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Andrew Lochaden

June 2012
Dedication

To my parents, Gordon and Joan
Acknowledgements

I would like to sincerely thank my supervisor, Dr Eric R. Farrell, for his immense support and guidance over the past few years. He always made himself available to me at a moment’s notice for discussions on geotechnical (and rugby) matters, and it was much appreciated. Enjoy your “retirement”!

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Finally, I would like to thank my parents, Gordon and Joan. You have supported me in any way you possibly could, and given me every opportunity to succeed. Your dedication to your children is an inspiration.
Tunnel construction inevitably causes ground movements in the surrounding soil mass, which can have a negative effect on adjacent surface and sub-surface structures in the congested urban environment. Therefore in order to avoid costly delays, it is important to have a thorough understanding of the implications of tunnel projects, one of which is the phenomenon of arching. Arching is the variation in stress associated with ground movements, and is the primary topic under consideration in this research. The arching effect due to tunnel construction has been investigated through a combination of physical models in dry sand of the active trapdoor and of the advancement of a purpose-built miniature tunnel boring machine (MTBM) through a sand mass, and through numerical modelling using commercially available software.

The active trapdoor has consistently been used in the literature as a means of simplifying the stress-transfer mechanisms due to tunnel construction. For the purposes of this work, the arching on the trapdoor itself and in the surrounding sand mass was quantified using a load cell and a number of miniature earth pressure cells (EPCs), respectively. The cells were calibrated by the author using a Rowe cell. Although the measured trapdoor stresses were in good agreement with those measured in similar tests in the literature, they compared poorly with those obtained from a number of analytical solutions. It was concluded that the accepted development of the failure mechanisms in the literature is inconsistent with the results obtained from the EPCs, as both internal and external failure surfaces appear to develop immediately upon trapdoor yielding.

The arching effect due to the advancement of the MTBM through the sand mass was also quantified using EPCs, whilst both surface and sub-surface settlements were measured. The surface and sub-surface settlement troughs were well predicted by the respective solutions in the literature, whilst the variations in the trough width parameter with both depth and volume loss were identified. The three-dimensional behaviour of the arching zone as the tunnel face progressed towards and past the plane in which the miniature earth pressure cells were placed was observed, thereby indicating the importance of simulating the advancement of the tunnel face. The dependence of the magnitude of arching on both the degree of settlement and length of the shear planes was highlighted.
A two-dimensional finite element (FE) simulation of the laboratory trapdoor tests, and both a two and three-dimensional simulation of the laboratory MTBM tests were carried out in order to investigate the variations in the horizontal and vertical stresses. The vertical stress results obtained from the trapdoor FE simulation were compared with those from the laboratory tests. Although the Hardening Soil model provided a closer prediction than the Mohr-Coulomb model, the ability of both to predict the behaviour of the sand was limited. With regard to the MTBM tests, the relationship between the stresses and the distance from the MTBM face was assessed, and the importance of modelling the three-dimensional aspect of the problem was highlighted.
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1. Introduction

1.1 Background

The need for transport and services infrastructure has increased greatly in recent years due to many factors, including population growth and urbanisation. The economic success of a country is dependent on a well-developed infrastructure, of which tunnels are an important element. Despite the large costs related to their construction, tunnels provide many benefits, including those related to environmental issues and sustainable development. The Dublin Port Tunnel, for example, has dramatically reduced the quantity of heavy goods vehicles in Dublin city centre. This has resulted in improved air quality and less traffic congestion, whilst providing vehicles with quick and easy access to the national motorway network.

Tunnels are often constructed using tunnel boring machines (TBMs), during which process a degree of settlement occurs in the soil mass surrounding them. The magnitude of this settlement depends on many factors, such as the ground conditions, the tunnelling method and the quality of the workmanship. The settlement results in a stress transfer mechanism, known as the arching effect. Arching is the transfer of stress from yielding soil masses to non-yielding adjacent masses, and occurs due to shear stresses resisting the movement of the yielding soil mass. In order to avoid damage to existing services and structures (for examples, buildings, foundations, other tunnels), it is very important to have a good understanding of the settlements and stresses induced by the tunnelling process. This understanding can be obtained from the instrumentation of tunnels, which is very expensive. Alternatively, it can be obtained from physical and numerical models, which are less expensive and offer better control of test conditions. This thesis contributes to the understanding through a combination of physical models of an active trapdoor and of the construction of a tunnel through dry sand using a miniature TBM (MTBM), and through numerical modelling. The MTBM was designed by the author specifically for the purposes of this research.
1.2 Scope of study

The principal aim of this work is to develop the understanding of the arching phenomenon. This was investigated through physical modelling carried out at normal gravity and finite element (FE) analyses.

An active trapdoor has been utilised for many years to examine the arching effect due to the movement of the trapdoor away from the soil mass. It is a simplification of the stress distribution occurring around tunnels. Therefore, physical modelling of this problem was carried out to quantify the arching effect on the trapdoor itself and in the soil mass above and adjacent to it, using miniature earth pressure cells (EPCs) which were calibrated by the author. In the same way, the arching effect due to the more complicated problem of the advancement of an MBTM through the soil mass towards and past the plane in which the EPCs were placed was quantified. In addition, the magnitudes and properties of both the transverse and longitudinal settlement troughs were examined. Analysis of these settlement results and those results obtained from the miniature EPCs allows the stress transfer mechanisms within the arching zones to be described.

An FE investigation of the arching effect due to both an active trapdoor and tunnel advancement was carried out using commercially available software. The suitability of three soil models (namely the Mohr Coulomb, the Hardening Soil and the Hypoplastic model) in describing the behaviour of the sand mass was assessed, and stress transfer mechanisms occurring in the two scenarios were identified.

1.3 Thesis layout

In Chapter 2, a general introduction to tunnelling is given, followed by a discussion of the ground movements due to tunnelling and face stability. A subsequent summary of the existing literature related to the arching effect due to both trapdoor displacement and tunnel construction is presented.

The design of the test chamber, trapdoor configuration, and MBTM are discussed in Chapter 3. Details are provided of the instrumentation used throughout this research, and of the steps taken to calibrate them. Finally, details of both the trapdoor and MBTM test series are presented.
Chapter 1 Introduction

The effect of arching due to the vertical displacement of a circular trapdoor is discussed in Chapter 4. This discussion is based on a series of tests which investigated the force acting on the trapdoor itself, and in the soil mass above and adjacent to the trapdoor.

Chapter 5 presents and discusses the results of a series of tests carried out to investigate the displacements and associated arching effect due to the advancement of a MTBM through a sand mass. The characteristics of both the surface and sub-surface settlements are discussed, and the stress transfer mechanisms occurring above and adjacent to the tunnel centreline as the MTBM progresses towards and past the plane of measurement are described.

The results of a FE study carried out are presented in Chapter 6. Following a summary of the soil models used and the steps taken to obtain the necessary parameters, patterns in the stress transfer mechanisms identified from the FE study which occur due to the active circular trapdoor and tunnel construction are discussed.

Finally, the main findings and conclusions of the research are presented in Chapter 7. Recommendations for future research are also suggested.
2. Literature review

2.1 Introduction

The purpose of this chapter is to provide a summary of the current state of the art of tunnelling, and to provide background to the ground movements resulting from the construction of tunnels and the methods by which they are predicted. Following this, the stability of tunnels in both the drained and undrained condition is discussed. Finally, a review is presented of the theoretical and experimental work related to arching due to both an active trapdoor and tunnel construction.

2.2 Introduction to tunnelling

The last number of years has seen a significantly increasing number of tunnelling projects being carried out in loose ground. Many of these are in urban areas, and are often constructed using shielded tunnel boring machines (TBMs). Although open-face machines are sometimes used in stiff soils, the use of closed-face machines is more common. The purpose of the shielded TBM is to support the soil through which the tunnel is being excavated in order to reduce settlements and allow the construction of the final tunnel lining within. While the shield mechanically supports the soil radially along some distance from the tunnel face, there are a number of different methods of supporting the tunnel face. These are discussed below.

2.2.1 Face support systems

2.2.1.1 Compressed air

Compressed air may be used to control the flow of groundwater into the tunnel. However, this method has several disadvantages. Due to the linearly increasing water pressure at the face, the air pressure required at the crown of the tunnel will be less than that required at the invert. This means that the air pressure at the crown will be greater than the water pressure at the crown, causing air to be released. In tunnels with low cover, a blow-out may occur. This method is unsuitable for use in high permeability ground, and there are health and safety issues for the people working in this environment.
2. Literature review

2.1 Introduction

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2.2.1.2 Slurry tunnelling machines

Slurry tunnelling machines (STMs) use a bentonite fluid to support the tunnel face. A conceptual diagram is shown in Figure 2.1. The slurry is contained in a pressurised chamber, where it penetrates into the ground to form a mud cake. However, due to the small amount of openings in the cutting face, there is also considerable mechanical support. The size of these openings is altered to the expected maximum solids size. Once the soil has been excavated, the soil-slurry mixture is pumped to a separation plant, where recycling of the bentonite fluid takes place. Considerable difficulties and resulting costs may be experienced at the separation plant, particularly when dealing with silts and clays, which may be hard to separate. This machine is particularly effective in loose sand or gravel. There are environmental issues regarding the infiltration of the slurry into the soil mass.

![Figure 2.1 Conceptual diagram of STM](image)

2.2.1.3 Earth pressure balance machines

Earth pressure balance machines (EPBMs) are often used to construct tunnels in soft ground. A conceptual diagram is shown in Figure 2.2. A rotating cutter head excavates the soil, which then enters the extraction chamber through openings in the cutting face. The excavated soil is mixed with conditioning agents to form a mud, and the mud is removed from the excavation chamber by a screw conveyor. By balancing the volume of soil discharged from the screw conveyor with the volume of soil excavated by the cutter head, the pressure in the extraction chamber can be controlled. EPBMs are more suited to ground with a high percentage of fines, as this helps the formation of a seal in the screw conveyor.
In order to deal with the varied ground conditions which may be encountered on an individual project, TBMs have been developed which are based on a combination of the principles described above.

## 2.2.2 Selection of tunnel construction technique

The selection of the appropriate technique depends on a number of factors. These include:

- ground conditions (including permeability, shear strength, grading curve, etc.)
- hydrological conditions and groundwater level
- characteristics of tunnel required
- expected and allowable ground deformations
- proximity of tunnel to other structures (existing tunnels, foundations, services etc.)

The range of ground conditions for which STMs and EPBMs are suitable is shown below in Figure 2.3.

![Figure 2.2 Conceptual diagram of EPBM](image)

![Figure 2.3 Range of ground conditions for which STMs and EPBMs are suitable, from British Tunnelling Society (2005)](image)
2.3 **Ground movements**

The construction of tunnels results in a change in the state of stress in the surrounding soil, which leads to ground movements. The process of tunnel construction is a three-dimensional problem - deformations occur in both the transverse direction (perpendicular to the tunnel axis) and the longitudinal direction (parallel to the tunnel axis). The successful prediction of these ground movements is an important part of the design process, in order to protect existing structures, buildings and services.

### 2.3.1 Sources of ground movements

For shield tunnelling, Cording (1991) described five major components of ground movements:

- Movements towards the face due to stress relief
- Movements around the shield due to over-cutting in order to allow the shield to pass, as well as any ploughing or yawing of the machine
- Movements due to the tail void, which are due to the gap between the shield and the lining
- Movements due to deflection of the lining as the ground load develops
- Long-term movements due to consolidation

### 2.3.2 Deformations in the transverse direction

#### 2.3.2.1 Vertical settlement trough

The settlement trough above and ahead of a tunnel constructed in a “greenfield” environment may be well approximated by a Gaussian distribution (Martos 1958, Peck 1969). Although this was based on field observations and has no theoretical basis, it is a useful idealisation. The vertical settlement at any distance from the centreline ($S_v$) may be found using the following equation:

$$S_v = S_{v,\text{max}} e^{\left(-\frac{x^2}{2i^2}\right)}$$  \hspace{1cm} \text{Eqn. 2.1}

where $S_{v,\max}$ is the maximum settlement on the tunnel centreline, $x$ is the horizontal distance from the tunnel centreline and $i$ is the distance from the tunnel centreline to the point of inflection of the settlement trough, as shown in Figure 2.4.
Vorster (2005), from the results of centrifuge tests on tunnels in sands, observed that the traditional Gaussian curve predicts less settlement at the shoulders of the trough than observed in the centrifuge tests. The following modified Gaussian curve was proposed in order to improve the fit of the curve to the data:

\[ S_v = \frac{S_{v,max} n}{n - 1 + e^{\left[ \frac{\alpha}{n} \right]}} \quad n = 1 + \frac{e^{\alpha(2\alpha - 1)}}{2\alpha + 1} \]

Eqn. 2.2

When \( \alpha = 0.5 \), the modified Gaussian curve corresponds to the traditional Gaussian curve:

### 2.3.2.2 Volume loss

The volume of the traditional Gaussian settlement trough (\( V_s \)) was derived by Attewell and Farmer (1974):

\[ V_s = \sqrt{2\pi} i S_{v,max} \]

Eqn. 2.3

It is useful to express \( V_s \) as a proportion of the cross-sectional area of the tunnel. This is termed the volume loss (\( V_L \)), and is usually expressed in percentage form.

\[ V_L = \frac{V_s}{\pi D^2} \]

Eqn. 2.4

where \( D \) is the tunnel diameter. The volume loss is dependent on many factors, including the ground conditions, groundwater conditions, tunnelling method and quality of workmanship. It allows the comparison of the settlements resulting from the construction of tunnels of various diameters. Melis et al. (1997) reported volume losses of 2-4% for an EPB machine in mixed face conditions. Mair (1996) presented a review of the reported volume losses from many papers. These included:
Chapter 2  Literature review

- For open-face tunnelling in stiff clays, $V_L = 1-2\%$
- For EPB and slurry shields, volume losses can be as low as 0.5\% in sands, and of 1-2\% in soft clays (not including consolidation settlements)

2.3.2.3 Settlement trough width

Clough and Schmidt (1981) related the width of the settlement trough to the diameter and depth of the tunnel:

$$\frac{2l}{D} = \left(\frac{z_0}{D}\right)^2$$  \hspace{1cm} \text{Eqn. 2.5}

O’Reilly and New (1982) showed that the point of inflection ($i$) was linearly related to the distance from the surface to the tunnel centreline ($z_0$). This led to the following relationship:

$$i = K_s z_0$$  \hspace{1cm} \text{Eqn. 2.6}

where $K_s$ is the surface trough width parameter. It, as well as the sub-surface trough width parameter ($K_{s-s}$), are of considerable importance when assessing the impact of tunnel construction on adjacent surface and sub-surface structures, such as buildings, foundations and existing tunnels, as the point of inflection defines the width of the trough and the boundary between hogging and sagging deformation. Unlike $i$, the trough width parameter is a dimensionless value and is commonly used in the literature to compare the settlement troughs at various heights above a tunnel. $K_s$ may be taken as approximately 0.5 for clays and 0.25 for sands and gravels (O’Reilly and New, 1982), although for tunnels constructed in sands, this only applies for $z_0 = 6-10\text{m}$. However, Mair and Taylor (1997) show that there is a significant amount of scatter in the data. For tunnels in sands and gravels, $K_s$ ranged from 0.25-0.45, with an average of 0.35, whilst the $K_s$ values for tunnels in clays range from 0.4-0.6, with an average of 0.5.

Due to the effect of tunnel construction on other underground structures (such as existing tunnels, foundations etc.), it is important to be able to predict subsurface settlements. In a similar way to surface settlement profiles, Mair et al. (1993) demonstrated that subsurface settlement profiles may be reasonably well approximated by a Gaussian distribution. For subsurface profiles, $i$ is given by:

$$i = K_{s-s}(z_0 - z)$$  \hspace{1cm} \text{Eqn. 2.7}

where $z$ is some distance below the ground surface. $K_{s-s}$ may be found from the following equation (Mair et al., 1993):
Chapter 2 Literature review

The value of $K_{s-s}$ increases with depth, resulting in a wider trough than would be the case if a constant value of $K_{s-s}$ was used. This agrees with field data presented by Moh et al. (1996) and Dyer et al. (1996). Jacobsz (2002) adapted Eqn. 2.8 to take into account lower trough width parameters measured at the surface, based on the results of centrifuge tests in sand, and presented the following equation:

$$K_{s-s} = \frac{0.175 + 0.325 \left( 1 - \frac{Z}{z_0} \right)}{1 - \frac{Z}{z_0}}$$  \hspace{1cm} \text{Eqn. 2.8}

where $K_{s} = 0.35$ and $\frac{\delta i}{\delta z}$ is the rate of change of $i$ with depth ($=0.26$).

Vorster (2005) and Marshall (2009) found from the results of centrifuge tests in sands that the trough width parameter decreases as the volume loss increases (Figure 2.5). In this figure, $z_i$, $z$ and $D_i$ are the depth to tunnel axis, the depth of the settlement trough at which $i$ is measured, and the diameter of the tunnel face, respectively. The latter author concluded that this was due to the development of the chimney failure mechanism (discussed in § 2.3.4). Cording (1991) found from field data that the settlement trough width depended on the magnitude of volume loss - narrower troughs were found for larger volume losses.

