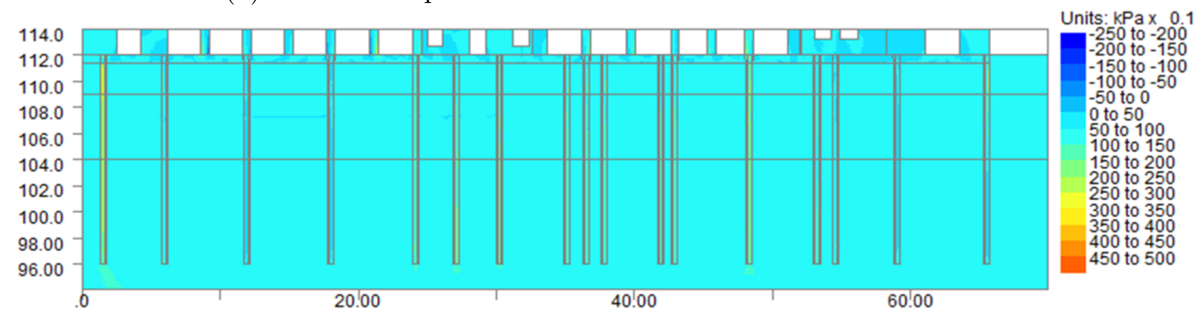
Figure 4.9: Displacement contour plots: initial analysis comparison to -100 kPa tension cut

4.1.5 Effect of varying tunnel horizontal position

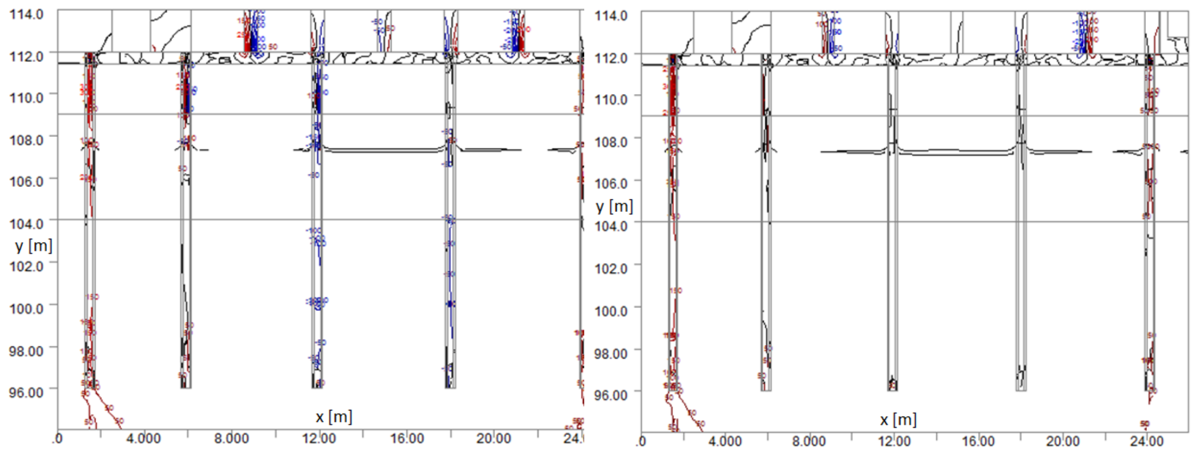
The next set of analyses involved varying the horizontal tunnel position with respect to the group pile foundation. To calculate the required input displacements for the different tunnel positions, a number of numerical calculations were required. The original input data for the model was shown in Figure 3.7. The input vertical displacements at the base of of the mesh were created according to Equation 4.1. As with the calculations made for volume losses, the *y-value* in Equation 4.1 corresponds to horizontal position, x, in the model. By reading off the graph in Figure 3.7a, the maximum settlements for each tunnel could be found: *122mm* (westbound) and *123mm* (eastbound).



(a) Contour map for vertical stress under initial conditions



(b) Contour map for vertical stress with a tension cut of -100kPa



(c) Zoomed-in contour plot for vertical stress under initial conditions [blue: tension, red: compression]

(d) Zoomed-in contour plot for vertical stress with -100kPa tension cut [blue: tension, red: compression]

Figure 4.10: Stress contour plots: initial analysis comparison to -100 kPa tension cut

Equation 4.1 can be modified in order to account for the fact that the tunnel centrelines in the model are not at *x=0*. In the modified Equation 4.4, the *y'* value accounts for the shifted *x-value*. For the displacements used in the original model, $y'_{west}=13.990$ and $y'_{east}=62.690$ are used. The points of inflexion *(i-values)* were found in the same manner as in subsubsection 4.1.1.1, being $i_{west}=5.0$ and $i_{east}=5.1$.

$$S_v = S_{v,max} \exp\left(\frac{-(y-y')^2}{2i^2}\right) \tag{4.4}$$

Equation 4.4 can be used to generate displacements of shifted tunnel positions. The input data is displayed below in Figure 4.11.

Horizontal displacement inputs can be found according to Equation 4.5, proposed by O'Reilly and New [1982], accounting for the shifted tunnel axis as in Equation 4.4. z_0 is the vertical position of the tunnel axis relative to ground level and is taken as *19.5m* (westbound) and *19.4m* (eastbound). The horizontal inputs for shifted tunnel axis are also displayed in Figure 4.11.

$$S_h = \left(\frac{(y-y')}{z_0}\right) S_v \tag{4.5}$$

Figure 4.12 and Figure 4.13, display the vertical displacements output by the FE model corresponding to the relevant input data for the shifted tunnel axis.

4.1.6 Discussion

Figure 4.12 and Figure 4.13, there is not a marked difference in the shapes of the graphs between the different situations, nor in the magnitudes of the maximum deflection which lies in the region of *-60mm* to *-65mm* for each case. The settlements calculated due to tunnelling when the tunnel axis lie outside the span of the building are small, cases b to d, and reduce to similar values to those seen from the greenfield tests of *-45mm* to *-50mm*. A reduction in settlement values outside the area of the building would be expected as the building influence on settlements is less. However, it is somewhat surprising that the settlements reduce towards greenfield values so quickly when the tunnelling axes move outside the building area. This is likely to suggest that the building is flexible and makes little stress contribution outside its foundation area.

4.1.7 Summary

This chapter looked at analysis conducted using two-dimensional finite element software, Safe. The following conclusions can be drawn from this chapter; from the initial analysis:

- The output observed from the greenfield analysis and under initial conditions suggest that the FE model is reasonably reliable, though the remains some uncertainty with the horizontal displacements output.

- The presence of a grouped pile foundation causes an increase in magnitude of vertical settlements due to tunnelling effects, though there is little difference in the shape of the settlement trough between the two scenarios. The largest settlements are observed directly above the tunnel centreline.
- The increased vertical settlements lead to increased volume loss for the case when a building is present.
- The increased settlements predicted by FE analysis could be explained by the fact that the effect of compensation grouting is not considered in the model; however, further analysis is needed to validate this.
- The presence of a group pile foundation causes a reduction in horizontal displacements.

From investigating the effect of building stiffness:

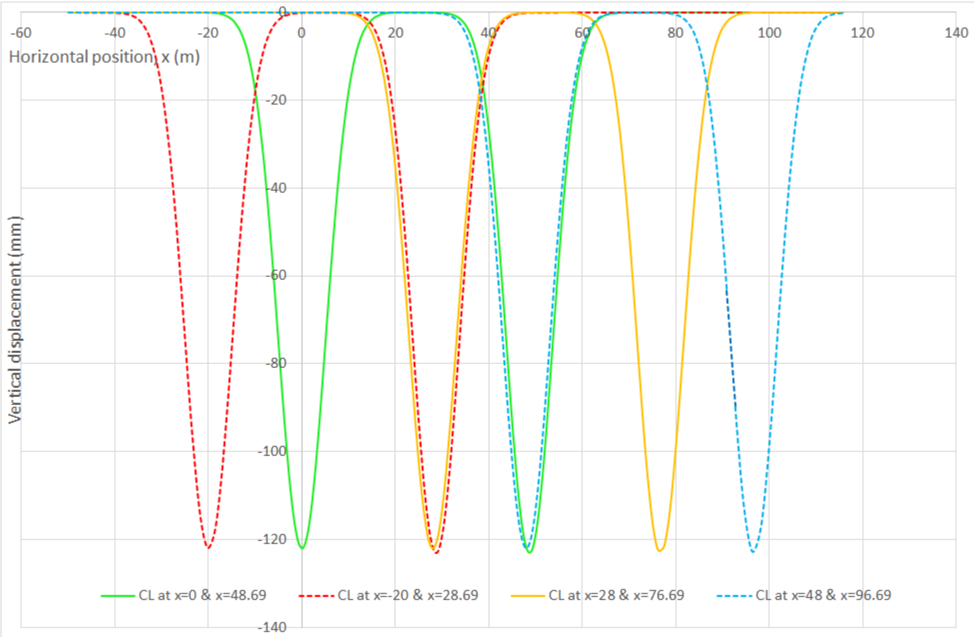
- The effect of façade stiffness should be considered in the analysis, with results showing that the façade makes a significant contribution to the overall stiffness of the building and foundation.
- Vertical settlements decrease as building stiffness increases. For the case for a "rigid" building, the settlement response is similar to that of a rigid punch solution.
- Larger settlement troughs are observed for stiffer scenarios. Less stiff buildings exhibit a similar settlement trough response.