![Figure 2.5](image)

**Figure 2.5 Relationship between the trough width parameter and volume loss, from Marshall (2009)**

2.3.2.4 Horizontal ground movements

Horizontal ground movements can result in damage to structures and services. For tunnels in clays, Attewell (1978) and O’Reilly and New (1982) suggested that ground
displacement vectors are directed towards the tunnel axis, resulting in the following equation:

$$S_h = \frac{x}{z_o} S_v$$  \hspace{1cm} \text{Eqn. 2.10}

where $S_h$ is the horizontal ground movement. This assumption allows the distribution of surface horizontal ground movements to be found from:

$$\frac{S_h}{S_{h,max}} = 1.65 \frac{x}{l} e^{\left(\frac{-x^2}{2l^2}\right)}$$  \hspace{1cm} \text{Eqn. 2.11}

For tunnels constructed in sands, the horizontal ground movements near the edge of the settlement trough ($2i < x < 3i$) may be significantly underestimated by the assumption that ground movements are directed towards the tunnel axis (Cording, 1991). Mair et al. (1997) noted however, that ground movements (both vertical and horizontal) are very small in this zone and therefore relatively insignificant.

### 2.3.3 Deformations in the longitudinal direction

Assuming the ground deformations take place at constant volume (i.e. for tunnelling in clays), the longitudinal settlement trough is of the form of a cumulative probability curve, as shown in Figure 2.6 (Attewell and Woodman, 1982). For tunnels constructed in stiff clays without face support, this was shown to be reasonably valid. It was also identified that the surface settlement directly above the tunnel face was approximately $0.5S_{v,max}$. However, this was considerably less for tunnels constructed in soft clays with face support. Mair et al. (1997) pointed out that pressurised face tunnelling resulted in much smaller settlements ahead of the tunnel face, and that for EPB and slurry shield tunnelling, the settlement is mainly due to the tail void. This leads to a translation of the cumulative curve (Figure 2.6), which allows for these smaller settlements ahead of the tunnel face, as opposed to the case without face support in which greater deformations ahead of the tunnel face result. It should also be highlighted that while it is reasonable to assume that the longitudinal settlement trough has the form of the cumulative probability curve, it has only been validated for tunnels in clays.
Chapter 2

The equation for the settlement in the longitudinal direction \( S_{v,\text{long}} \) was given by Attewell and Woodman (1982) as:

\[ S_{v,\text{long}} = \frac{V_s}{\sqrt{2\pi i}} e^{-\left(\frac{y^2}{2i^2}\right)} \left[ G\left(\frac{x-x_i}{i}\right) - G\left(\frac{x-x_f}{i}\right) \right] \]

Eqn. 2.12

where \( x \) is the point at which the settlement is being calculated, \( x_i \) and \( x_f \) are the initial and current positions of the face in the longitudinal direction, respectively, \( y \) is the distance from the tunnel centreline in the transverse direction, and \( G[(x-x_i)/i] \) and \( G[(x-x_f)/i] \) are integrated normal probability functions. This is provided in tabular format by the authors in their work.

2.3.4 Failure mechanisms

Mair and Taylor (1997) discussed the differences in the geometries of the failure mode for tunnels in clays and sands, as shown in Figure 2.7. The mechanism of failure in clays propagates upwards and outwards from the tunnel invert. The zone of failure becomes much wider than the tunnel. The mechanism of failure for sands is quite different. Failure in this case involves a narrow “chimney”, which propagates vertically. This may or may not reach the ground surface, depending on the depth of the tunnel. Experimental observations of the chimney failure mechanism in sands are discussed in § 2.5.3.
2.4 Face stability

In order to provide a safe working environment for construction workers and to restrict ground movements, the stability of the tunnel is highly important. The stability problem is treated as either drained or undrained. This is dependent on a number of factors including the properties of the soil, the dimensions of the tunnel and the speed at which the tunnel is advanced (Mair and Taylor, 1997). Anagnostou and Kovari (1996) concluded on the basis of the results of numerical parametric studies that drained conditions are to be expected when the permeability of the soil is higher than $10^{-7}$-$10^{-6}$ m/sec and the rate of excavation is 0.1-1 m/hr or less. Therefore, generally speaking, undrained stability is more relevant for low permeability clayey soils and drained stability is more relevant for sandy soils.

2.4.1 Undrained stability

Broms and Bennermark (1967) defined the stability ratio as:

$$N = \frac{\sigma_S + \gamma z - \sigma_T}{S_u}$$

Eqn. 2.13

where $\sigma_S$ is the surface surcharge pressure, $\gamma$ is the unit weight of the soil, $\sigma_T$ is the tunnel support pressure and $S_u$ is the undrained shear strength at the level of the tunnel axis.

Based on the results of finite element (FE) analyses and centrifuge testing, Mair et al. (1981) and Mair (1989) proposed that volume loss should be related to the load factor ($F_{load}$), which was defined as:

$$F_{load} = \frac{N}{N_c}$$

Eqn. 2.14
where \( N_c \) is the stability ratio at collapse. This parameter may be found from a series of curves which use the ratios \( P/D \) and \( C/D \), where \( P \) is the length of unsupported tunnel and \( C \) is the distance from the ground surface to the crown of the tunnel, as shown in Figure 2.8.

![Figure 2.8 Unsupported tunnel heading](image)

This approach is useful because it takes into account the three dimensional nature of the tunnel heading. Macklin (1999) extended this work to find a relationship between volume loss and load factor for overconsolidated clays, by compiling case history data. 15 of the 22 case studies were at London Clay sites. The conclusion of this study was to allow an estimate of the volume loss to be expected for tunnels with a \( C/D \) ratio in excess of 1 and for a load factor in excess of 0.2. Upper and lower bounds were shown. A regression line for the data from the 22 case studies is described by Eqn. 2.15:

\[
V_L = 0.23e^{4.4(F_{load})} \text{ for } F_{load} \geq 0.2
\]

**2.4.2 Drained stability**

Based on the assumptions that a zone of plastically yielding soil extends radially from the tunnel to the ground surface and that axisymmetric conditions exist around the tunnel, Atkinson and Cairncross (1973) defined the following lower bound solution for the tunnel support pressure for a shallow tunnel in a weightless granular soil loaded by a uniform surcharge:

\[
\sigma_T = \sigma_s \left( \frac{z_0}{R} \right)^{1-\mu'} \quad \mu' = \frac{1 + \sin \varphi_p'}{1 - \sin \varphi_p'}
\]

**Eqn. 2.16**

where \( R \) is the tunnel radius and \( \varphi'_p \) is the peak angle of shearing resistance.
Chapter 2  Literature review

Potts (1976) developed this work and with the same assumptions, presented a lower bound solution for the stability of a shallow tunnel in a non-weightless soil which is loaded by a uniform surcharge. Atkinson and Potts (1977) simplified this to:

\[ \sigma_T = \frac{R}{(\mu' - 2)^2} \left\{ \left( \frac{R}{z_0} \right)^{\mu'-2} \left[ \left( \frac{\sigma_T}{z_0} + \frac{2\gamma}{\mu} \right) (\mu' - 2) + \gamma \right] \right\} \]

Eqn. 2.17

The authors also developed an upper bound solution based on a collapse mechanism involving a wedge at the crown of the tunnel collapsing:

\[ \sigma_T = \frac{2\gamma R}{4 \cos \varphi'} \left( \frac{1}{\tan \varphi'} + \varphi' - \frac{\pi}{2} \right) \]

Eqn. 2.18

where \( \varphi' \) is the angle of shearing resistance. The experimental data, as well as the plasticity solutions, of Atkinson and Potts (1977) show that the support pressure is independent of \( C/D \). A lower bound was also defined as follows:

\[ \frac{\sigma_T}{2\gamma R} = \frac{\mu'}{\mu'^2 - 1} \]

Eqn. 2.19

Atkinson and Mair (1981) developed a lower bound method for finding the support pressure for a two-dimensional tunnel in granular soil. Two equations are presented, depending on the magnitude of the surface surcharge. If the surcharge is large enough, the weight of the soil becomes negligible.

\[ \sigma_T = \gamma DT_y \quad \text{for} \sigma_S = 0 \]

Eqn. 2.20

\[ \sigma_T = \sigma_S T_S \quad \text{for} \sigma_S \gg 0 \]

Eqn. 2.21

where \( T_y \) is a dimensionless number related to the angle of shearing resistance of the soil and \( T_S \) is a dimensionless number related to both \( \varphi' \) and \( C/D \).

Leca and Dormieux (1990) present upper and lower bound plasticity solutions for the stability of the three-dimensional tunnel heading \((P/D=0)\) in dry sand. The support pressure was found to be only dependent on the surface surcharge for very shallow tunnels. Their findings included the fact that the support pressure required to prevent collapse was independent of the \( C/D \) ratio. The upper bound solution was found to agree well with the results of the centrifuge testing carried out by Chambon and Corte (1989). Similarly, Anagnostou and Kovari (1996) found that the support pressure was independent of \( C/D \). This finding came as a result of a limit equilibrium solution for a lined tunnel heading which the authors derived, based on a wedge and a prismatic body. This is expressed in terms of the effective shear strength parameters, \( c' \) and \( \varphi' \), where \( c' \) is the cohesion of the soil.
Chapter 2

2.5 Arching

The effect of arching was first identified by French military engineers in the design of silos (Feld, 1948). It was noted that the base of the silo supported only a fraction of the total mass above it and that the side walls carried a much greater load than expected. This has led to a considerable amount of research into the area of arching. As it is a simplification of the more complex stress distribution around a tunnel, the active trapdoor has been used for many years to investigate the arching effect. Therefore, many of the theoretical solutions which have been proposed in the literature have been based on it.

The active trapdoor (i.e. movement away from the soil mass) has been used to investigate the arching effect for many years. It is a simplification of the stress distribution occurring around tunnels. Some of the work in the literature, both theoretical and experimental, is discussed below.

2.5.1 Theoretical solutions

Many of the theoretical solutions of the arching effect have been based on an active trapdoor, which has been utilised for many years to examine the arching effect due to the movement of the trapdoor away from the soil mass. It is a simplification of the stress distribution occurring around tunnels. Therefore, physical modelling of this problem was carried out to quantify the arching effect on the trapdoor itself and in the soil mass above and adjacent to it, using miniature earth pressure cells (EPCs) which were calibrated by the author.

Engesser (1882) presented one of the earliest theories on arching, when he suggested that the arch is parabolic, with a tangent through the edges of the trapdoor at an angle $\phi'$ with the horizontal. The vertical load acting on the trapdoor after undergoing a displacement then consists of the weight of soil below the arch and the vertical stress induced by the increased lateral stress just above the trapdoor which is influenced by the material above the arch. From a free body diagram of the imaginary structural arch, an equation for the effective load acting on the trapdoor was presented (Eqn. 2.22); however, this was valid only for cases where $H_{OB}/B>1.5$, where $H_{OB}$ is the height of overburden and $B$ is the width of the trapdoor.
where $K_a$ is the coefficient of active earth pressure.

Janssen (1895), translated by Sperl (2006), derived a method for the design of silos holding granular material, known as Janssen’s Silo Theory. Janssen investigated the forces acting on a horizontal element within a cylindrical silo. The theory is based on two assumptions: firstly, that the coefficient of lateral earth pressure $K_{ep}$ is the same at all depths in the silo, and secondly, that the material settles enough to develop shear stresses over the full height of the silo. An equation was presented which allows the force acting on the base of the silo to be calculated:

$$F = B^2\gamma \left( \frac{H_0 K_a}{2H_T \tan \varphi} + \frac{\tan \varphi}{6} \right)$$  \hspace{1cm} \text{Eqn. 2.22}

Marston and Anderson (1913) investigated the loads acting on buried conduits, and found that the load acting on them was less than the weight of the soil above it. The authors identified that this was due to the arching effect, and that it depended on the relative movement of the soil surrounding the conduit. Formulae for the vertical load acting on various types of conduits were presented. These were found by equating the upward and downward forces acting on a horizontal slice above the conduit, and have been derived by Spangler and Handy (1982). The vertical force ($F$) at the top of the conduit may be found from Eqn. 2.24, where $\mu'$ is defined in Eqn. 2.16:

$$F = \gamma B^2 \left[ 1 - e^{-2K_{ep} \mu' \left( \frac{H_0 R}{B} \right)} \right]$$  \hspace{1cm} \text{Eqn. 2.24}

Bierbaumer (1913) suggested an expression for the minimum load acting on a tunnel roof (and therefore a trapdoor) by considering the equilibrium of a triangular prism whose sides makes an angle of $\varphi$ with the vertical (Figure 2.9). The ultimate load was derived by considering a soil mass bounded by two vertical failure planes with side friction along the planes. The vertical normal stress was assumed to be linearly distributed.
The minimum and ultimate loads acting on the tunnel \( (F_{\text{min}} \text{ and } F_{\text{ult}} \text{, respectively}) \) can then be found from:

\[
F_{\text{min}} = \frac{\gamma B^2}{4\tan \varphi} \quad \text{Eqn. 2.25}
\]

\[
F_{\text{ult}} = \gamma (H - K_d H_0 B \tan \varphi') \quad \text{Eqn. 2.26}
\]

Although Terzaghi (1936) identified curved sliding surfaces which were slightly wider at the sand surface than the trapdoor width in his experimental work, for the purposes of his theoretical approach (1943), sliding surfaces rising vertically from the trapdoor corners were assumed, as shown in Figure 2.10.
As the experimental results suggested that soil stresses at a distance more than 2B-2.5B above the trapdoor were not affected by the trapdoor displacement, Terzaghi assumed that shearing resistance was only mobilised along the lower part of the sliding surface, the remaining overburden then acting as a surcharge. The load acting on the trapdoor can be found from:

\[ F = \frac{B^2(\gamma - \frac{2c}{B})}{2K_{ep}\tan\varphi} \left(1 - e^{-4K_{ep}\tan\varphi}\right) + B[\sigma_5 + \gamma(H - 2B)]e^{-4K_{ep}\tan\varphi} \]  

Eqn. 2.27

Amongst the assumptions made is that one of the parameters used in the analysis, the coefficient of lateral earth pressure \(K_{ep}\), is treated as an empirical constant, which increases from a value of 1 at the trapdoor to 1.6 at a height of 1B-1.5B above the trapdoor, and then reduces to the coefficient of earth pressure at rest \(K_0\) at heights above the trapdoor greater than 2.5B, where

\[ K_0 = 1 - \sin\varphi \]  

Eqn. 2.28

Krynine (1945) suggested that the use of \(K_a\) in Terzaghi’s approach is only valid when there is no friction on the vertical failure planes. Instead, he proposed that an alternative value of \(K_{ep}\) can be determined from the Mohr Circle as follows:

\[ K' = \frac{1 - \sin^2\varphi}{1 + \sin^2\varphi} = \frac{\cos^2\varphi}{1 + \sin^2\varphi} \]  

Eqn. 2.29

where \(K'\) is Krynine’s coefficient of earth pressure.

Finn (1963) presented closed form solutions for the change in vertical stress resulting from the translation or rotation of a trapdoor. The analysis was restricted to problems where the soil displacements were very small and elastic. The bottom horizontal boundary is considered rigid, with a trapdoor located in it, and is overlain by a linear elastic medium of infinite height with no self weight. The theory yields infinite tensile stresses occurring at the edges of the trapdoor and infinite compressive stresses on the base next to the door, as shown in Figure 2.11. The results provided by the theory were found to compare well to published results, but only for very small trapdoor displacements.
Chelapati (1964) extended the work of Finn (1963) by considering a soil field of finite height with self weight. The author overcame the problem of infinite stresses at the trapdoor edge by assuming the actual vertical stress on the door to be zero when tensile stresses are predicted (as granular soils cannot sustain tension). Accordingly, compressive stresses adjacent to the trapdoor on the base were reduced so that there was no subsequent net change in the total vertical force on the bottom horizontal boundary.

Davis (1968) provided an upper bound on the collapse load for the trapdoor problem by assuming that the velocity field at collapse can be modelled by a mechanism of one rigid block (Figure 2.12). He presented a kinematically admissible mechanism by equating the work done by external loads to the internally dissipated energy. He also provided a lower bound. These upper and lower bounds enclose the stability number quite closely for $0<H_{OB}/B\leq 2$, but are not good for $H_{OB}/B>2$. Davis's upper bound solution was later improved by Gunn (1980) who used an increased number of rigid blocks (Figure 2.13). Gunn used the expansion of a thick cylinder in homogeneous soil to construct a statically admissible stress field, in order to derive a lower bound. Gunn’s upper and lower bound solutions were an improvement on Davis’s for trapdoors with $H_{OB}/B>2$ and $H_{OB}/B>3.5$, respectively.
Ladanyi and Hoyaux (1969) derived a method to estimate the pressure reduction on the trapdoor. They did this by modifying classical silo theory, and by assuming two vertical failure planes extending from the edges of the trapdoor to the sand surface. As the experimental results were similar to the calculated results, it was concluded that the assumption of the failure planes was correct. However, it was also acknowledged that this prediction was only possible when the lateral stress ratio and normal stresses acting on the vertical failure plane are properly taken into account.
Bjerrum et al. (1972) extended the work of Chelapati (1964) to provide values of the change in vertical pressure ($\Delta \sigma_v$) acting on the centre of a flexible trapdoor located in a rigid horizontal boundary, using Eqn. 2.30.

$$\Delta \sigma_v \approx 2a_B \frac{\delta}{B} E_s$$  \hspace{1cm} \text{Eqn. 2.30}

$E_s$ is the Young’s Modulus of the soil and $a_B$ is a dimensionless parameter which is a function of a number of different parameters, including $H_{OB}$, $B$ and $E_s$. This extension, however, is only valid for small variations in vertical pressure. Therefore, it is of limited use as the variations seen experimentally are generally large.