From investigating the effect of a tension cut:

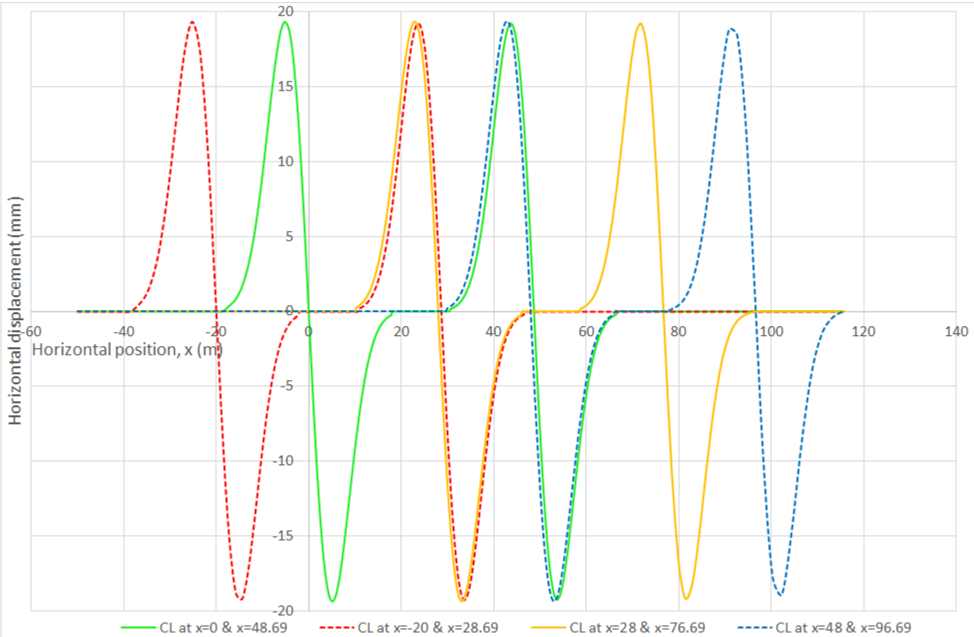
- The response when applying a tension cut to the piles does not hugely differ from when no tension cut is applied, with respect to vertical settlement. There are differences in the way that stresses are distributed through the structure as load paths are changed by the application of a tension cut.
- Applying a tension cut essentially removes the effectiveness of piles under high tension, essentially modelling the piled foundation with these piles not present. The fact there is little influence on the settlements suggests that pile cracking is not the cause of the variation between greenfield settlements and settlements in the presence of a piled foundation.

From investigating the effect varying horizontal tunnel position:

- The piled foundation appears to exhibit a similar response, independent of tunnel horizontal position.
- Settlements reduce rapidly to approximately a greenfield response when the tunnel axis lies outside the building area, suggesting the building is quite flexible.



(a) Vertical



(b) Horizontal

Figure 4.11: Input nodal displacements for shifted tunnel axis (a) & (b) centreline at x=0, x=48.69 (c) & (d) CL at x=-20, x=28.69 (e) & (f) x=28, x=76.69 (g) & (h) x=48, x=96.69

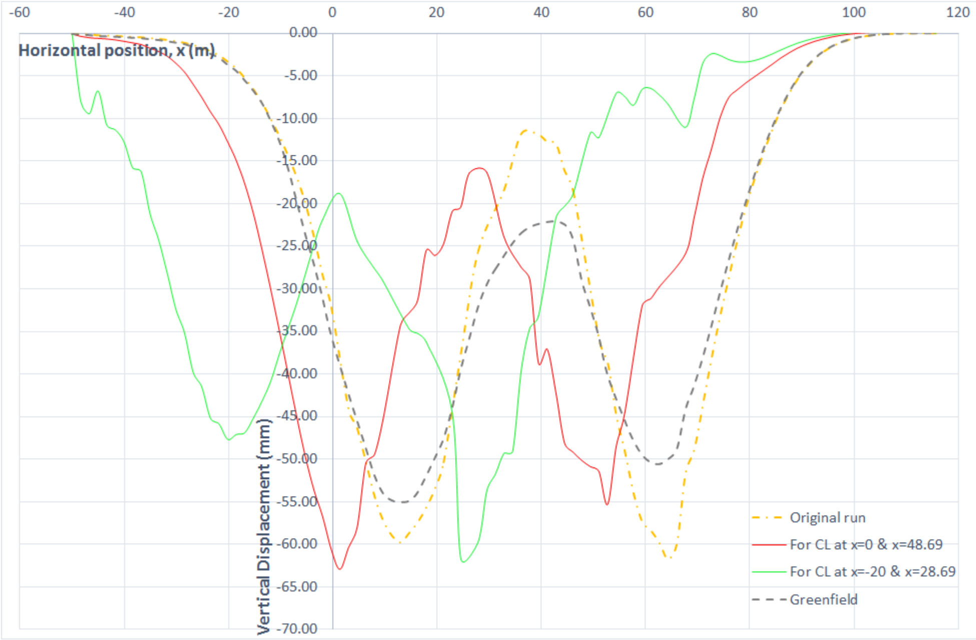


Figure 4.12: Vertical settlements at ground level for tunnel centrelines shifted to x=0, x=48.69 & x=-20, x=28.69 & initial analysis

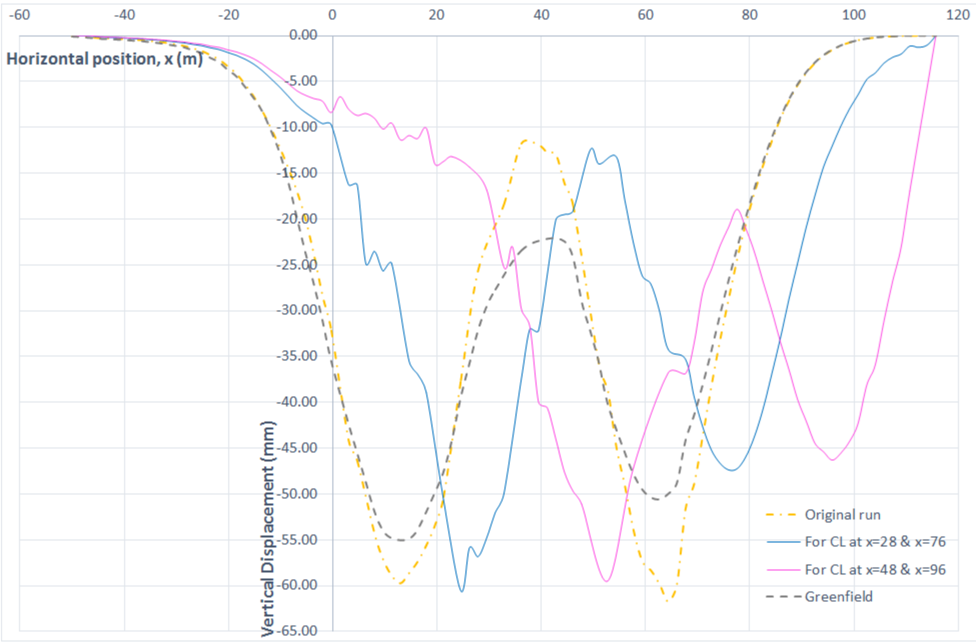


Figure 4.13: Vertical settlements at ground level for tunnel centrelines shifted to x=28, x=76.69 & x=48, x=96.69 & initial analysis

Chapter 5

Three-dimensional Finite Element Analysis

This chapter concerns the 3D FE analysis using a simplified group pile foundation response to tunnelling. The analysis involved trying replicate those results seen in 2D FEM in a more complex 3D FEM.

5.1 Effect of ground beam stiffness on tunnelling effects

Having conducted relevant greenfield analysis, building stiffness analyses in 3D was conducted. As shown in subsection 4.1.3, modelling KC as solely a ground beam and ignoring the effects of façade stiffness, leads to slight differences in the settlement values predicted. An approximation is made in this section such that KC may be modelled as solely a ground beam. Three different stiffness values: $E{=}3.5e6\ kN/m^2$, $E{=}28e6\ kN/m^2$ and $E{=}56e6\ kN/m^2$, representing a "flexible", "normal" and "stiff" building were applied to the ground beam in the model, to see what effect stiffness has on tunnelling effects.

The settlement results for the three different stiffness values are displayed in Figure 5.1 using contour plots having applied a volume loss of *1.5%*. The results appear to reaffirm the observations made during the 2D analysis. There is little difference in the settlement profiles; although slightly larger settlements are observed for the "flexible" pile cap. The profiles of the whole mesh suggest that the pile group is effectively acting as a rigid punch solution due to the fact that the piled area of the mesh exerts a different response (shallower settlement) to the rest of the mesh, with settlements increasing outside of the piles area; the settlements directly below pile cap are smaller than outside this area due to the effects of the pile cap stiffness.

The results suggest that most "activity" is taking place below the surface, shown by the fact that the movement of pile cape is not affected to a large degree, though the centre of the pile is cap is subject to higher stresses due to the higher displacements experienced. It would be useful to conduct further analyses which focus on the behaviour of the subsurface, some of which have been conducted in the proceeding section.