A differential equation for an active trapdoor force was proposed by Vardoulakis et al. (1981), based on shear band inclinations which were taken directly from small scale model tests. The equation, which is independent of whether plane strain or axisymmetric conditions exist, was derived by integration of the equilibrium conditions along horizontal slices and using the mean value for the vertical stresses.

Evans (1983) used in his analysis a frictional, cohesionless, isotropic and homogeneous granular material with height, $H_{OB}$ and unlimited lateral extent. The Coulomb failure criterion and the associative flow rule were used. The angle of dilation ($\psi$) was highlighted as a major factor in governing the deformation of a soil body around a trapdoor. It was discovered from a two dimensional analysis that, theoretically, when the angle of dilation is greater than zero (and less than $\phi$), the failure zone above the trapdoor is a triangular prism. When the angle of dilation is zero, the failure zone is rectangular, as observed by Terzaghi (1936). This work was then extended to a three dimensional analysis - the shape of the failure zone for both circular and rectangular trapdoors were identified and are shown in Figure 2.14 and Figure 2.15 respectively. The minimum force ($F_{min}$) acting on the trapdoor may be found from either Eqn. 2.31 or Eqn. 2.32.

$$F_{min} = \frac{\pi D_T^2}{24 \tan \psi} \sqrt{\frac{D_T}{H}} , \text{ for } \frac{D_T}{H} \leq 2 \tan \psi$$  \hspace{1cm} \text{Eqn. 2.31}

$$F_{min} = \frac{\pi D_T^2}{24 \tan \psi} \left( \sqrt{\frac{D_T}{2 \tan \psi}} - H \right)^3 , \text{ for } \frac{D_T}{H} > 2 \tan \psi$$  \hspace{1cm} \text{Eqn. 2.32}

where $\psi=\phi$, in accordance with the flow rule. Eqn. 2.33 can be used to find the ultimate force ($F_{ult}$) acting on it. A $K_{ep}$ value of 1.2 was measured experimentally and used by Evans (1983) in his solution.
\[ F_{ult} = \frac{\pi \gamma D_{TD}^3}{4} \left[ 1 - e^{-8K_{ep} \sin \varphi} \frac{4K_{ep} \sin \varphi}{4K_{ep} \sin \varphi} + \left( \frac{H_{OB}}{D_{TD}} - 2 \right) e^{-8K_{ep} \sin \varphi} \right] \]

Eqn. 2.33

The results of Evans’ theoretical work were compared to both his experimental results and those obtained from the existing solutions. It was found that Evans’ work provided an improved description of the arching effect and the associated ground deformations when compared to those obtained from the solutions in the literature.

Figure 2.14 Prism of soil for active arching above a circular trapdoor, from Evans (1983)

Figure 2.15 Prism of soil for active arching above a rectangular trapdoor, from Evans (1983)

Koutsabeloulis and Griffiths (1989) carried out a FE analysis of the trapdoor problem, and considered the soil to be an elastic-perfectly plastic material. A Mohr-Coulomb failure criterion was used with a non-associated flow rule. Both the active and passive modes were modelled, and the results of the parametric studies were presented in the form of influence
charts. For the active case, two formulae for the vertical stress acting on the trapdoor ($\sigma_{TD}$) were presented, depending on whether the trapdoor was shallow or deep:

\[
\sigma_{TD} = \gamma H_{OB} \left[ \left( R_{G1} \right) G_{1} \left( \frac{H_{OB}}{D_{TD}} \right) \tan \varphi \right], \quad \text{for } \frac{H_{OB}}{D_{TD}} \leq 2.5 \tag{Eqn. 2.34}
\]

\[
\sigma_{TD} = \gamma H_{OB} G_{2} \left( R_{G2} \right) G_{1} \left( \frac{H_{OB}}{D_{TD}} \right) \tan \varphi, \quad \text{for } \frac{H_{OB}}{D_{TD}} > 2.5 \tag{Eqn. 2.35}
\]

$R_{G1}$, $R_{G2}$, $G_{1}$, $G_{2}$ and $G_{3}$ are parameters which are a function of $\varphi$, and may be found from charts presented in Koutsabeloulis and Griffiths (1989).

Sloan et al. (1990) derived upper and lower bounds on the load required to support the trapdoor against active failure using numerical techniques as well as the limit theorems of plasticity. These were found to significantly improve the bounds which had been presented at that time - the authors found that their bounds determined the exact solution to within less than 10%.

### 2.5.1.1 Adaptation of analytical solutions to axisymmetric conditions

Many of the solutions discussed in § 2.5.1 are only applicable to plane strain conditions, but Santichaiananant (2002) presents an adaptation of some of these to satisfy axisymmetric conditions. These are discussed below.

The work of Engesser (1882) was altered by using a parabolic dome rather than a structural arch to determine the force, yielding Eqn. 2.36:

\[
F_{\text{min}} = D_{TD}^{3} \gamma \left( \frac{K_{a} \tan \varphi H_{OB} \pi}{8H_{OB} + K_{a} D_{TD} \tan \varphi} + \frac{\pi}{32} \tan \varphi \right) \tag{Eqn. 2.36}
\]

The original solution of Bierbaumer (1913) which considered the equilibrium of a triangular prism has been adapted to the axisymmetric case by considering conical and cylindrical failure shapes for the minimum and maximum forces respectively. This leads to Eqn. 2.37 and Eqn. 2.38.

\[
F_{\text{min}} = \frac{\gamma \pi D_{TD}^{3}}{24 \tan \varphi} \tag{Eqn. 2.37}
\]

\[
F_{\text{ult}} = \frac{\pi D_{TD} H_{OB}}{2} \left( \frac{D_{TD}}{2} - K_{a} H_{OB} \tan \varphi \right) \tag{Eqn. 2.38}
\]

At a certain depth $H$, the side friction acting along the shear planes bounding the cylindrical failure shape is greater than the cylinder’s self-weight, resulting in a negative
value of $F_{ult}$. Santichaianant (2002) showed that the maximum ratio of $H_{OB}/D_{TD}$ which can be used before a negative value of $F_{ult}$ is obtained is given by Eqn. 2.39:

$$\frac{H_{OB}}{D_{TD}} = \frac{1}{2K_{d} \tan \phi} \quad \text{Eqn. 2.39}$$

The solution presented by Terzaghi (1936) was adapted by considering a circular element rather than a rectangular one. This led to Eqn. 2.40:

$$F_{ult} = \left(\frac{\pi D_{TD}^{2}}{4}\right) \left\{ \frac{\gamma D_{TD}}{4K_{ep} \tan \phi} + \left[ \gamma(H_{OB} - 2D_{TD}) - \frac{\gamma D_{TD}}{4K_{ep} \tan \phi} \right] e^{-8K_{ep} \tan \phi} \right\} \quad \text{Eqn. 2.40}$$

2.5.2 Experimental work – trapdoor

2.5.2.1 Trapdoor tests carried out at normal gravity

One of the first to carry out trapdoor tests was Engesser (1882). His relatively simple set-up involved the use of a traditional balance beam weighing scales. On one side of the scales was the trapdoor of a 950mm x 200mm rectangular bin, whilst a series of counterweights sat on the other side. The bin was filled with sands to varying heights, while ensuring the trapdoor remained flush with the base of the bin by altering the weights on the other side of the balance. The trapdoor was then lowered by removing the weights on the balance, and the mass of sand that fell from the bin was measured. This was carried out for three overburden heights. Engesser concluded that, in order for arching to occur, the depth of overburden had to be 1.5 times greater than the trapdoor width:

Terzaghi (1936) presented the results of experimental investigations, as carried out by Kienzl. A trap-door of width (B) 73mm and length 463mm was contained in the base of a box containing sand to a height (h) of 310mm. The box had a length of 463mm. Tests were conducted using both loose and dense sand. The trap-door was gradually lowered, while the load acting on it and its displacement were measured. Typical results are shown in Figure 2.16. As the door was lowered, the load acting on it decreased quickly, reaching a minimum at approximately 0.5% and 1.0% of the trapdoor width for dense and loose samples respectively. This minimum value was less than 10% of the initial overburden pressure, and was lower for dense sand than for loose, i.e. the arching effect was more pronounced for dense samples. The arching effect then reduced as the door was further lowered, and the load acting on it became constant (approximately 13% of the total overburden pressure) for displacements of approximately 11% of the trapdoor width. The
vertical and horizontal stresses in the sand mass above the trap-door were measured using the friction tape method. This consists of pulling a steel tape from between two adjacent steel tapes. The force required to do this can then be related to the stress acting at that point in the sand mass. The change in these stresses as the trapdoor was translated downwards was investigated - the results are shown in Figure 2.16 and Figure 2.17. The variation of the coefficient of lateral stress normalised with respect to $K_0$ ($=0.5$) is shown in Figure 2.18. The vertical stresses decreased within $2.5B$ of the trapdoor, whilst the horizontal stresses increased above the trapdoor before decreasing. Terzaghi concluded that the lowering of the trap-door did not affect the sand mass above a height of $2B-2.5B$ from the trapdoor.

![Figure 2.16 Normalised load vs displacement for Terzaghi's experiments, from McNulty (1965)](image)

![Figure 2.17 Variation of vertical stress above centre of trapdoor for Terzaghi's experiments, from McNulty (1965)](image)
McNulty (1965) carried out experiments which were similar to those performed by Terzaghi (1936). A cylindrical test chamber with an internal diameter of approximately 1188mm was used, with a circular trapdoor located at its base. Two different trapdoors were used, with diameters ($D_{TD}$) of 76mm and 152mm. The sand was placed in a dense state. The stresses in the sand mass were increased by the use of air pressure at the surface, and the pressure acting on the trapdoor was measured. The effect of parameters such as the depth of sand, trapdoor diameter, surface pressure and various soil properties were investigated. From the load-displacement curves presented by the author, it was evident that there was a reduction in pressure acting on the trapdoor to an ultimate value. Unlike Terzaghi's observations, there was not a subsequent increase in the stress acting on the trapdoor. It was clear that very small movements of the trapdoor can cause significant changes in the pressure acting upon it. It was concluded that for an increase in $H_{OB}/D_{TD}$, there was an increase in the amount of load transferred from the trapdoor, i.e. the arching effect was more pronounced.

Ladanyi and Hoyaux (1969) published the results of experiments investigating the trapdoor problem using aluminium rods to simulate an ideal granular mass in plane strain. The granular mass was approximately 1000mm high and 2000mm wide. The aluminium rods had diameters of 3mm and 4.8mm, and were 64mm long. The trapdoor had a width of 76mm. Both the passive and active cases were investigated. The movements of the mass were recorded photographically, while $H_{OB}/B$ ratios of 2-5.33 were investigated. For the
active case, a sudden decrease of pressure was observed. A minimum pressure was reached when the trapdoor was lowered to 8-10% of the width of the trapdoor, before increasing gradually to a constant value. The reduction in pressure increased as the depth of overburden acting on the trapdoor increased. The authors derived a theoretical approach to estimate the pressure reduction on the trapdoor, and this compared well to the measured results.

Harris (1974) investigated the change in stress around a long wall coal mining operation by using multiple trapdoors in a box containing sand. The trapdoors were lowered independently to simulate the advancement of the face. The stress acting on the trapdoors and on the adjacent stationary base was measured using small pressure cells. The author identified that a “vault” was produced, which supported the sand above it and transferred the vertical load. The results of the experiments clearly illustrated the three-dimensional nature of the problem.

Vardoulakis et al. (1981) performed trapdoors tests in a test chamber 1000mm long, 150mm wide and 500mm high. The force on the trapdoor was not measured; instead, the failure and deformation mechanisms were monitored by using thin layers of coloured sand, and were recorded photographically. After a small amount of trapdoor displacement, a small dilatant zone was visible above the trapdoor. The boundaries between the dilatant zone and the stationary mass were clearly visible, starting vertically at the edges of the trapdoor and joining above the centreline. As the trapdoor was further displaced, the dilatant zone expanded upwards to the surface. At the ultimate state, a failure region was observed which was limited by two vertical shear bands starting from the edges of the trapdoor and extending to the surface, as shown in Figure 2.19.

Figure 2.19 Active mode after large trapdoor displacements, from Vardoulakis et al. (1981)
Evans (1983) carried out a series of trapdoor experiments at normal gravity. These included the active case above a circular trapdoor, the active case above a rectangular trapdoor under plane strain conditions and the active case above a series of independently lowered trapdoors. The pressure acting on the trapdoor was measured and the soil displacement pattern was monitored. A dilatant triangular-shaped vertically-expanding (i.e. increasing base angles) zone above the trapdoor was visible. The results from this series of tests were similar to those of Terzaghi (1936): the vertical stress acting on the trapdoor decreased quickly to a minimum with very small trapdoor displacements, and then increased to a constant with further displacement of the trapdoor (of the order of 10% of the trapdoor width). Evans found consistency between the results of the circular and the plane strain trapdoors; however, a trapdoor displacement of 3.5% of the trapdoor width was necessary to mobilise full arching for the axisymmetric case, whereas the figure for the plane strain case was 1.8%. From the experiments in which the series of trapdoors were lowered successively, Evans concluded that it was necessary to model the three-dimensional nature of the problem in order to accurately model the stress redistribution, and noted that maximum arching took place at trapdoor displacement of approximately 0.005B. A series of tests was carried out in order to determine the coefficient of lateral earth pressure. A pressure transducer was placed just above the trapdoor to measure the horizontal stress. The coefficient of earth pressure at-rest $K_0$ was found to be 0.51. After a trapdoor displacement of 0.01B, $K$ increased to a maximum value of approximately 1.2 as the vertical stress decreased rapidly and the horizontal stress remained almost constant. After further displacement, there was an increase in vertical stress and a decrease in horizontal stress, resulting in a lowering of the $K_{ep}$ value, until it reached a constant value. Evans therefore recommended a $K_{ep}$ value of 1.2 to be used for analyses.

Tanaka and Sakai (1993) carried out both active and passive trapdoor tests, under normal gravity plane strain conditions. Two different sizes of trapdoor were used. The load acting on the trapdoor was measured and the development of shear bands was observed using thin layers of coloured sand. The experimental results were similar to those of McNulty (1965). The experimental results were compared to the results of FE analyses, discussed further in §2.5.2.4.

Park and Adachi (2002) presented the results of trapdoors tests using aluminium rods to simulate a granular mass. A series of trapdoors were lowered in turn, and the pressure
acting on them and the corresponding surface displacements were measured. The influence of the trapdoor width, height of overburden and angle of the inclined layers ($\theta_i$) were investigated. The earth pressure distribution and average earth pressure acting on the trapdoor are shown in Figure 2.20 and Figure 2.21. A FE analysis was also carried out, which is discussed in § 2.5.2.4.

![Figure 2.20 Earth pressure distribution for $H_{OB}=200$mm and $\theta_i=90^\circ$, from Park and Adachi (2002)](image1)

![Figure 2.21 Earth pressure on trapdoor for $H_{OB}=200$mm and various $\theta_i$, from Park and Adachi (2002)](image2)

Adachi et al. (2003) also studied the distribution of earth pressure around active trapdoors under normal gravity. The test chamber was square in plan with sides of 1090mm, a height of 600mm and contained six trapdoors. The distribution of earth pressure acting on the trapdoors and on the adjacent stationary base of the chamber was measured using pressure cells. The trapdoors were lowered in turn to a maximum of 5mm in order to simulate the advancement of a tunnel face, while the surface settlement was measured. The arching effect was found to be more significant for larger overburden heights, and the three-dimensional nature of the problem was highlighted.

Shahin et al. (2004) investigated the influence of tunnel excavation on surface settlement and earth pressure by using a series of trapdoors which could be lowered independently. The arching effect was observed by placing load cells on one of the trapdoors. $H_{OB}/B$ ratios of 0.5-3 were investigated. Alumina balls were used to simulate a granular mass. The
three-dimensional nature of the arching effect was highlighted. A typical measured earth pressure distribution is shown in Figure 2.22.

The arching mechanism above circular active trapdoors of diameters 100mm, 200mm and 300mm was investigated by Sadrekarimi and Abbasnejad (2010). The load acting on the trapdoor was measured, and the surface settlement was monitored using a linear variable displacement transducer (LVDT). Loose and dense sand masses were tested. The results presented were similar to those of Terzaghi, i.e. the load acting on the trapdoor decreased to a minimum before increasing again until a constant value was reached. It was noted that the minimum and ultimate loads increased as the trapdoor diameter increased and/or the relative density of the sand decreased. The authors describe how the formation of a stable arch was dependent on the trapdoor diameter and the density of the sand. Below a critical density, a stable arch does not form. The load decrease to a minimum and subsequent increase to a constant was explained as follows: on trapdoor lowering, following a small elastic strain, the sand deforms plastically and pressure applied to the trapdoor decreases to a minimum. As trapdoor displacement continues, the pressure increases to a constant value, depending on the dilation angle and relative density of the sand. If a stable arch has been formed, the yielding sand mass then separates from the rest of the mass.