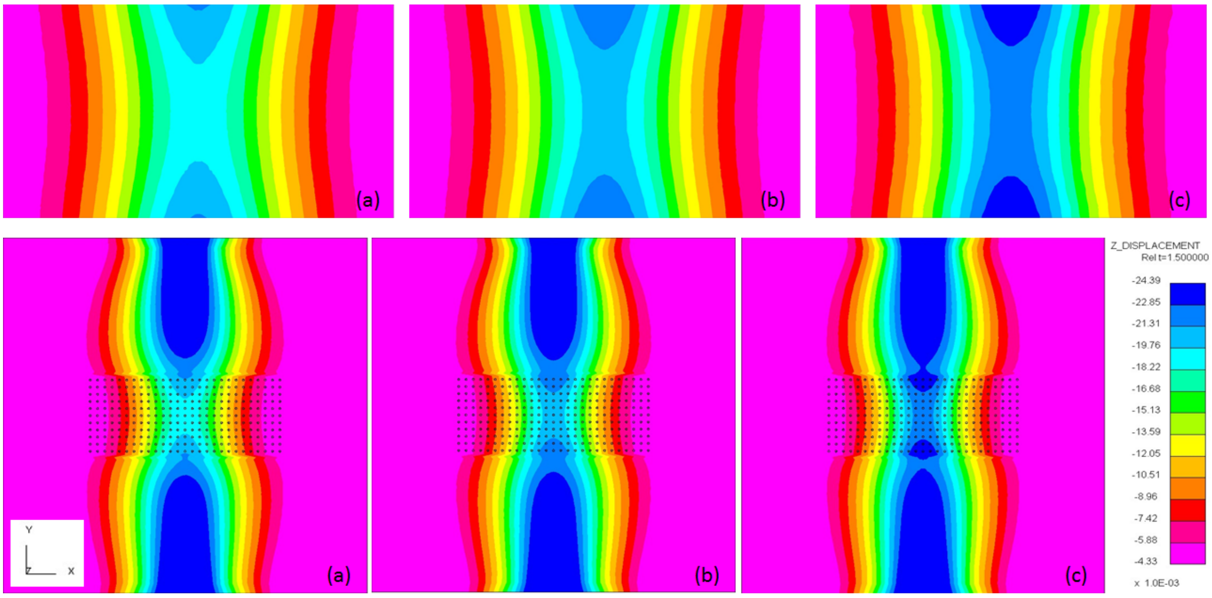


Figure 5.1: Contours showing vertical settlement (z-displacement) due to tunnelling effects for just ground beam and for entire mesh: (a) $E=56e6\ kN/m^2$ (b) $E=28e6\ kN/m^2$ (c) $E=3.5e6\ kN/m^2$

5.2 Bending moment effects with distance from the tunnel axis

It is of interest to look at the bending moments induced in the piles as a function of distance away from the tunnel axis. Figure 5.2 appears to suggest that bending moments are greatest as horizontal distance from the tunnel centreline increases. These observations are confirmed by numerical calculations of the bending moments in each pile, displayed graphically in Figure 5.3. Bending moments were plotted along the centreline of the group piles for $y=0$ and varying x distance from the tunnel centreline in the negative x-direction, according to the coordinate system adopted in Figure 5.1.

5.3 Deflection Ratio

The deflection ratio, $\triangle/L$, of a building can be found according to the diagram in Figure 5.4 and can give an indication of how the stiffness will affect the differential settlements of the building and hence the damage induced in the building. The length L_{AD} in Figure 3.10 of the piled foundation is $52m$. Therefore the ratios for the three stiffness values are 2.67×10^{-4}, 2.97×10^{-4} and 3.56×10^{-4} for the stiff, normal and flexible building

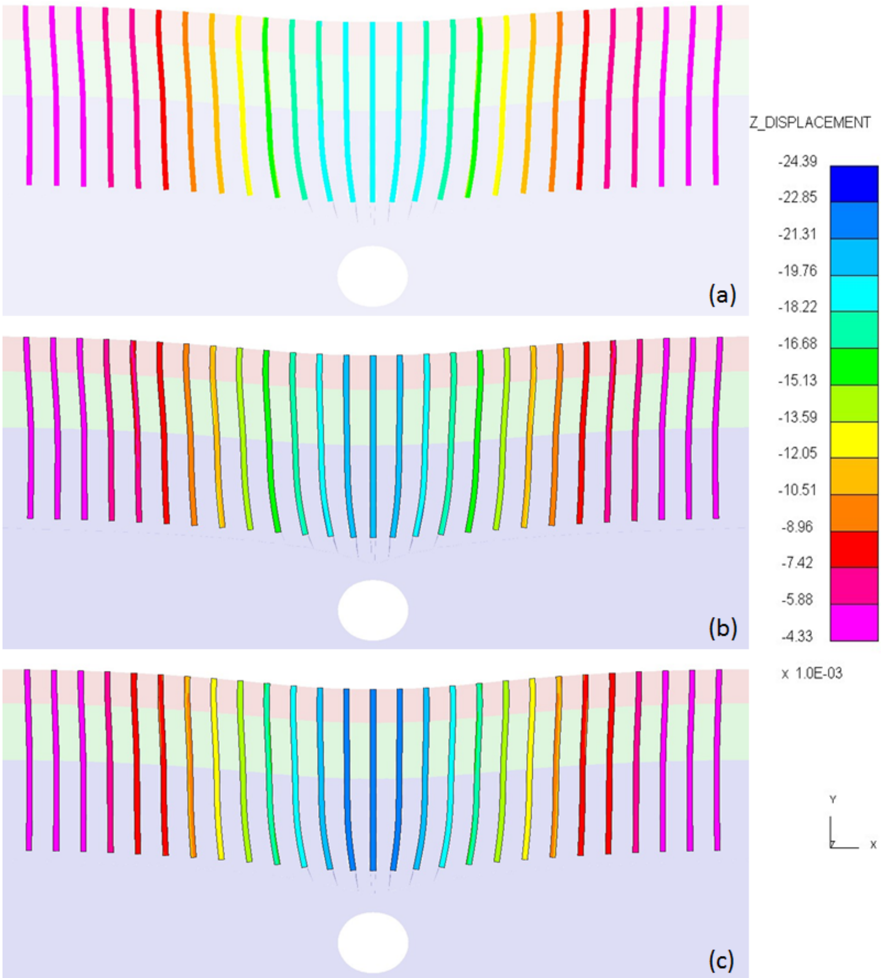
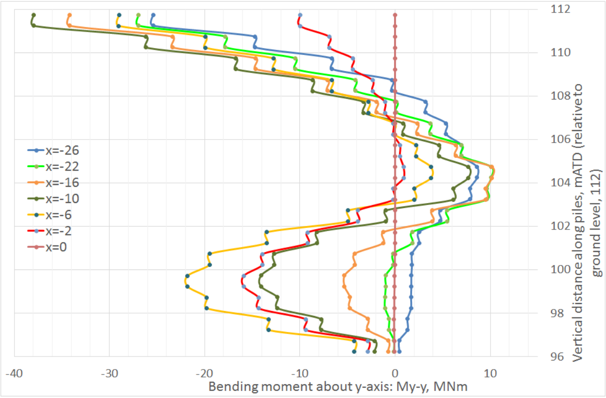


Figure 5.2: Displacements of piles due to tunnelling (a) *E=56e6 kN/m²* (b) *E=28e6 kN/m²* (c) *E=3.5e6 kN/m²*

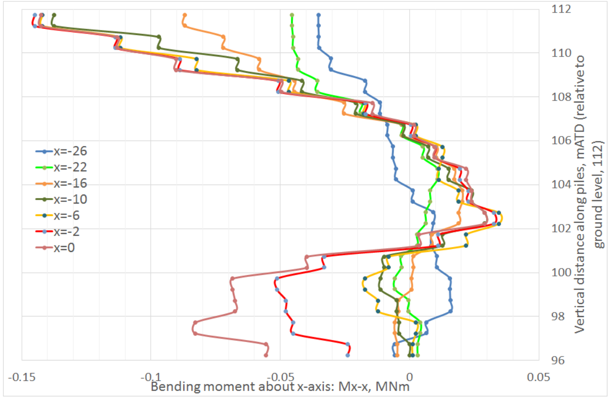
respectively. There is not a great difference in the values implying that building stiffness, in the range of stiffness value analysed, appears to not have a large effect on the deflection ratio, and hence the shape of settlement profile, of the pile group

5.4 Kempton Court building stiffness

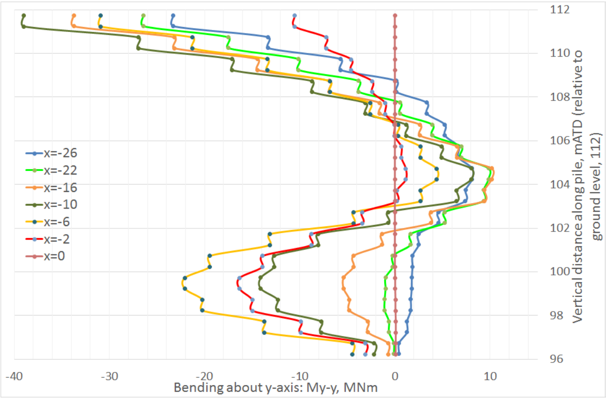
A method for calculating building stiffness was referenced in subsection 2.2.5. Using this method, hand calculations were conducted to estimate the building stiffness of KC to compare with the analyses. The calculations showed that the effect of the internal structural elements had a small effect on the overall stiffness of the building, meaning these stiffness components can be ignored as a conservative assumption. The E-value for the building was found to lie between the flexible and normal stiffness cases, suggesting KC should respond in a similar fashion to the flexible pile-cap; this matches the field observations.