The experimental results were compared with the theoretical results given by the equation provided by Terzaghi (1936). However, according to Terzaghi’s theory, the pressure acting on the trapdoor increases with relative density. This is attributed to the fact that the stress values obtained from Terzaghi’s theory correspond to very low displacements. Finally, the shapes of the separated sand masses are compared to the theoretical shapes. It was
concluded that the theoretical shapes (using $2\psi$, where $\psi$ is the angle of dilatancy) are very conservative, and that the use of $2\varphi$ as the vertex angle is more justifiable.

### 2.5.2.2 Trapdoor tests carried out under increased gravity

The centrifuge trapdoor experiments of Iglesia *et al.* (1990) were mainly carried out using jointed media (rods of balsa wood and aluminium). However, initial tests using sand at increased gravity level were performed. The pressure acting on the trapdoor and the trapdoor displacement were measured. Various factors such as particle size, overburden height, trapdoor width and gravitational acceleration were investigated. The pressure-displacement plots obtained were of a similar shape to those of McNulty (1965), and it was also clear that, as the depth of overburden increased, the magnitude of the arching effect increased. An adaptation to the equation of Engesser (1882) was presented:

$$F_{\text{min}} = D^3\gamma \left( \frac{K' \cot \varphi H \frac{\pi}{4}}{8H + K'D \cot \varphi} + \frac{\pi}{32} \cot \varphi \right)$$

**Eqn. 2.41**

### 2.5.2.3 Trapdoor tests carried out under both normal and increased gravity

Costa *et al.* (2009) investigated the failure mechanisms induced by the active movement of a deep rectangular trapdoor (having a width and length of 85mm and 17.5mm, respectively) under normal and increased gravity conditions. The lack of information on failure mechanisms of deep trapdoors was highlighted. Sand samples were prepared by air pluviation. Most tests were carried out with the sand at a relative density of 85%, while a small number of tests used sand at a relative density of 42%. A soil cover ratio ($H_{OB}/B$) of 4.5 was used throughout. The trapdoor was not lowered gradually: instead, it plunged suddenly by a predetermined displacement. Displacement ratios ($\delta/B$) of 14%, 29% and 57% were used. Failure mechanisms were investigated by monitoring the movements of thin layers of coloured sand, and strain data were obtained by sand markers placed against a Perspex wall. The surface settlement directly above the trapdoor was measured using a LVDT. The difference in the failure mechanisms between deep and shallow active trapdoors was noted. In the case of shallow trapdoors, multiple failure surfaces develop above the trapdoor. This is shown schematically in Figure 2.23. The angle formed between the vertical and the tangent at any point along OA equals the soil dilatancy angle ($\psi$), the magnitude of which depends on the magnitude of trapdoor displacement. For deep trapdoors, a single internal failure surface develops which gradually approaches the vertical as the displacement of the trapdoor increases. However, a failure surface also developed adjacent to the trapdoor in the longitudinal direction, as shown in Figure 2.24.
These only occurred after significant trapdoor displacement ratios of between 14 and 29%, and formed a very small angle with the vertical. The surface settlement in loose samples was approximately three times greater than in dense samples.

![Figure 2.23 Propagation of failure surfaces in the transverse direction around a shallow \((H_{OB}/B<2)\) active trapdoor, from Costa et al. (2009)](image)

![Figure 2.24 Failure surfaces around a deep active trapdoor in the longitudinal direction, from Costa et al. (2009)](image)

Stone and Wood (1988) mainly performed trapdoor tests using a centrifuge, although some tests at normal gravity were also carried out. During the centrifuge tests, deformation patterns were monitored by film measurement data and photographs, while horizontal layers of coloured sand were also used. After a very small trapdoor displacement, an inclined shear band could be seen extending from the edges of the trapdoor. The corresponding maximum shear strain and vertical displacement contours formed an arch. After further trapdoor displacement, a secondary shear band could be seen developing from the trapdoor corner, which was steeper than the initial band. The arch appeared to be
triangular. At larger trapdoor displacements, the failure surfaces became almost vertical. Stone observed more shear bands developing in the tests carried out under normal gravity than in those carried out at increased gravity.

Dewoolkar et al. (2007) carried out active trapdoor tests under both normal and increased gravity. Two circular trapdoors with diameters of 38mm and 76mm were used. Loose and dense samples were tested with $H_{OB}/D_{TD}$ values of 0.67-6. The authors noted that the shear band formations in normal gravity tests were different from those in the centrifuge tests. These differences were explained by the stress level dependent nature of dilatancy. Maximum arching was seen at movements of approximately 1.5% of the trapdoor diameter, and the arching effect decreased after this. Increasing the relative density from 40% to 80% slightly decreased the minimum and ultimate force acting on the trapdoor, and also decreased the trapdoor movement required for maximum arching to develop.

2.5.2.4 Numerical investigations of trapdoor system

The results of the experimental work carried out by Tanaka and Sakai (1993) and described in § 2.5.2.1 were compared with the results of FE analyses. An elasto-plastic soil model was used, with both a “simple” and a “refined” strain softening model. The distribution of earth pressure from the FE analyses showed good agreement with the experimental results.

Using the $t_{ij}$ elastoplastic sand model, Nakai et al. (1997) performed a numerical investigation into the settlement and arching caused by an active trapdoor. The measured and computed settlements were compared, but as the earth pressures were not measured in the experimental work, the computed earth pressures are simply presented. The authors concluded that the arching effects in sand are more pronounced than those in clay, due to dilatancy.

Park and Adachi (2002) compared their experimental results (discussed in § 2.5.2.1) with those of an FE analysis. There was not good agreement between the numerical and experimental results for the load reduction on the trapdoor as trapdoor displacement occurred. There was relatively good agreement for the earth pressure distribution in the longitudinal direction. The experimental surface settlement profiles compared well with those obtained from the numerical model.
Shahin et al. (2004) presented the results of a numerical investigation using a three-dimensional FE elasto-plastic model, and compared these to their experimental results (discussed in § 2.5.2.1). Good agreement was found between the experimental and FE surface settlement results. The authors concluded that the model used can precisely simulate the arching effect due to an active trapdoor.

2.5.3 Experimental work – tunnelling

2.5.3.1 Introduction

The stability of the tunnel face and the movements associated with tunnelling have been widely researched through physical modelling. A number of different methods have been used, and are discussed by Meguid et al. (2008). Some of these are outlined in the following paragraphs.

A common method of investigating the construction of a tunnel is by placing a tube lined with a thin cylindrical rubber membrane of negligible strength and stiffness in a soil mass. The annulus between the lining and the rubber membrane is then filled with water or air. By controlling the volume of water/air in the membrane, precise control of the volume loss can be achieved. However, the three-dimensional nature of tunnelling is not taken into account, and the forces imposed on the soil mass due to the face rotation of a TBM in the prototype situation are not simulated. This method has been used extensively by many authors including Atkinson and Potts (1977), Hagiwara et al. (1999), and Farrell (2010).

The stability of the tunnel face may be investigated by closing the face of a buried tube with a covering which may be either rigid or flexible. It succeeds to some extent in modelling the three-dimensional nature of the problem. However, it does not impose the shear forces on the soil mass at the tunnel face that are encountered in the prototype situation. Chambon and Corte (1994), for example, carried out centrifuge tests with flexible tunnel faces supported by both air and water for various C/D ratios and varying gravity level in Fontainebleau sand. The zone of failure at the tunnel face was observed using intermittent layers of coloured sand, and by wetting the sand after testing and cutting the soil mass along vertical planes. For fully lined deeper tunnels and for C/D>1, it was evident that a stabilising arch had formed, with force transfer to the crown of the tunnel.
For shallow tunnels (C/D<1), the failure bulb was unable to close. This is shown in Figure 2.25.

![Figure 2.25 Failure bulbs for fully lined tunnel, from Chambon and Corte (1994)](image)

This method was also used by Oblozinsky and Kuwano (2005), who conducted two series of centrifuge tests in sand using both water and air to support the face. Only half the tunnel was modelled, so that the failure bulb in front of the face could be observed, as shown in Figure 2.26.

![Figure 2.26 Failure development for fully lined tunnel (left) and partly lined tunnel (right), from Oblozinsky and Kuwano (2005)](image)
Kamata and Mashimo (2003) used a rigid aluminium plate to investigate the effect of face bolting in dry Toyoura sand under increased gravity. Similar results to those discussed above were obtained.

Kim et al. (1998) and Chapman et al. (2007) used a soil auger to excavate a tunnel. However, as this method is primarily used to construct tunnels through clays, it is not discussed in more detail here. This method successfully models the three-dimensional problem, and in some way simulates the forces imposed on the soil at the face in the prototype situation.

A small number of investigations have been carried out to investigate specifically the arching effect around tunnels, rather than the stability of the face or ground movements alone. These are outlined in § 2.5.3.2 and § 2.5.3.3.

2.5.3.2 Tunnel tests carried out at normal gravity

Branque et al. (2006) designed a miniature EPBM (Figure 2.27) and performed a number of tests at reduced scale. The EPBM, with a cutter head diameter of 0.5m, constructed a tunnel through dry silica sand contained within a test chamber of height, width and length of 1.3m, 1.3m and 2m, respectively. This resulted in a relatively small distance to the vertical boundary from the tunnel centreline of 1.3D. A discussion of the effect of boundary conditions is given in § 3.4.1.2. The machine was instrumented to measure the pressure within the working chamber, the thrust provided by the hydraulic jacks and the cutter head torque.

Figure 2.27 Miniature EPBM of Branque et al. (2006)
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The surface settlements and settlements within the soil mass in front of the advancing face were measured. The soil mass within a limited area (primarily in front of the face) is instrumented with pressure cells in order to measure the total stress. No details of the method used to measure the settlements within the soil mass or of the pressure cells are provided. A very limited amount of earth pressure data is presented. For these reasons, there is significant difficulty in interpreting the validity of the results. The authors do however successfully model the three-dimensional nature of the tunnelling problem, as well as simulating the actual forces imposed on the soil mass during the tunnelling process.

2.5.3.3 Tunnel tests carried out at increased gravity

Nomoto et al. (1999) designed and manufactured a shield TBM for use in centrifuge tests in Toyoura sand. The shield of the TBM, which itself took four years to construct, was made up of three tubes. The innermost tube had a diameter of 100mm and contained a conveyor with a cutter at the face to excavate and remove spoil. The middle tube, with an external and internal diameter of 98mm and 96mm, respectively, acted as a tunnel lining and was instrumented with a series of load cells. The purpose of the outermost tube, which had a diameter of 100mm, was to simulate the tail void formation by removing it after completion of the shield advancement. This arrangement is shown schematically in Figure 2.28.

![Figure 2.28 Model TBM of Nomoto et al. (1999)](image)

The effect of C/D, tail void thickness and construction sequence on lining stress and ground surface settlement was measured. A limited amount of earth pressure data was collected using earth pressure cells (EPCs) at five positions below and adjacent to the tunnel. No technical specifications of the EPCs were provided. The horizontal stresses
measured at horizontal distances of 1D and 1.5D from the tunnel centreline (corresponding to gauges no. 2-2 and 2.1, respectively) are shown in Figure 2.29.

![Ground earth pressure measurement results](image)

**Figure 2.29 Ground earth pressure measurement results, from Nomoto et al. (1999)**

The authors discuss how the initial increase in stress as the cutter approaches is due to the soil arching ahead of the face. During the tail void formation process, the earth pressure drops suddenly before stabilising at a level below the initial level, and this is attributed by the authors to local plastic yield around the shield. The importance of the different construction sequences on the earth pressure is highlighted. No further details of the measured earth pressures are given.

Lee et al. (2006a) present the results of centrifuge tests carried out on tunnels in saturated sand. Sand was poured around the tunnel to a height corresponding to C/D=3. The excavation was modelled in the first series of tests by reducing the pressure of the supporting air and in the second, by decreasing the volume of the supporting water in the tunnel. A number of tests were carried out to investigate the arching effect by measuring the change in stress in the soil mass, using three EPCs per test. The majority of EPCs were used to measure the vertical stress. The EPC data from the tests were collated to provide details on the extent of the arching zone. No technical specifications of the cells are provided. The arching zone is identified, and the variation of vertical pressure with ground loss is presented. Nomoto et al. (1999) highlight the influence of the construction sequence on the earth pressures. Therefore, as the method of tunnel construction does not simulate the construction sequence used in the prototype situation, the validity of the results from the EPCs is called into question.
Xu et al. (2011) performed a series of tests using a model EPBM to construct tunnels in silty clay. Although not explicitly stated, it is believed these tests were carried out at increased gravity. The soil mass was contained within a test chamber of height, width and length of 2.4m, 1.2m and 2.4m, respectively. The diameter of the machine was 0.4m, resulting in a distance to the vertical boundary from the tunnel centreline of only 1.5D, which suggests that the proximity of the boundaries would significantly influence the results. The surface displacement is measured, and a number of EPCs positioned in the soil mass within limited proximity of the tunnel measure the horizontal earth pressure (Figure 2.30). No technical details of the EPCs were presented.

![Figure 2.30 EPC layout of Xu et al. (2011)](image)

For EPCs A2, A4 and A6, the maximum horizontal stress developed when the cutter head was 150mm away from the plane in which they were placed. The horizontal stress then decreased to approximately a constant, which was greater than the initial stress. These results are shown in Figure 2.31. They are presented in terms of the variation of earth pressure increment, which is the difference between the measured earth pressure and the initial horizontal stress.
The variation of earth pressure increment for cells A1, A2 and A3 is shown in Figure 2.32. It is evident that the cells closer to the tunnel, A2 and A3, exhibit greater arching than the most shallow EPC A1, and also reach their maximum earth pressure increment before the shallower EPC.

2.5.3.4 Numerical investigations of arching around tunnels
Lee et al. (2006b) performed a numerical investigation into the arching effect around tunnels in clay using the two-dimensional explicit finite difference program FLAC\textsuperscript{2D} (Cundall et al., 1993), and compared the results from this with the results of centrifuge
tests. The volume loss along the length of the tunnel was achieved by reducing the air pressure in a rubber membrane. As pore water pressure and not earth pressures were measured in the physical tests, these are not discussed further. The soil mass was treated as an isotropic and elastic perfectly plastic continuum using the Mohr-Coulomb failure criterion. The arching ratio (AR), expressed as a percentage, was introduced and defined as:

\[
AR = \frac{\Delta \sigma_v}{\sigma_{v0}} \times 100
\]

where \(\Delta \sigma_v\) is the change in vertical stress during tunnelling and \(\sigma_{v0}\) is the initial total overburden stress. If an element of soil transfers load to adjacent elements, it has a negative arching ratio. On the other hand, if it receives load from another element, it generates a positive arching ratio. The authors identified both the negative and positive arching zones for various C/D ratios, as well as the zone of soil behaving plastically. An example of their numerical results is given in Figure 2.33.

It was identified that shallower tunnelling imposes a larger arching effect on the soil mass than deeper tunnelling. An expression, derived from the results of the numerical investigation, was presented which shows the extent of the arching zones for both single and parallel tunnels.
2.6 Measurement of soil displacements

There are a number of different methods of measuring the displacement of a soil mass in a physical model. Some of these are discussed below.

The X-ray method was used by Roscoe et al. (1963) to measure the strains in a soil. Lead shots were placed in the soil mass during its preparation. X-rays of the soil mass were taken over time, and this allowed the movements of the shots to be tracked and provided information on the strain field. This method suffers from the disadvantage that it takes a significant amount of time to carry out each X-ray, as well as the health and safety concerns with their use.

Atkinson et al. (1977) used markers placed at the interface between the soil mass and a transparent wall. The positions of the markers are then tracked using a standard camera, removing the need for X-rays. However the markers may become obscured by the soil during the test. There are some difficulties regarding the number of lead shots and markers which should be used during an experiment. A dense grid may affect the behaviour of the soil, whereas a sparse grid may not provide sufficient data.

Wu and Lee (2003) used spaghetti, marked at 10mm intervals, to provide information on the movement of a clay due to a tunnelling sequence. As the spaghetti absorbed water, it moved with the soil. Once the test had been completed, the clay was carefully excavated to expose the spaghetti, and the graduations were traced onto a transparency. A comparison of the initial position of the graduations with the final position of the graduations provided settlement data and showed collapse mechanisms. This method was reported to produce satisfactory results, but there are clearly many inherent sources of inaccuracy.

Layers of coloured sand were used by Chambon and Corte (1994), so that the development of the failure mechanism around a tunnel was visible (Figure 2.34). While the soil mass was being prepared, layers of coloured sand were placed at regular intervals. After the test had been carried out, the mass was cut in vertical planes, allowing the way in which the soil failed to be observed. This method has been used in other studies, including Lee et al. (2004).
LVDTs are used to measure both surface and sub-surface vertical soil movements. They measure the movement of a metal plate, referred to as settlement plates. Surface settlements may also be measured using laser sensors (Farrell, 2010).

Particle Image Velocimetry (PIV) is a non-invasive technique which is used to obtain plane displacements. It may also be referred to as Digital Image Correlation (DIC). It was originally developed for use in experimental fluid mechanics (Adrian, 1991). In recent years, it has been increasingly used in other areas, such as aerodynamics and geotechnical engineering. It allows the investigation of soil movements at a grain-size level. This method is discussed in more detail in § 2.6.1.