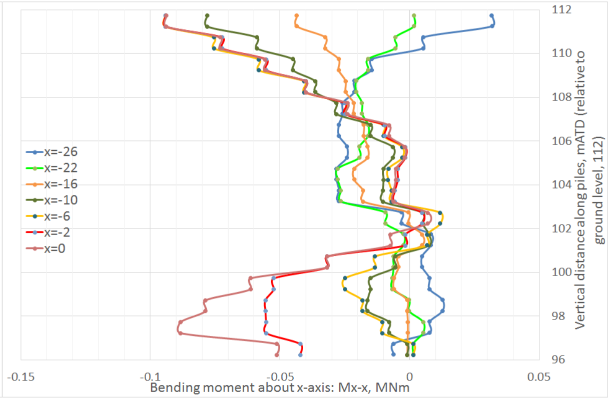
(a) *E=56e6 kN/m²*, $M_{y\text{-}y}$



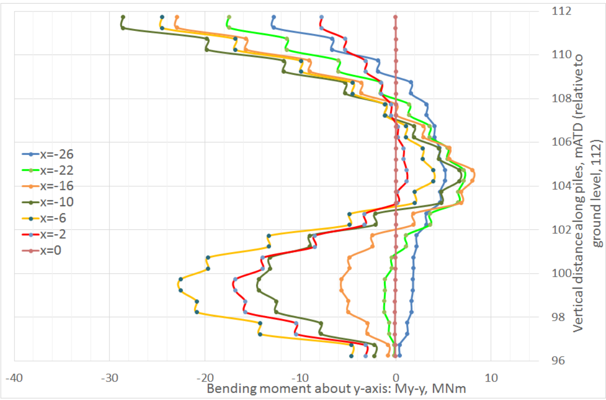
(b) *E=56e6 kN/m²*, $M_{x\text{-}x}$



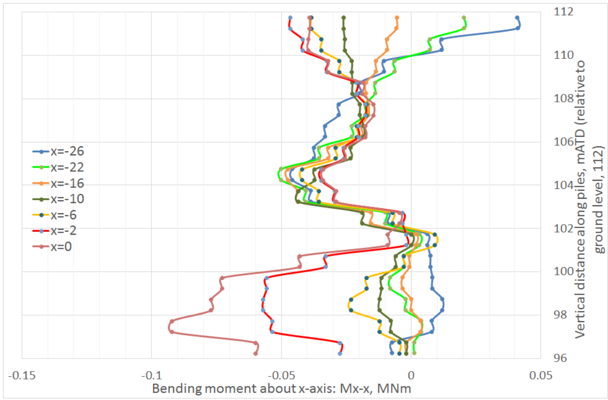
(c) *E=28e6 kN/m²*, $M_{y\text{-}y}$



(d) *E=28e6 kN/m²*, $M_{x\text{-}x}$



(e) *E=3.5e6 kN/m²*, $M_{y\text{-}y}$



(f) *E=3.5e6 kN/m²*, $M_{x\text{-}x}$

Figure 5.3: Bending moment plots

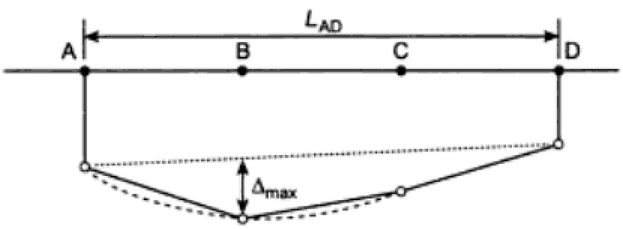


Figure 5.4: Deflection ratio diagrammatic definition, Burland and Wroth [1974]

5.5 Discussion

Comparing bending moments about both axis for the flexible and stiff pile cap, it can be seen that the bending moments are larger for the stiff scenario, especially for bending about the *x-axis*. This reflects the fact that the stiff foundation is under a greater stress as it resists the extra deformation seen in the flexible case; the ability for the flexible case to deflect causes a reduction in the stress that it experiences. The graphs show that, generally the pile directly above the tunnel, $x=0$, experiences the greatest $M_{x\text{-}x}$ value, due to the largest settlements observed in this zone. The piles under greatest $M_{y\text{-}y}$ at the pile head are located midway between the centre of the pile-cap and the edge, suggesting this is the location where the steepest gradient of settlement trough occurs. The negative bending moment values suggest the pile head is under a compressive force due to tunnel deflections. Midway down the piles there is a transition from negative to positive bending moments, indicating the central section of the piles are experiencing tension. This could lead to the onset of pile cracking, which would represent an intriguing area of further research. However, the magnitude of the bending moments, which peak around *10kNm* in the tensile zone, would suggest pile cracking should not be an issue.

5.6 Summary

The following can be concluded from the 3D analysis:

- Increasing ground beam stiffness leads to decreasing settlement, though the effect is small, reinforcing the theory that KC is acting as a flexible building and foundation system.
- Bending moments about the *x-axis* are greatest directly above the centreline of the tunnel, which corresponds to the area of largest deflection of the pile cap.
- Bending moments around the *y-axis* are greatest midway between the edge and centre of the pile cap, corresponding to the location where the settlement trough has its largest gradient, or the point of inflection.
- Using the simplified model of KC would suggest that pile cracking should not be an issue for the foundation, though further analysis must be conducted in this area.
- The deflection ratio of the building does not differ hugely with varying stiffness, suggesting that the stiffness does not have a huge impact on the differential settlement, settlement profile and hence damage to the structure.
- Hand-calculations suggest that KC should follow similar behaviour to the "flexible' building case.

Chapter 6

Conclusions

6.1 Summary

This project has focused on building settlements due to tunnelling effects. Comparisons were made between field data from Crossrail and results produced from finite element modelling, citing possible reasons from any discrepancies. The results presented have provided useful data and insight into the effect of tunnelling on piled foundations, as well as reaffirming some known observations.