2.6.1 Particle Image Velocimetry (PIV)

GeoPIV (White et al., 2003) is a MatLab module which uses PIV for the purposes of geotechnical engineering. It uses the principles of PIV by tracking the texture variation (i.e. the variation of brightness) of the soil in a particular plane, through the monitoring of digital images captured over time. This allows the movement undergone by the soil in that plane to be quantified. This process is summarised in Figure 2.35 below. Natural sand has a texture in the form of the subtle difference in colour between grains, and the light and shadow between adjacent grains. Tracking the movement of the soil in this way means that intrusive target markers are not necessary.
Figure 2.35 Conceptual diagram of GeoPIV process

An image is divided into a grid of square patches, with side length $L_{patch}$. In order to find the displacement of each patch between images, a search patch is created in the second image which is marginally larger than the initial patch. This patch defines the area in which the initial patch is searched for. The correlation between the patch in image one and the larger patch in image two is found using the Fast Fourier Transform. The location at which the highest correlation occurs indicates the new position of the patch. By fitting a bicubic interpolation around the highest integer peak, the displacement vector is established to sub-pixel resolution. This process is carried out for all patches within the image, and for all images within the test series. An integer termed "leapfrog" (LF) indicates how often the first image is updated. If LF=1, GeoPIV compares image 1 with image 2, followed by image 2 and 3, etc. By increasing LF, precision is increased. However, if an excessive value of LF is chosen, accuracy decreases. Precision is defined as the variation in multiple measurements of the same value, whilst accuracy is defined as the difference between a measured value and the actual value.

The image-space deformations (in pixels) are converted into object-space coordinates (in mm) using close-range photogrammetry. Many of the reasons why a single factor should not be used to do this have been discussed by White et al. (2003). These include non-coplanarity of the camera's charged couple device (CCD) and the soil plane, lens distortion (both radial and tangential), refraction through a viewing window (i.e. Perspex) and the fact that CCD pixels are not perfect squares.

The procedure used to transform the image-space coordinates to object-space coordinates is outlined below. The object-space coordinates of a number of target markers (a black circle with a diameter of 6mm on a square white background with side length of 10mm) are found using a certified photogrammetric reference field, known as a Mylar card (Figure
These target markers are stationary and are affixed to the sand-Perspex interface. The Mylar card has an array of dots on it, which provide a reference field of known object-space coordinates. After identifying their image-space location in the initial image, their movement is tracked throughout all subsequent images using PIV. From the relationship between the known object-space coordinates found using the Mylar card and the image-space coordinates, a number of parameters can be found. These include parameters which describe the position of the camera relative to the object, the focal length of the camera, and the refractive index of the Perspex viewing window. The transformation of the image-space movement of the soil to object-space movement can then be completed.

Marshall (2009) carried out an investigation into the relationship between the friction between Perspex and a silica sand, and tunnel induced ground movements. Displacements measured at the Perspex-sand interface were found to be approximately 10% lower than displacements measured within the sand mass, indicating that there were some frictional effects. This effect increased with depth as the horizontal stresses acting against the Perspex increased. However, this test was performed at increased gravity, meaning the horizontal stresses were much greater than in the case of normal gravity. The results of
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Marshall’s test would suggest that frictional effects are not significant for the tests carried out in the test chamber described in § 3.4.1.

2.6.1.1 Previous investigation of PIV precision

The performance of PIV can be evaluated by considering the errors associated with three terms: accuracy, precision and resolution. Accuracy and precision have previously been defined in § 2.6.1. Resolution is defined as the smallest interval that can be present in a reading.

A series of experiments were carried out by White et al. (2003) in order to assess the performance of GeoPIV. The authors concluded that the precision of the program was strongly dependent on the patch size chosen by the user, and gave the following equation (Eqn. 2.43) which allows an estimate of the random errors ($\rho_{\text{pixel}}$) present in the data to be made:

$$\rho_{\text{pixel}} = \frac{0.6}{L_{\text{patch}}} + \frac{150,000}{L_{\text{patch}}^8}$$

Eqn. 2.43

This error may be expressed as a percentage of the field-of-view (FOV) by dividing by the width of the image in pixels. According to the equation above, the error is reduced by increasing $L_{\text{patch}}$. However, smaller patches provide more measurement points, allowing greater detail in areas of high strain to be observed. Figure 2.37 shows the dependency of the measurement precision on the resolution of the camera’s CCD and the patch size, where the x axis refers to the number of measurement points within the image. It also shows the improved precision of the GeoPIV system when compared to other methods, including the use of target markers.
Figure 2.37 Precision of GeoPIV system, from White and Take (2002)
3. Experimental investigation

3.1 Introduction

The purpose of this chapter is to detail the techniques and equipment used to carry out the physical model tests, the results of which are presented and discussed in Chapters 4 and 5. Following a discussion of physical modelling and the applicable scaling laws, the test apparatus is described. Details of the soil material and the instrumentation used during the tests are subsequently discussed. Finally, the test programme and its specific arrangements are summarised.

3.2 Introduction to physical modelling

Physical model testing is a powerful experimental technique which has long been used for the purposes of geotechnical research, such as shallow foundations, retaining walls and slope stability, as reviewed by White (2008). Physical modelling of tunnels in particular allows the investigation of the influencing factors on tunnel and soil behaviour, and of failure mechanisms and deformation patterns (Meguid et al., 2008).

Field trials are expensive and time-consuming, and have many added complications, including those which arise due to the variation of ground conditions. Repeatability is very difficult. Full-scale field tests of tunnels may be limited by safety concerns, as it may not be possible to access tunnels that are near collapse. Therefore, tunnelling is more commonly investigated by reduced scale physical models (normal gravity and centrifuge) and numerical modelling.

Centrifuge modelling involves radially accelerating a specimen box containing a model, thereby subjecting it to high centrifugal forces. If a test is carried out at N times earth’s gravity, the model represents a prototype with dimensions N times greater than the model. For example, a model tunnel with a diameter (D) of 60mm tested at 100g represents a prototype tunnel of diameter 6m, although not all parameters are scaled in this manner (for example time properties related to consolidation), and a rigorous dimensional analysis
must be carried out. A particular advantage of centrifuge tests is that in-situ stresses are realistically simulated. However, tests of this kind have several disadvantages. They are very expensive and require long preparation times, due in part to the complex equipment (including the centrifuge itself) and the necessary instrumentation. In addition, the radial forces throughout the model are not the same. Taylor (1995) provides further details of the various limitations.

Normal gravity model tests are an inexpensive method of investigating relatively complex problems under controlled conditions. Although stress levels characteristic of real geotechnical problems cannot be recreated, these tests are easily reproducible, meaning that they are particularly suitable for parametric studies. They are also more economical than full-scale and centrifuge tests.

Two series of tests have been carried out. The first series investigated the arching effect around an active trapdoor. The second investigated the arching effect and corresponding displacements due to tunnel construction by a miniature tunnel boring machine (MTBM). Although technically feasible to control both an active trapdoor and a MTBM during a centrifuge flight, this would have led to considerable difficulties. Due to this, and coupled with the availability of equipment and lack of a centrifuge, normal gravity model tests were employed.

3.3 Scaling laws

3.3.1 Introduction

As the physical model is being performed at a reduced scale, it is important to be able to establish the validity of the model. Unless a scaling law is applied, a physical model is only valid when the dimensions in the model are similar to those in the prototype. In order to appropriately extrapolate from model scale to prototype scale, scaling laws are used. The understanding of these scaling laws and the dimensional analysis which controls them is therefore important. The Buckingham II theorem (Buckingham, 1914) is a key theorem in dimensional analysis, which states that a problem involving $n$ physical variables, themselves expressible in terms of $k$ fundamental physical quantities, can be expressed in terms of $j$ dimensionless variables, $\Pi_j$. These dimensionless variables are products of
different powers of the physical variables. If similarity has been achieved the
dimensionless variables of the model will be the same as those of the prototype, i.e.
\[ \Pi_i,_{\text{model}} = \Pi_i,_{\text{prototype}} \]  
Eqn. 3.1
If similarity has not been achieved, judgement is required to decide whether or not the
results have been influenced.

### 3.3.2 Scaling issues

For the purposes of reduced scale model tests, all the dimensions of the prototype are
usually scaled in the same proportion. However, soil is not generally scaled in this way,
and the same sand is usually used in the model as in the prototype, as described by Kirsch
(2010):
\[ \left( \frac{d_{50}}{D} \right)_{\text{model}} \neq \left( \frac{d_{50}}{D} \right)_{\text{prototype}} \]  
Eqn. 3.2
Although there are few recommendations available in the literature regarding the scaling
issues that may be encountered during tests carried out under normal gravity, a review of
those encountered during centrifuge modelling was presented in a report by ISSMGE
Technical Committee 2 (Physical modelling in Geotechnics) (2007). In order to minimise
grain size effects on the soil-structure interaction of a tunnel face stability problem, the
following recommendation was presented:
\[ \frac{D}{d_{50}} > 175 \]  
Eqn. 3.3
As the tunnel diameter and mean particle size used in the author’s tests were 88mm and
0.198mm, respectively, a ratio well in excess of that recommended is obtained (≈440). In
the ISSMGE report, however, it is also highlighted that there may be grain size effects on
the development of shear band patterns for a trapdoor even for \( D/d_{50} \) ratios as high as 1000.

The scaling effects due to soil non-linearity have been highlighted by Muir Wood (2004).
It was contended in this work that a small model may not reproduce the system response as
the behaviour of the interface will be controlled by relative displacement across it. This
was explained with reference to Figure 3.1. A small model only mobilises small relative
displacements along a shear plane due to the reduced scale, resulting in relatively high
shear stresses in comparison with the prototype situation.
This is relevant for the purposes of this work as the author’s series of tests investigate the arching effect and involve the relative movement of interfaces between separate blocks of soil. In order to deal with this, it was recommended by Muir Wood to normalise the displacements with the mean particle diameter.

The above points have been taken into account when interpreting the results of the tests.

3.3.3 Dimensionless variables

A dimensional analysis was carried out by identifying the governing quantities of both the trapdoor and MTBM tests. For the series of trapdoor tests, the parameters outlined below were identified as the principal ones governing both the vertical stress acting on the trapdoor ($\sigma_{v,TD}$) and the stress acting within the soil mass. They can be split into two principal groups:

- Geometry of the problem
  - diameter of the trapdoor ($D_{TD}$)
  - soil cover above trapdoor ($z_0$)
  - vertical displacement of trapdoor ($\delta$)
  - horizontal and vertical distance from initial trapdoor position ($x_{TD}$ and $z_{TD}$, respectively)

- Soil properties
  - dry density of the soil ($\rho_d$)
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- void ratio of the soil (e)
- critical angle of shearing resistance of the soil (φc)
- angle of dilation of soil (ψ)

These parameters were combined to form the following dimensionless variables:
- normalised stress acting on trapdoor (σv,TD/σv,TD,i) where σv,TD,i is the initial vertical stress prior to any trapdoor displacement
- normalised stress acting within the soil mass (σ/σi), where σi is the initial stress prior to any trapdoor displacement
- normalised soil cover to diameter ratio (zTD/DTD)
- normalised vertical displacement of trapdoor (δ/DTD)
- normalised distance from trapdoor (XTD/DTD and ZTD/DTD)
- void ratio (e)
- critical angle of shearing resistance (φc)
- angle of dilation (ψ)

For the series of MTBM tests, the parameters outlined below were identified as the principal ones governing the settlements and stresses acting within the soil mass. They are split into three principal groups:
- Geometry of the problem
  - diameter of the tunnel face and lining (Df and Di, respectively)
  - depth (z)
  - distances from MTBM face in the transverse and longitudinal directions, (x and y respectively)
  - vertical distance from MTBM crown (zcrown)
- Soil properties
  - dry density of the soil (ρd)
  - void ratio of the soil (e)
  - critical angle of shearing resistance of the soil (φc)
  - angle of dilation of soil (ψ)
- MTBM parameters
  - Excavation rate (Q)
  - Face pressure (Pf)

These parameters were combined to form the following dimensionless variables:
normalised stress acting within the soil mass (σ/σi), where σi is the initial stress prior to any excavation
- volume loss (Vf), as defined in Eqn. 2.4
- normalised distances from MTBM (x/Df, y/Df and z_crown/Df)
- void ratio (e)
- critical angle of shearing resistance (φc)
- angle of dilation (ψ)
- Over-extraction ratio, Q' (=Q/Q_theor), where Q_theor is the theoretical excavation rate
- Normalised face pressure (Pf/γDf), where γ is the unit weight of the soil

3.4 Apparatus design

This section discusses the equipment which was designed and manufactured for the purposes of the tests.

3.4.1 Test chamber and frame design

3.4.1.1 Design criteria

The advancement of a MTBM imposes a three-dimensional stress and strain field on the soil mass. The structure which holds the soil is referred to throughout this work as the test chamber. Several basic design criteria of the test chamber were identified. These included:
- Adequate structural capacity
- Provision of facilities to allow uniform and repeatable samples to be formed
- Provision for observing soil displacements
- Provision for trapdoor experiments

The test chamber was elevated above the laboratory floor by supporting it within a steel frame to allow access to the trapdoor assembly.

3.4.1.2 Selection of test chamber dimensions

An investigation into the effect of the boundary conditions was carried out in order that the results of the tests could be accurately interpreted and their validity assessed. Although it is unlikely that it is possible to remove the influence of the boundary conditions entirely, it is possible to reduce it to levels such that their impact on the results is insignificant.
Oteo and Sagaseta (1982) recommended a distance of 9D (where D is the tunnel diameter) to the boundary from the tunnel centreline. This value appears to refer to the vertical boundary to the side of the tunnel, rather than the base. However, other testing programmes have adopted much smaller values, over a relatively wide range. Values of 3D (Nomoto et al., 1996) to 5.6D (Taylor, 1984) have been adopted for the distance from the vertical boundary to the tunnel centreline. Distances to the base of the test chamber tend to be much smaller, with values of 1.5D (Nomoto et al., 1996) to 3.2D (Kim et al., 1998) used in the literature. A wide range of distances from the edge of a trapdoor to the nearest vertical boundary have been adopted in the literature. Tanaka and Sakai (1993), for example, adopted values of 9.5D_{TD} and 8.65D_{TD}, whereas Evans (1983) adopted a value of 1.96D_{TD}.

A series of 2D and 3D finite element (FE) analyses were carried out to investigate the appropriate dimensions of the test chamber and the trapdoor, using the commercially available software PLAXIS version 9.2 and PLAXIS 3D Tunnel version 2.4, respectively. As the tests carried out were primarily concerned with the re-distribution of stresses in the sand mass, the FE analyses concentrated on the effect of the dimensions on the stress field. The stress field after a specified tunnel contraction was therefore compared to the initial stress field prior to any contraction. The contraction method is the way in which the volume loss due to the construction of a tunnel is simulated in PLAXIS, and is expressed as the ratio of the reduction in the cross-sectional area of the tunnel to the original area. The effect of other aspects on the numerical results was also investigated, such as the diameter of the tunnel and characteristics of the sand.

Figure 3.2 shows the influence of the horizontal distance from the tunnel centreline to the vertical boundary (x) on the vertical stresses at 1D from the tunnel centreline. The "initial stress" refers to the stress field through the tunnel centreline prior to any soil movements occurring. The difference in the vertical stress distribution between x=4.5D and x=6.5D is larger than the difference between x=6.5D and x=8.5D, where little difference is evident, i.e. there is a reduction in the impact on the vertical stresses as the value of x increases.
Figure 3.2 Effect of distance to vertical boundary (x) on vertical stress field at 1D from tunnel centreline

Figure 3.3 shows the influence of the vertical distance from the tunnel centreline to the base of the test chamber (y) on the vertical stresses at 1D from the tunnel centreline. In the same way as for Figure 3.2, the “initial stress” refers to the stress field through the tunnel centreline prior to any soil movements occurring. The ideal position at which the tunnel should be placed is not clear from these results.

Figure 3.3 Effect of distance to base boundary (y) on vertical stress field at 1D from tunnel centreline

It was necessary to compromise between the difficulties in using vast volumes of sand (i.e. for a test chamber with very large dimensions) and the influence of the boundaries. Owing to the range in which the miniature earth pressure cells (EPCs) used to quantify the arching
operate (see § 3.6.1), it was necessary that the stresses acting around the tunnel are as large as possible. Therefore, a further compromise on the value of $y$ was necessary.

A test chamber having final internal dimensions of 1.250m x 0.80m x 1.50m (width x length x height), was selected based on the results of the FE analyses. The width of 1.250m corresponds to an $x$ value of 7.1D, whilst the tunnel is positioned so that the tunnel centreline is 396mm from the test chamber base (i.e. $y=4.5D$). The walls of the test chamber are constructed from 12mm thick Perspex in order to allow deformations to be observed, while the base was constructed from 10mm thick mild steel in order to satisfy structural requirements. A drawing showing the test chamber and its dimensions is given in Figure 3.4.

![Figure 3.4 Elevation (left) and plan view (right) of test chamber (dimensions in mm)](image)

Circular and square trapdoors were incorporated into the base of the test chamber, although the square trapdoor was not used in this study. The circular trapdoor has a diameter ($D_{TD}$) of 150mm. The minimum distance to a vertical boundary from the edge of the trapdoor is 325mm, which corresponds to a distance of $2.2D_{TD}$. A drawing showing the general layout of the trapdoors is given in Figure 3.5.
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The frame which elevates the test chamber above the laboratory floor in order to allow access to the trapdoor is constructed from 70mm x 70mm x 8mm angle mild steel. The sand pluviation device, which consists of a hopper (which holds the sand prior to pluviation) and a shutter plate, was incorporated within this frame. Further details of the sand raining device are given in § 3.5.3. The frame and test chamber assembly and its dimensions are shown in Figure 3.6.

3.4.2 Trapdoor assembly design

For the purposes of the active trapdoor tests, a system was devised whereby the vertical downward movement of the trapdoor could be controlled, monitored and quantified, and where the vertical load acting upon the trapdoor could be measured. The trapdoor displaces vertically within a cylinder which is rigidly attached to the base of the test chamber. The cylinder is thoroughly greased before each test to reduce friction between the trapdoor and the cylinder and to stop sand grains from falling in the small gap between the trapdoor and the cylinder. The trapdoor has a maximum displacement ($\delta_{\text{max}}$) of approximately 50mm, corresponding to a normalised displacement ($\delta/D_{\text{TD}}$) of 33.3%. The displacement of the trapdoor is controlled using a hydraulic jack. Very small displacements of the order of 0.10mm ($\delta/D_{\text{TD}}=0.067\%$) may be achieved, and measured to a resolution of 0.01mm. The trapdoor arrangement is shown schematically in Figure 3.7.

---
The displacement of the trapdoor was monitored using 3nr. linear variable differential transformers (LVDTs) manufactured by RDP Electronics (model type ACT2000) and a dial gauge. A LVDT was positioned at three of the four corners of the square base of the
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trapdoor. As the data logging equipment for these instruments was situated at a distance from the trapdoor, it was not possible to operate the hydraulic jack and see the output from the instrumentation simultaneously. Therefore, a dial gauge was positioned at the corner nearest the hydraulic jack operator, in order to allow a visual quantitative value of the displacement which had occurred. This arrangement of four displacement measuring instruments allowed any tilt in the trapdoor as displacement occurred to be detected. This was not an issue during the tests. A load cell was placed between the trapdoor and the hydraulic jack to measure the force required to restrain the trapdoor. This value was equal to the average vertical stress acting on the trapdoor. Further details of the LVDTs and load cell are provided in § 3.6.3.2 and § 3.6.4 respectively.

3.4.3 MTBM design

3.4.3.1 Introduction

Various techniques of modelling the construction of a tunnel have been discussed in § 2.5.3.1. Many of these do not accurately simulate the mode of construction that occurs or the forces imposed on the soil mass in the prototype situation. The aim of this study was to investigate the arching effect and settlements due to tunnel construction. Therefore, considerable effort was made to ensure that the tunnel construction procedure was as realistic as possible.

A model MTBM was constructed for the tests, as this method simulates the mode of tunnel construction and takes into account the principal sources of ground movement (see § 2.3.1) which occur in the prototype situation. The model MTBM achieves this in the following manner: a rotating cutterhead excavates the soil at the tunnel face, resulting in radial and longitudinal ground strains at and ahead of the face. This also imposes the shear forces on the soil mass that are experienced in the prototype at the tunnel face through the rotating of the cutterhead. Although over-cutting and the tail void have been simulated previously by Nomoto et al. (1996) by having a pull-out tube, it is simulated in this study by having a tapered lining section behind the cutting face. In this way, a known volume loss is achieved. It is considered by the author that soil movements due to tunnel lining deflection will be very limited, due to the high stiffness of the tunnel lining. This is discussed in further detail in § 3.4.3.2. Consolidation effects are a long-term time-dependent effect and are clearly not relevant in the case of dry sands.
3.4.3.2 Cutting face and lining

The tunnel is excavated using an MTBM which has been developed by the author. The rotary cutting face has a diameter of 88mm (Df), and the sand passes into the tunnel lining through three cutting blades. The diameter chosen allowed ease of fabrication of the cutters, whilst also satisfying requirements related to the effect of the test chamber dimensions influencing the results, as discussed in § 3.4.1.2. The cutting face imposes forces on the sand mass that are similar to those experienced in practice.

A movable face plate support, machined from PVC, was used to support the sand at the face during the period between MTBM drives. It was positioned against the back of the cutter face and prevented any loose sand from moving through the cutters into the lining. The spoil is removed, using a vacuum, through a 12mm diameter tube which is connected to the movable support. The support is brought in to and out of contact with the back of the cutting face by moving the spoil tube. The cutting face is immediately followed by a separate stainless steel lining, which tapers over a distance of 100mm from a diameter of 88mm to a final lining diameter (Di) of 84mm, thereby simulating the tail void. The taper corresponds to a reduction in area of 8.88%. Although this value is large, it was chosen so as to generate considerable movements in the soil which can be measured easily. Frictional forces between the rotating face and the lining are reduced by a thin layer of felt, which is attached to the 2mm thick lining wall (tL) using adhesive. This assembly and its dimensions are shown in Figure 3.8.

The stiffness of a tunnel lining relative to the soil in which it is constructed may be characterised by two dimensionless numbers, termed the flexibility ratio (FR) and the compressibility ratio (CR) (Peck et al., 1972). The flexibility ratio defines the relative bending flexibility of the lining, and the compressibility ratio defines its relative flexibility in hoop compression. These parameters were defined by Burns and Richards (1964) and Hoeg (1968), and are shown in Eqn. 3.4 and Eqn. 3.5:

\[
CR = \frac{E_s R}{t_L(1 + v_s)(1 - 2v_s) \left( \frac{E_L}{1 - v_L^2} \right)}
\]

Eqn. 3.4

\[
FR = \frac{E_s R^3(1 - v_s^2)}{6E_L l_L(1 + v_s)}
\]

Eqn. 3.5
where $E_S$ and $\nu_s$ are the Young’s Modulus and Poisson’s ratio of the soil respectively, $R$ is the tunnel radius, and $E_L$, $I_L$, and $\nu_L$ are the Young’s Modulus, second moment of area and Poisson’s ratio of the material from which the tunnel lining is constructed, respectively. The soil properties were obtained from laboratory tests, which are discussed in more detail in § 6.3.2. According to Kim et al. (1998), typical values of FR in practice lie in the range of 5 to 300, where a high value indicates a flexible lining. Using Eqn. 3.5, the lining used in these tests has a flexibility ratio of approximately 0.75, signifying a stiff lining. This suggests that ground movements due to lining deflection will not be significant.

![Figure 3.8 Cutting face (left) and cutting face and tapered lining (right) (dimensions in mm)](image)

The lining and cutting face rotation shaft are supported by three bearings positioned along the 790mm non-tapered length of the lining. These details are shown in Figure 3.9.

![Figure 3.9 Lining and bearings (dimensions in mm)](image)

### 3.4.3.3 Movement of cutting face and lining

A Lucas DC motor was used to rotate the cutting face. The armature and field currents were controlled by Skytronic (model number 650.682) and Isotech (model number IPS302A) adjustable DC power supplies, respectively. The speed of rotation of the cutting
face was measured using a mechanical counter equipped with a spring arm. The counter was positioned so that the trigger (a short metal bar affixed to the cutting face rotation shaft) struck the spring arm at one point during each revolution.

The cutting face rotation motor, a load cell (used to measure the pressure acting on the tunnel face) and a steel bar were positioned on a moving carriage. The purpose of the steel bar was to transfer the forward thrust of the moving carriage to the lining itself. Forward advancement of the moving carriage was achieved by using a thrust motor to rotate a 12mm diameter threaded bar, which the moving carriage sat upon. This motor was manufactured by Citenco FHP Motors, and provided a speed of up to 150rpm. It was powered by AC mains supply.

The maximum forward displacement achievable by the carriage was 905mm. A nylon wire was connected from the moving carriage to a rotary potentiometer manufactured by Bourns (model number 53RAA-R25A-13L). The wire was wrapped around the potentiometer and a counter-weight attached to the other end. In this way, the forward movement of the carriage and therefore the lining could be quantified after a simple calibration test. The data from the rotary potentiometer was corroborated by physical measurements taken after every drive using a measuring tape. The potentiometer was powered and the data acquired using the same system as the LVDTs used to measure surface settlement (see § 3.6.3.3). The assembly design is presented in Figure 3.10. Photographs of the arrangement without and with the motor and lining are shown in Figure 3.11 and Figure 3.12, respectively.

Prior to the first drive in each MTBM test, the cutting face and lining were positioned approximately 10mm into the test chamber. A rubber gasket, affixed to the inside face of the test chamber around the MTBM entry hole, ensured no sand could escape from the test chamber as the MTBM moved into it.
Figure 3.10 Schematic elevation (top) and plan (bottom) of assembly providing movement and support to cutting face and lining (dimensions in mm)

Figure 3.11 Photograph of assembly providing movement and support to cutting face and lining
3.4.3.4 Measurement of face pressure

A fitting was machined (shown in Figure 3.12 above) which connected directly to the load cell and rested against the back of the cutting face rotation motor. The motor was positioned on its base such that a very small amount of movement was possible. Therefore, when a force was imposed on the cutting face, a displacement occurred which was transferred through the cutting face shaft to the motor. This caused a subsequent increase in the stress acting on the load cell, and allowed the force acting on the cutting face to be quantified. The small amount of movement of the cutting face did not affect its functioning in any way. Further details of the load cell are given in § 3.6.4.

Recording of the face pressure commenced prior to sand placement in the test chamber. As the height of sand extended above the level of the face of the MTBM, the face pressure increased. This was then compared to the theoretical face pressure, which was obtained using the theoretical vertical stress and the coefficient of earth pressure at-rest ($K_0$). $K_0$ was calculated using Jaky’s Law (Eqn. 6.6) and $\phi_c=36^\circ$ (Table 6.4).

3.5 Soil

3.5.1 Properties of soil used

Commercially available air-dried Glenview sand was used for all tests. It is supplied by Hillstreet Quarries Ltd. It has $d_{10}$, $d_{30}$, $d_{50}$ and $d_{60}$ values of 0.08mm, 0.15mm, 0.198mm and 0.222mm, respectively. These values were found in accordance with BS 1377-2 (BSI,
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This results in a coefficient of uniformity ($C_U$) and a coefficient of curvature ($C_Z$) of 2.78 and 1.27, respectively. A particle size distribution curve for the sand is shown in Figure 3.13. The specific gravity was found to be 2.60 using BS 1377-2 (BSI, 1990a). The minimum void ratio ($e_{\text{min}}$) was found using BS 1377-4 (BSI, 1990b) as 0.520. Although the same standard was used to find the maximum void ratio ($e_{\text{max}}$), erroneous results were obtained. An $e_{\text{max}}$ value of 0.846 was found using ASTM D4254-00 (ASTM, 2006). Further information on the properties of the sand are given in § 6.3. The maximum moisture content value of the sand measured over the duration of the tests was 0.14%. A summary of the soil properties is given in Table 3.1.

![Figure 3.13 Particle size distribution curve for Glenview sand](image)

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$d_{10}$</td>
<td>0.08mm</td>
</tr>
<tr>
<td>$d_{30}$</td>
<td>0.15mm</td>
</tr>
<tr>
<td>$d_{50}$</td>
<td>0.198mm</td>
</tr>
<tr>
<td>$d_{60}$</td>
<td>0.222mm</td>
</tr>
<tr>
<td>Coefficient of uniformity, $C_U$</td>
<td>2.78</td>
</tr>
<tr>
<td>Coefficient of curvature, $C_Z$</td>
<td>1.27</td>
</tr>
<tr>
<td>Specific gravity, $G_s$</td>
<td>2.60</td>
</tr>
<tr>
<td>Minimum void ratio ($e_{\text{min}}$)</td>
<td>0.520</td>
</tr>
<tr>
<td>Maximum void ratio ($e_{\text{max}}$)</td>
<td>0.846</td>
</tr>
</tbody>
</table>

Table 3.1 Summary of soil properties
3.5.2 Sample preparation

As the behaviour of a granular material is dependent on its degree of compaction, sand samples must be prepared to uniform and repeatable values of relative density ($I_D$). Relative density is defined as:

$$I_D = \frac{e_{\text{max}} - e}{e_{\text{max}} - e_{\text{min}}}$$  

where $e_{\text{max}}$ and $e_{\text{min}}$ are as defined in § 3.5.1 and $e$ is the void ratio of the placed sand.

Several methods of sand sample preparation have been proposed in the literature. These have been divided into two groups by Butterfield and Andrawes (1970). The first group consists of methods whereby the density is adjusted after deposition, e.g. tamping and vibrating. The second group consists of methods whereby the density is adjusted during deposition, e.g. stationary pluviation (both wet and dry) and travelling pluviation. Okamoto and Fityus (2006) reported that one of the advantages of the pluviation method over the ASTM vibration method was that samples prepared by pluviation simulates a soil fabric most similar to the one found in natural deposits formed by sedimentation (Rad and Tumay, 1987). A dry stationary pluviation method was used for all tests in this study.

3.5.3 Stationary pluviation apparatus design

A typical pluviation rig consists of a hopper (which contains the sand prior to pluviation) and a shutter plate (which releases the sand from the hopper through holes). A diffuser, which is a sieve used to spread the relatively concentrated flow of sand falling from the shutter plate over a greater area, may be used.

There are several principal variables in the design of a pluviation system. These include the drop height from the shutter plate to the sand surface and the diameter and spacing of holes in the shutter plate. Okamoto and Fityus (2006) found that the density was insensitive to the drop height once it exceeded 500mm. It was concluded that this is probably due to the fact that this height is adequate to allow the sand particles to attain their terminal velocity. Similarly, Rad and Tumay (1987) concluded that the effect of drop height was negligible above a critical height, whilst Walker and Whitaker (1967) concluded that there was no need to maintain a constant drop height.
Prior to pouring, the sand is contained within a 300mm high hopper, in the base of which is a number of 20mm diameter holes at a triangular spacing of 100mm. The shutter plate has the same pattern of holes. The minimum drop height possible in these tests, i.e. the distance from the bottom of the shutter plate to the top of the test chamber, was 600mm. Once the required volume of sand had been placed in the hopper, the position of the shutter plate was altered until its holes were aligned with those in the hopper, thereby allowing the sand to pour into the test chamber.

Measurement of $I_D$ achieved by the method described above was carried out using containers of known volume and mass placed in the test chamber at various positions. Extra sand that fell into the container was slowly cut away in thin layers using a smooth straight edge. The mass of sand in the containers was then used to calculate the $I_D$ of the sample. An average value of 27% was achieved, corresponding to a loose sample.

### 3.6 Instrumentation

#### 3.6.1 Miniature earth pressure cells

#### 3.6.1.1 Properties

The earth pressures in the sand mass were quantified using miniature earth pressure cells (EPCs). It was important that the dimensions of these instruments were as small as possible so that their presence affected the behaviour of the sand minimally. Correspondingly, the EPCs, which were manufactured by Tokyo Sokki Kenkyujo Co. Ltd., have a total diameter ($d_{c,t}$) of 6.5mm and a thickness ($t_{c,t}$) of 1mm. These cells consist of a stiff outer ring enclosing a thin circular diaphragm, which has four strain gauges mounted on its underside in order to sense deflection. Two of the strain gauges act in tension and two act in compression, providing a fully active Wheatstone bridge. A schematic of the cell is shown in Figure 3.14 and their properties are presented in Table 3.2.
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Figure 3.14 Schematic of miniature EPCs

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Capacity [kPa]</td>
<td>200</td>
</tr>
<tr>
<td>Cell diameter, $d_{c,d}$ [mm]</td>
<td>6.5</td>
</tr>
<tr>
<td>Cell thickness, $t_{c,t}$ [mm]</td>
<td>1</td>
</tr>
<tr>
<td>Cell diaphragm diameter, $d_{c,d}$ [mm]</td>
<td>5.80</td>
</tr>
<tr>
<td>Cell diaphragm thickness, $t_{c,d}$ [mm]</td>
<td>0.15</td>
</tr>
<tr>
<td>Cell diaphragm stiffness, $E_{c,d}$ [kPa]</td>
<td>$1.23 \times 10^8$</td>
</tr>
<tr>
<td>Cell diaphragm Poisson’s ratio, $v_{c,d}$ [kPa]</td>
<td>0.30</td>
</tr>
</tbody>
</table>

Table 3.2 Properties of miniature EPCs (* from Zhu et al. (2009))

Weiler and Kulhawy (1982) and Dunnicliff (1988) made several recommendations with regard to the specification of appropriate EPCs. Amongst these were:

- Diaphragm diameter to mean soil grain size ($d_{c,d}/d_{50}$)$>10$
- Cell aspect ratio, $d_{c,d}/t_{c,t}>5$
- Diaphragm diameter to deflection ratio ($d_{c,d}/\Delta_{c,d}$)$>2000 \rightarrow 5000$, where $\Delta_{c,d}$ is the average deflection of the diaphragm after Timoshenko and Woinowsky-Krieger (1959), as shown in Eqn. 3.7:

$$\Delta_{c,d} = \frac{q_c d_{c,d}^4 (1 - v_{c,d}^2)}{160 E_{c,d} t_{c,d}^3}$$

Eqn. 3.7

where $q_c$ is the uniformly distributed load acting on the cell.

- Sensing area ratio, $d_{c,d}^2/d_{c,t}^2<0.25 \rightarrow 0.45$

The purpose of the recommendation regarding the diaphragm diameter to mean soil grain size is to minimise the impact of point loads on the EPCs. This will occur if the sand grains
are too large, i.e. a small number of large particles loading the cell. This criterion was also relevant when choosing the soil type to use for the tests, as discussed in § 3.5.1.