The primary objective of this research was to attempt to replicate field observations from Crossrail using FEM. A number of observations were identified in chapter 4 from the 2D finite element analysis conducted.

1. The two-dimensional finite element model was capable of successfully modelling greenfield ground movements caused by tunnelling effects. Ground movements were successfully modelled for the case when a building is present, suggesting that the finite element model could be used for further analysis.

2. Settlements predicted by the model with the presence of a group pile foundation were larger than those predicted by greenfield analysis. Both surface settlement profiles exhibited Gaussian behaviour.

3. Façade stiffness is shown to make a small, yet significant, contribution to the overall building and foundation stiffness and be included in the analysis.

4. Building stiffness was shown to have an impact on the magnitude of surface soil settlements; larger building stiffness values exhibited smaller magnitude settlement profiles. For high stiffness values, this corresponded to a shallower, wider settlement trough, indicating a larger influence zone due to tunnelling for stiffer buildings. When decreasing the building stiffness from the parameters set-out in the initial analysis, there was a much smaller difference in the settlement values than for the

case when stiffness was increased. This suggests the building was already behaving in a flexible manner and was little affected by increasing its flexibility.

5. The finite element model gave settlement values larger than those observed in the field observations. Compensation grouting during the Crossrail project was suggested as reason for this, along with the possible effect of pile cracking causing a reduction in foundation capacity and subsequent increase in settlements.
6. Further analysis on the possible effect of tensile pile cracking suggested that this is unlikely to be the reason for increased settlement in the model. Modelling the effect of pile cracking in the finite element model appeared to cause a redistribution of load paths through the structure, but there appeared to be no increased settlement after the onset of cracking.
7. Analysing the stress distribution in the structure during pile cracking showed a redistribution of stresses in the structure, with some areas of the structure (including some of the piles) experiencing large increases in the magnitude of stress experienced.
8. For cases when the tunnel axis lay within the building area, larger settlements were observed compared to when the tunnel axis lay outside the building area. For this case, a rapid reduction to greenfield settlement values was observed, reinforcing the theory that Kempton Court exhibits a flexible response to tunnelling.
9. In general, the effect of tunnelling position appeared to have little effect on the building response.

In chapter 5, the findings of the three-dimensional analysis conducted on a group pile foundation subject to tunnelling effects from a single tunnel excavation were summarised, with a number of observations made.

1. Increasing the stiffness of a structure (ground beam) causes a slight reduction in settlements, with the converse observed for a decrease in stiffness. Building stiffness is found to have a limited effect on ground settlements for the stiffness range chosen.
2. For a piled foundation extending beyond a tunnel excavation, the bending moments induced in the piles about an axis perpendicular to the tunnelling direction are greatest directly above the tunnel centreline. This corresponds to the region of greatest displacement and stress. Bending moments about an axis parallel to the tunnelling direction are largest at the point of inflection of the settlement trough, about midway between the centre and extreme edge of the piles.
3. Building stiffness has little effect on the deflection ratio of the building, with the settlement profile shape similar for different values of building stiffness.

4. Hand-calculations suggest that KC should exhibit a flexible response, based on the E-value found, similar to that for the "flexible" building case.

6.2 Further areas of research

The 2D analysis was comprehensive and little would be gained from further analysis in this model. However, there remains some questions about the results observed from this model, notably the increased settlements predicted in the presence of the building when compared to greenfield values. These increased settlements were not seen in the field observations during Crossrail tunnelling, leading to a slight discrepancy between observed and predicted results. Possible explanations were presented; however, further analysis would be useful to add credibility to these claims.

The 3D analysis is still in its infancy. It would be useful to develop the model to more accurately model KC specifically, rather than a generic building such that better comparisons with field data could be made. Further analysis similar to that conducted in 2D would be useful in order to gain further insight and understanding into the mechanisms involved when a piled foundation is subject to tunnelling effects.

It would be useful to conduct further analysis into the subsurface behaviour of the soil, especially the soil-foundation interaction and possible effects of pile stiffness on the surrounding soil stiffness.

For both cases, it would be interesting to conduct analyses using just a single pile so that the relative behaviour between pile group foundations and singe piles may be better understood.

Chapter 7

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Appendix A

Risk Assessment Retrospective

All risks were fully considered throughout the course of the project. At the outset of the project, a risk assessment was conducted, discussing the possible sources of risk and ways of alleviating them. As a computer analysis based project, the greatest risks predicted were injuries caused from working at a screen with poor posture, staring at a screen for too long, or repetitive strain injury.

During the project, excessive times in front of a screen were avoided, ensuring a working environment that allowed good practice to be observed. There were no issues during the project associated with any of the risks and none to be reported.

Acknowledgements

I would like to thank my project supervisor, Dr Mohammed Elshafie, and my project advisor, Dr Michael Williamson, for their advice during the project. I am grateful for the help received by Anton Pillai at Arup Geotechnics and the software licenses provided by the company.