Both the diaphragm diameter to mean soil grain size and the cell aspect ratio for the miniature EPCs used in this study satisfy the recommendations made above (29.29 and 6.5 respectively). The EPCs also satisfy the recommendation regarding the diaphragm diameter to deflection ratio at the maximum expected stress in the test chamber of approximately 20kPa, with a value of approximately 18,700. The cells do not satisfy the recommendation concerning the sensing area ratio, with a value of 0.80. This suggests that the cells may over-register, as stress concentrations at the cell edge may increase the stress over the active cell face. However, this effect appeared to have no influence on the performance of the cells during the calibration tests, which are discussed in § 3.6.1.3 and § 3.6.1.4.

3.6.1.2 Placement effects
Due to their dimensions, the behaviour of the miniature EPCs is highly dependent on placement effects, as described by Dunnicliff (1988), particularly when placed in the sand mass rather than on the base of the test chamber. The cells could not simply be placed on the surface of the sand, as their light weight meant that the input/output wires caused them to move from their placed position. Due to this, and as the most reproducible placement techniques are generally the most simple (Garnier et al., 1999), the EPCs in the tests were placed by gently pushing them approximately 20mm into the sand.

3.6.1.3 Introduction to calibration tests
The EPC calibration tests were carried out in a Rowe cell, with an internal diameter of 254mm and a height of 127mm. Loading was applied by means of a Bellofram seal. The sand was placed in a loose condition, similar to the density in the trapdoor and MTBM tests. It should be noted that the load was applied by changing the water pressure in the Bellofram seal and observing the magnitude of the pressure on a dial gauge. This inevitably leads to some inaccuracy, particularly at the relatively low stress levels investigated.

Two principal types of calibration tests were adopted: those in which the EPCs were placed on the base of the Rowe cell, and those in which they were placed within the sand mass.
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The tests and their details are summarised in Table 3.3. For tests in which the maximum load imposed on the EPCs was approximately 50kPa, the load sequence was repeated three times. For tests in which the maximum load imposed on the EPCs was approximately 20kPa, the load sequence was repeated four times.

<table>
<thead>
<tr>
<th>Test name</th>
<th>Miniature EPC position</th>
<th>Adhesive used</th>
<th>Load sequence [kPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>EPC-CAL_001</td>
<td>Base of Rowe cell</td>
<td>No</td>
<td>0 – 20 – 30 – 40 – 50 – 40 – 30 – 20 – 0</td>
</tr>
<tr>
<td>EPC-CAL_002</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>EPC-CAL_003</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>EPC-CAL_004</td>
<td>Base of Rowe cell</td>
<td>Yes</td>
<td>0 – 20 – 30 – 40 – 50 – 40 – 30 – 20 – 0</td>
</tr>
<tr>
<td>EPC-CAL_005</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>EPC-CAL_006</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>EPC-CAL_007</td>
<td>Base of Rowe cell</td>
<td>Yes</td>
<td>0 – 20 – 0</td>
</tr>
<tr>
<td>EPC-CAL_008</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>EPC-CAL_009</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>EPC-CAL_010</td>
<td>Within sand mass</td>
<td>N/A</td>
<td>0 – 20 – 30 – 40 – 50 – 40 – 30 – 20 – 0</td>
</tr>
<tr>
<td>EPC-CAL_011</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>EPC-CAL_012</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>EPC-CAL_013</td>
<td>Within sand mass</td>
<td>N/A</td>
<td>0 – 20 – 0</td>
</tr>
<tr>
<td>EPC-CAL_014</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>EPC-CAL_015</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3.3 Summary of miniature EPC calibration tests

In tests EPC-CAL_001 to EPC-CAL_003, no adhesive was used to affix the EPCs directly to the base of the Rowe cell. Rather, adhesive tape was used to affix the input-output wires to the base in close proximity to the connection of the wires with the EPC. For tests EPC-CAL_004 to EPC-CAL_009, the EPCs themselves were affixed to the base using adhesive provided by the manufacturers.

3.6.1.4 Results of calibration tests

EPC-CAL-001 to EPC-CAL-003:
Tests EPC-CAL_001 to EPC-CAL_003 investigated the behaviour of the EPCs in a stress range up to approximately 50kPa without the use of adhesive between the cell and the base
of the Rowe cell. Although within each test some EPCs performed very well (for example, cell 9332 shown in Figure 3.15), there were also some cells which performed poorly.

In order to investigate a loading pattern which is somewhat similar to the one experienced in the trapdoor and MTBM tests (albeit at higher stress levels) and to investigate repeatability between tests, a comparison was made between the first load cycles for each of the three tests. These results are shown in Figure 3.16. Although the magnitudes of the output voltages are different, the behaviour of the EPCs is similar.

Initially, the differences in the calibration results were assumed to be due to poor functionality of the EPCs themselves, for example, electrical wiring issues. During the trapdoor tests, however, it was clear that sand particles had become trapped between the EPC and the base of the test chamber, resulting in poor results. This led to tests EPC-CAL_004 to EPC-CAL_009 being performed after the trapdoor tests had been completed.
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Figure 3.16 First load cycle calibration curves for EPC 6 from tests EPC-CAL_001 to EPC-CAL_003

EPC-CAL_004 to EPC-CAL_009:

Due to the aforementioned reasons, further calibration tests were carried out using adhesive provided by the manufacturers to affix the EPCs themselves to the base of the test chamber. Tests EPC-CAL_004 to EPC-CAL_006 investigated the behaviour of the EPCs to a maximum stress of approximately 50kPa. Tests EPC-CAL_007 to EPC-CAL_009 were used to investigate their behaviour in a stress range which is similar to that experienced in the test chamber (up to approximately 20kPa).

All cells in tests in which the adhesive was used provided acceptable results. A sample of results obtained in tests EPC-CAL_004 to EPC-CAL_006 is given in Figure 3.17 and Figure 3.18. Figure 3.17 shows a typical EPC response for the three load cycles imposed in test EPC-CAL_004, whilst Figure 3.18 compares the first load cycle from each of the three tests. Although the results are qualitatively very similar to the results of the tests where adhesive was not used between the EPC and the base (tests EPC-CAL_001 to EPC-CAL_003), the use of adhesive allows the EPCs to perform consistently. The results of these tests show the importance of using adhesive directly between the base and the EPCs in order to obtain repeatable output voltages.
The results from the tests where the cells were imposed with a loading pattern of up to approximately 20kPa (EPC-CAL_007 to EPC-CAL_009) exhibited a high level of consistency. A sample of the results obtained from these tests is given below. Figure 3.19 shows a typical EPC response for the three load cycles imposed. Although the first load/unload stage shows hysteresis, the following stages do not. Figure 3.20 compares the first load cycle from each of the three tests. Very good repeatability is achieved.
Figure 3.19 Calibration curve for EPC 46 from test EPC-CAL_007

Figure 3.20 First load cycle calibration curves for EPC 46 from tests EPC-CAL_007 to EPC-CAL_009

EPC-CAL-010 to EPC-CAL-012:
The purpose of tests EPC-CAL_010 to EPC-CAL_012 was to calibrate the miniature EPCs when placed within the sand mass for a number of load steps up to approximately 50kPa. During the preparation of test EPC-CAL_010, the Rowe cell was filled to such an extent that, on placing the Bellofram seal on top of the cell, a significant load was imposed on the sand mass. This load cannot be quantified; therefore the results of this test have been omitted.
Figure 3.21 shows typical results from test EPC-CAL_012. A comparison of the first load stages from tests EPC-CAL_011 and EPC-CAL_012 is shown in Figure 3.22. These results (from cell 5) are typical of the results obtained from the tests. Although the output voltages from tests where the EPCs are placed in the soil mass vary quantitatively (probably due to placement effects), the results are qualitatively very similar.

![Figure 3.21 Calibration curve for EPC 5 from test EPC-CAL_012](image)

![Figure 3.22 First load cycle calibration curves for EPC 5 from tests EPC-CAL_011 and EPC-CAL_012](image)

**EPC-CAL-013 to EPC-CAL-015:**
Tests EPC-CAL_013 to EPC-CAL_015 were used to calibrate the miniature EPCs within the sand mass in a stress range which is similar to that in the tests (up to approximately...
20kPa). A typical calibration curve obtained from test EPC-CAL_015 is shown in Figure 3.23. Although hysteresis is observed in the first load cycle, none is evident in the following load cycles. There is good agreement between the cycles. Figure 3.24 shows a comparison between the first load cycles for each of the three tests. Although the results vary quantitatively, the results are qualitatively very similar. This variation is likely due to placement effects.

![Figure 3.23 Calibration curve for EPC 7 from test EPC-CAL_015](image)

The EPC hysteresis factor ($F_{hys}$) is defined as follows:

$$F_{hys} = \frac{V_{max}}{V_{min}}$$

Eqn. 3.8

![Figure 3.24 First load cycle calibration curves for EPC 7 from tests EPC-CAL_013 to EPC-CAL_015](image)
where $V_{\text{max}}$ and $V_{\text{min}}$ are the maximum and minimum voltage outputs recorded in tests EPC-CAL_013 to EPC-CAL_015 at an applied pressure of 20kPa and after returning to the initial pressure, respectively. For example, in test EPC-CAL_013, EPC 7 had $V_{\text{max}}=0.000069$ and $V_{\text{min}}=0.000014$, resulting in $F_{\text{hys}}=20.4\%$. The standard deviation of $F_{\text{hys}}$ values for EPC 7 shown in Figure 3.24 was 0.4%. The average standard deviation of $F_{\text{hys}}$ for all EPCs in the tests was 4.5%.

3.6.1.5 Conversion of EPC output from voltage to stress

A method was devised from the results of the calibration tests through which the output voltages from the EPCs could be converted to stress values. This was necessary so that the arching effect during the trapdoor and MTBM tests could be quantified accurately. Tests EPC-CAL_007 to EPC-CAL_009 and EPC-CAL_013 to EPC-CAL_015 were deemed to be most suitable to use for this purpose as the cells in these tests were subject to stresses in a range similar to those in the trapdoor and MTBM tests.

The following method was used to convert the output voltages from the EPCs during the trapdoor and MTBM tests to stress values:

- Calculate theoretical vertical stress ($\sigma_{\text{v, theor}}$) at level of EPC after sand filling using density of sand and overburden height above EPC. This corresponds to voltage output from EPC after filling is finished ($V_{\text{fill}}$) to give Point A ($\sigma_{\text{v, theor}}, V_{\text{fill}}$) in Figure 3.25 and Figure 3.26 as shown.
- Calculate EPC calibration factor ($C_f$) by dividing $\sigma_{\text{v, theor}}$ by $V_{\text{fill}}$
- If voltage output from EPC increases after filling (Figure 3.25):
  - Use $C_f$ to calculate increased stress level ($\sigma_{\text{max}}$) to give Point B ($\sigma_{\text{max}}, V_{\text{max}}$), where $V_{\text{max}}$ is the maximum output from the EPC
  - If voltage output then decreases from $V_{\text{max}}$, calculate EPC voltage at 0kPa ($V_0$) using the hysteresis factor ($F_{\text{hys}}$), and assume a linear relationship to Point C (0,$V_0$)
  - If voltage output then increases, assume a linear relationship back to Point B ($\sigma_{\text{max}}, V_{\text{max}}$)
- If voltage output from EPC decreases after filling (Figure 3.26):
  o Calculate EPC voltage at 0kPa ($V_0$) using $F_{\text{hys}}$, and assume a linear relationship to Point B ($\sigma_0, V_0$)
  o If voltage output then increases, assume a linear relationship back to Point A ($\sigma_v, V_{\text{fill}}$)

The EPC calibration factor ($C_f$) obtained from the calibration tests was generally higher than those obtained from the trapdoor and MTBM tests. This may be due to minor differences in conditions, for example, relative density of the sand. However it is not
considered to be significant, as the method of conversion from output voltages to stress value described above is based on qualitative rather than quantitative trends.

3.6.1.6 Data acquisition
The National Instruments SCXI-1322 terminal block mounted within the SCXI-1000 chassis was used to acquire the data from the miniature EPCs. A sampling rate of 1/sec was used. This chassis also provided an excitation voltage of 2.333V to each EPC.

3.6.2 Particle Image Velocimetry (PIV)

3.6.2.1 Current investigation of PIV precision
The precision of PIV was investigated by utilising a similar experimental set-up to that used by White et al. (2003). A sample of Glenview sand was placed in a carriage and displaced linearly in prescribed displacements along a track. The displacements of 0.25mm were measured using a dial gauge, with a precision of 0.01mm. The images taken during this test and all subsequent tests were captured by a Canon Powershot SX 110 IS digital camera, which has an image resolution of 3456 x 2592 (i.e. 8.96 mega pixels$^2$). It is a standard "off-the-shelf" camera, whose shutter may be operated remotely from a computer using a USB cable. This feature is necessary as manually pressing the shutter button may change the position of the camera. Images of the sand sample were taken before any movement had taken place, and after each displacement. Lighting was provided by a 500W tungsten halogen lamp adjacent to the sand sample. The FOV was approximately 675mm x 505mm, corresponding to an image scale of approximately 0.195mm/pixel. These dimensions were chosen as they are similar in magnitude to those required for the MTBM tests. The effect of patch size was investigated by carrying out the analysis for four different patch sizes: 10, 16, 32 and 50 pixels. For each patch size, LF values of 1 and 5 were used.

Figure 3.27 shows the resulting standard deviation ($\rho_{PIV}$) in mm resulting from the different patch size and leapfrog values. The error suggested by Eqn. 2.43 is also shown for comparison purposes. The results show that a LF value of 5 results in lower standard deviations when compared to a LF value of 1. It is also clear that error reduces to a constant with increasing patch size. The errors obtained from this analysis are greater than those suggested by Eqn. 2.43. For the patch sizes investigated and LF=5, the ratio between
the errors from the equation and those obtained from the analysis range from 4 to 13. A similar series of tests carried out found a ratio range of 2 to 15 (Boylan and Long, 2009).

![Figure 3.27 Effect of patch size and leapfrog on PIV precision](image)

It should be noted that while it may be advantageous to use large patch sizes in order to increase precision, this reduces the number of measurement points, thereby reducing the level of detail that can be seen in areas of high strain gradient (White et al., 2003). Accordingly, it is important to find a balance between the precision required and the number of measurement points.

### 3.6.2.2 Data acquisition

The camera was connected to a computer, and RemoteCaptureDC (part of Canon Utilities CameraWindow) was used to remotely capture the images. The images were saved on both the camera and the computer. A sample rate of 12/min was used, which is the maximum rate of the software.

### 3.6.3 Potentiometers

A number of slide potentiometers were used to measure the soil displacements occurring within the sand mass during the MTBM tests. These instruments, with a maximum travel length of 30mm, were manufactured by Alps Electrics (part number RS3011114A02) and are shown in Figure 3.28.
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Figure 3.28 Slide potentiometer used in MTBM tests to measure soil displacement

The potentiometers were positioned in the required array by attaching them using adhesive to 5mm x 20mm rectangular section steel members. These members were supported in position by placing their ends within holes at the top of the test chamber. The travel arms of the potentiometers were attached to 1mm thick 15mm diameter stainless steel settlement plates by 1mm diameter steel rods. The lengths of these rods varied depending on the position within the sand mass at which the settlement was to be measured. The diameter of the settlement plates was chosen based on the results of a series of calibration tests which were carried out. If a smaller diameter plate is used, the force acting on the travel arm is not great enough to cause its movement. A photograph of the potentiometer arrangement (and the LVDT arrangement) for the MTBM tests is shown in Figure 3.29.

Figure 3.29 LVDT and potentiometer arrangements for MTBM tests

3.6.3.1 Calibration of potentiometers

The potentiometers were calibrated by clamping the instrument above a column of sand with a diameter and height of 0.152m and 1m, respectively, contained within a 1.1m length of high-density polyethylene pipe. The sand, placed at a density similar to that of the
trapdoor and MTBM tests, was positioned on a steel plate supported by a hydraulic jack. A 0.8m long connector rod was used to connect the potentiometer to a settlement plate. The hydraulic jack was then used to displace the column of sand vertically. The magnitude of this displacement was measured by a dial gauge. Two series of tests were carried out. The first displaced the sand column vertically in intervals of approximately 1mm, whilst the second used intervals of approximately 0.1mm. These tests are summarised in Table 3.4.

<table>
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<tr>
<th>Test name</th>
<th>Maximum vertical displacement [mm]</th>
<th>Vertical displacement interval [mm]</th>
</tr>
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<td>POT-CAL-001</td>
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<td>1</td>
</tr>
<tr>
<td>POT-CAL-002</td>
<td>9</td>
<td>1</td>
</tr>
<tr>
<td>POT-CAL-003</td>
<td>9</td>
<td>1</td>
</tr>
<tr>
<td>POT-CAL-004</td>
<td>0.9</td>
<td>0.1</td>
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<td>POT-CAL-005</td>
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<td>0.1</td>
</tr>
<tr>
<td>POT-CAL-006</td>
<td>0.9</td>
<td>0.1</td>
</tr>
</tbody>
</table>

Table 3.4 Potentiometer calibration tests

The results of these tests are shown in Figure 3.30 and Figure 3.31. It can be seen from Figure 3.30 that there is a linear relationship between the voltage output from the potentiometer and the displacement. There is also very good repeatability between tests. Figure 3.31 identifies a limitation of the potentiometers – the first 0.2mm of movement went undetected. This is probably due to the development of the appropriate tensile stress in the rod that connects the settlement plate to the potentiometer. Ideally, the connector rod should be positioned so that any movement of the sand is transferred immediately to the arm of the potentiometer to which it is attached. If the rod is not at the appropriate stress, the movement of the settlement plate is not transferred to the potentiometer, but instead reduces the “slack” in the rod until the tension is reached which results in movement of the potentiometer arm. Although considerable care was taken during placing of the settlement plates and attaching of the connector rods to the potentiometers, the aspect described above should be kept in mind when interpreting the settlement values obtained in this manner.
3.6.3.2 Use of linear variable differential transducers (LVDTs)

3nr. LVDTs were used to measure the vertical surface settlement during the MTBM tests. Two of these instruments were the same as those used in the trapdoor tests (see § 3.4.2). The other instrument was the DC25 model manufactured by Solartron Metrology. Calibration of all LVDTs prior to testing showed a linear response with displacement.
For the purposes of the MTBM tests, a 10mm square metal plate was attached to the end of the armature of each LVDT. The LVDTs were clamped to vertical steel bars which were connected to the rectangular steel section members (discussed in § 3.6.3), and then positioned to measure the vertical settlement at the required point. A photograph of the LVDT arrangement (and the potentiometer arrangement) for the MTBM tests is shown in Figure 3.29.

3.6.3.3 Data acquisition
An excitation voltage of 2.4V was provided to the potentiometers by a Luscar PSU 130 power supply. A Picolog 1216, manufactured by Pico Technology, logged the data at a sampling rate of 1/sec. The LVDTs were powered and their data were recorded by the Vishay Strainsmart 6000 system at a sampling rate of 1/sec.

3.6.4 Load cells
In order to measure the force acting on the trapdoor during the tests, a load cell was placed between the trapdoor and the hydraulic jack (§ 3.4.2). The load cell used was manufactured by RDP Electronics Ltd. (Model 41), and has a capacity of 8.9kN with a resolution of 1N. A model 615 load cell manufactured by Tdea Huntleigh was used to measure the pressure acting on the face of the MTBM. It has a capacity of 2kN with a resolution of 0.1N. Calibration of these load cells prior to testing showed a linear response.

3.6.4.1 Data acquisition
The data from the RDP Electronics load cell was recorded manually. The data from the Tedea Huntleigh load cell was recorded using an Xplorer GLX logger manufactured by Pasco. During the sand pluviation into the test chamber, the sample rate selected was 4/min. During the MTBM drives, the sample rate was 2/sec.

3.7 Test procedure

3.7.1 Trapdoor tests
The procedure used during the trapdoor tests was as follows:
- EPCs were placed on the base of the test chamber. Voltage data from all cells were acquired prior to any sand pouring taking place and continuously until after the final trapdoor displacement had taken place.
- The sand was pluviated in layers, and interrupted in order to place the EPCs at locations throughout the sand mass. Pluviation finished when the test chamber contained sand to a height of 900mm. This corresponds to a height of $6D_{TD}$, resulting in the trapdoor being classified as "deep".
- Trapdoor displacements took place.

Each test took approximately a day to prepare and carry out. Emptying of the test chamber and removal of instrumentation also took approximately a day.

### 3.7.2 MTBM tests

The procedure used during the MTBM tests was as follows:

- The cutting face and lining were positioned approximately 10mm within the test chamber, and the PVC support placed against the back of the cutting face. The load cell measuring the face pressure began acquiring data at a sample rate of 4/minute. Voltage data from all miniature EPCs were acquired prior to sand pouring and continuously until after the final MTBM drive.

- The sand was poured in layers. This was interrupted in order to place the settlement plates and EPCs throughout the sand mass. The connecting rods for the potentiometers were affixed temporarily to wooden dowels which sat in place of the potentiometer and LVDT support system. This was necessary because the potentiometers were rigidly attached to the steel support system, and would have become unusable if left in the dusty environment of the test chamber during pouring.

- Pouring was continued until a height of 1250mm had been reached. The wooden dowels were then removed, the steel support system put in place, and the connecting rods attached to the potentiometers. Data acquisition from the potentiometers then began.

- The camera, tripod and two nr. 500W tungsten halogen lamps were put in place at the opposite side of the test chamber from which the MTBM was progressing (i.e. the MTBM was moving towards the plane at which the images for PIV were being captured). The LVDTs were put in place and began recording, the spoil removal pipe was attached to the vacuum, the sample rate of the load cell measuring the face pressure was increased to 2/sec, and the rotary potentiometer measuring the forward movement of the carriage began logging.
A drive sequence consisted of the following steps. The remote capture of images for PIV was started. The MTBM drive of approximately 100mm was completed, followed by the remote capture of images being stopped. The mass of sand excavated was then measured. The camera was used in this manner in order to reduce the amount of images required to be analysed after testing. The time taken solely to track the patches throughout the series of images for a typical test was approximately 48 hours. This would have been increased significantly had the steps described above not been taken.

Eight MTBM drives were carried out until the cutting face had progressed to the plane at which the PIV images were being captured, i.e. the rear face of the test chamber.

Each test took approximately a day to prepare and carry out. Emptying of the test chamber and removal of instrumentation also took approximately a day.

### 3.8 Summary of tests

#### 3.8.1 Trapdoor tests

A series of trapdoor tests were carried out. The results of six of these are presented in Chapter 4, namely CTD002 to CTD007. Tests CTD002 to CTD004 primarily involved the instrumentation of the sand mass directly above the trapdoor. Initially, the author intended to place the miniature EPCs vertically in order to measure the horizontal stress. However, it proved difficult to position them in this orientation reliably, and the cells were therefore placed horizontally (i.e. measuring the vertical stress). The co-ordinate system shown in Figure 3.32 is used to describe the positions of the instrumentation used in the MTBM tests. The positions of the EPCs for these tests are shown in Figure 3.33, in which the distances from the trapdoor centreline and from the base of the test chamber have been normalised by the diameter of the trapdoor.
In tests CTD005 to CTD007, the sand mass adjacent to the trapdoor was primarily instrumented with miniature EPCs. The cells were placed horizontally in order to measure the vertical stress. The positions of the EPCs for these tests are shown in Figure 3.34. Similarly to Figure 3.33, the distances from the trapdoor centreline and from the base of the test chamber have been normalised by the diameter of the trapdoor.
The exact co-ordinates of all EPCs used in each test are given in Table 3.5 and Table 3.6, using the co-ordinate system shown in Figure 3.32.

The miniature EPCs were placed at offsets of $0.7D_{TD}$ and $1.2D_{TD}$ from the trapdoor as the author concluded from the literature (presented in Chapter 2) that these positions would be in close proximity to the boundary between negative and active arching zones. Therefore, these positions had the potential to provide the most useful data, as the variation in the stress ratio would be greatest.
<table>
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<tr>
<th>Test name</th>
<th>EPC name</th>
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<th>y co-ordinates [mm]</th>
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</tr>
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</tr>
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<td>75</td>
</tr>
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Table 3.5 Co-ordinates of miniature EPCs for tests CTD002-CTD004, expressed in both mm and as multiples of the trapdoor diameter (\( D_{TD} \))
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<tr>
<th>Test name</th>
<th>EPC name</th>
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<td>8128</td>
<td>175</td>
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</tbody>
</table>

Table 3.6 Co-ordinates of miniature EPCs for tests CTD005-CTD007, expressed in both mm and as multiples of the trapdoor diameter ($D_{TD}$)
3.8.2 MTBM tests

A series of MTBM tests were carried out. The results of six of these are presented in Chapter Five, namely TBM001 to TBM006. All EPCs were placed horizontally (i.e. measuring vertical stress) for the reason described in § 3.8.1.

The co-ordinate system shown in Figure 3.35 is used to describe the positions of the instrumentation used in the MTBM tests. The positions of the EPCs for this series of tests are shown in Figure 3.36, in which the distances from the tunnel centre have been normalised by its diameter. The exact co-ordinates of all EPCs used in each test are presented in Table 3.7 and Table 3.8. It should be noted that all cells were placed in a plane 475mm (5.4Df) from the front face of the test chamber, i.e. all EPCs have a z co-ordinate of 475mm. The position chosen reduced the likelihood of influence of the test chamber boundaries on the earth pressures recorded, whilst also allowing a considerable amount of data to be gathered as the MTBM approached and continued past the plane in which they were placed.

The positions of the settlement plates were the same for all tests in the MTBM test series, and are shown in Figure 3.37, in which the distances from the tunnel centre (x) and from the front face of the test chamber (z) have been normalised by its diameter. The exact co-ordinates of all settlement plates used in each test are given in Table 3.9.

The positions at which the surface settlement was measured using LVDTs varied between tests, and are presented in Table 3.10.
Figure 3.36 Normalised positions of miniature EPCs for MTBM tests, expressed as multiples of the tunnel face diameter
<table>
<thead>
<tr>
<th>Test name</th>
<th>EPC name</th>
<th>x co-ordinates</th>
<th>y co-ordinates</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>[mm]</td>
<td>[Df]</td>
</tr>
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</tr>
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</tr>
<tr>
<td></td>
<td>6</td>
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</tr>
<tr>
<td></td>
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Table 3.7 Co-ordinates of miniature EPCs for tests TBM001-TBM003, expressed in both mm and as multiples of the tunnel face diameter (Df)
<table>
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<th>Test name</th>
<th>EPC name</th>
<th>x co-ordinates [mm]</th>
<th>y co-ordinates [Df]</th>
<th>x co-ordinates [Df]</th>
<th>y co-ordinates [Df]</th>
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<tr>
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<td>132 1.5</td>
<td>132 1.5</td>
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<td>308 3.5</td>
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<td>7</td>
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<td>308 3.5</td>
<td>22 220 2.5</td>
<td>308 3.5</td>
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<tr>
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<td>484 5.5</td>
<td>45 132 1.5</td>
<td>0 0</td>
</tr>
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<td>46</td>
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<td>0 0</td>
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<td>46</td>
<td>132 2</td>
<td>200 2.3</td>
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Table 3.8 Co-ordinates of miniature EPCs for tests TBM004-TBM006, expressed in both mm and as multiples of the tunnel face diameter (Df)
Figure 3.37 Normalised positions of settlement plates for MTBM tests, expressed as multiples of the tunnel face diameter
<table>
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<th>y co-ordinates [mm]</th>
<th>z co-ordinates [mm]</th>
<th>x co-ordinates [Df]</th>
<th>y co-ordinates [Df]</th>
<th>z co-ordinates [Df]</th>
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<td>325 3.7</td>
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<tr>
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<td>325 3.7</td>
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<td>400 4.6</td>
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<td>550 6.3</td>
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<tr>
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<td>572 6.5</td>
<td>550 6.3</td>
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<td>11</td>
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<td>308 3.5</td>
<td>625 7.1</td>
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<tr>
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<td>132 1.5</td>
<td>700 8.0</td>
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<td>700 8.0</td>
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Table 3.9 Co-ordinates of settlement plates for MTBM tests, expressed in both mm and as multiples of the tunnel face diameter (Df)
<table>
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<th>Test</th>
<th>LVDT number</th>
<th>x co-ordinates [mm]</th>
<th>z co-ordinates [D_f]</th>
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</tr>
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<td>665</td>
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</tr>
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<td>0</td>
<td>500</td>
</tr>
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<td></td>
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<td>0</td>
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<td>500</td>
</tr>
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<td></td>
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<td>0</td>
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</tr>
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</tr>
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<td>500</td>
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<td>690</td>
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<td>3</td>
<td>260</td>
<td>475</td>
</tr>
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<td>0</td>
<td>475</td>
</tr>
<tr>
<td></td>
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<td>250</td>
<td>475</td>
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</table>

Table 3.10 Positions at which surface settlement was measured using LVDTs, expressed in both mm and as multiples of the tunnel face diameter (D_f)
4. Active trapdoor tests

4.1 Introduction

This chapter presents and discusses the results of six of the tests which were carried out to investigate the arching effect around an active trapdoor using the test chamber and trapdoor configuration described in Chapter 3. The stress acting on the trapdoor and in the soil mass was measured using a load cell and miniature EPCs respectively. As the load cell was placed underneath the trapdoor, it was assumed that the force measured by it was equal to the average vertical stress acting on the trapdoor. The EPCs were placed mainly directly above the trapdoor in tests CTD002 to CTD004, and primarily adjacent to it in tests CTD005 to CTD007. The exact positions of these are illustrated in Figures 3.37 and 3.38, and presented in Tables 3.5 and 3.6.

Reference is made throughout this and the subsequent chapters to positive and negative arching zones. A positive arching zone is one which experiences an increase in the stress acting on it due to the transfer of stress from adjacent soil masses. Conversely, a negative arching zone is one which experiences a decrease in the stress acting on it, because it is transferring stress to adjacent soil masses.

4.2 Trapdoor displacement

The method of measuring the trapdoor displacement is described in § 3.4.2 and shown in Figure 3.7. A comparison was made of the displacements measured by each of the LVDTs and the dial gauge (DG) during each of the trapdoor tests in the series. Typical results are shown in Figure 4.1, from test CTD005. There is very good agreement between the four measurements, and any tilt in the trapdoor due to the slightly differing displacements at each of the four corners is deemed negligible.
Chapter 4

Active trapdoor tests

4.3 Stress acting on trapdoor

As discussed in § 3.6.1.4, it was discovered during the series of trapdoor tests that sand grains were becoming trapped between the EPC and the test chamber base, as no adhesive was used to affix the cell. This led to erroneous results from all cells placed in this position. Therefore, the data from these cells have been omitted, and only the data from the load cell placed underneath the trapdoor are presented in this section. The force acting on the trapdoor measured by the load cell was converted to an average stress using the area of the trapdoor.

4.3.1 Presentation of results

Figure 4.2 shows the results from the load cell positioned underneath the trapdoor. The trapdoor displacement ($\delta$) has been normalised by its diameter ($D_{TD}$) in order to allow a comparison with the data in the literature, while the average stress acting on the trapdoor ($\sigma_{TD}$) is expressed as a proportion of the initial stress prior to any displacement taking place ($\sigma_{TD,i}$), and is termed the trapdoor stress ratio. The minimum and ultimate trapdoor stress ratios ($\sigma_{TD}/\sigma_{TD,i}\text{min}$ and $\sigma_{TD}/\sigma_{TD,i}\text{ult}$ respectively, as well as the corresponding normalised trapdoor displacements ($\delta/D_{TD}\text{min}$ and $\delta/D_{TD}\text{ult}$ respectively, are presented in Table 4.1. These variables are defined in Figure 4.3. The ultimate trapdoor stress ratio is defined as the constant stress ratio recorded after considerable trapdoor displacement.
Figure 4.2 Stress acting on trapdoor from tests CTD002-CTD007

Table 4.1 Minimum and ultimate normalised trapdoor stresses and corresponding displacements, from tests CTD002-CTD007

<table>
<thead>
<tr>
<th>Test name</th>
<th>$\left(\frac{\sigma_{TD}}{\sigma_{TD,i}}\right)_{\min}$ [-]</th>
<th>$\left(\frac{\delta}{D_{TD}}\right)_{\min}$ [%]</th>
<th>$\left(\frac{\sigma_{TD}}{\sigma_{TD,i}}\right)_{\ult}$ [-]</th>
<th>$\left(\frac{\delta}{D_{TD}}\right)_{\ult}$ [%]</th>
</tr>
</thead>
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<td>0.167</td>
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<td>0.196</td>
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<td>6.9</td>
</tr>
<tr>
<td>CTD007</td>
<td>0.084</td>
<td>1.02</td>
<td>0.213</td>
<td>6.2</td>
</tr>
</tbody>
</table>

4.3.2 Discussion of results

4.3.2.1 General behaviour

Although there were some differences in the results obtained from the load cell, there was generally good repeatability between tests, and a common trend was evident. On initial trapdoor yielding, $\sigma_{TD}$ decreased rapidly to a minimum value of approximately $0.1\sigma_{TD,i}$ and occurred at a normalised trapdoor displacement of approximately 1%. Upon reaching this value, the trapdoor stress ratio then increased gradually to a constant value of approximately 0.2, occurring at a normalised displacement of approximately 6%. A similar trend and values are common in the literature, as discussed in Chapter 2.
4.3.2.2 Stress-transfer mechanisms

The behaviour described in § 4.3.2.1 has been explained by several authors, including Evans (1983), with reference to Figure 4.3, as follows. After a small amount of trapdoor yield, the soil behaves elastically and the stress acting above the trapdoor is transmitted to the adjacent parts. Simultaneously, the stress acting on the trapdoor decreases to a minimum value (the point of maximum arching, point 1) and the first shear bands begin to form from the edge of the trapdoor to a point above its axis, forming a dome 'a'. On continued yielding of the trapdoor, the soil behaves plastically and dilation of the failing mass occurs, resulting in an increase in the load acting on the trapdoor (between points 1 and 2), and further shear bands ('b' and 'c') extend from the trapdoor edge at an angle greater than those formed by previous shear bands. Finally, after a considerable amount of trapdoor yielding, vertical shear bands are formed from the trapdoor edge to the soil surface ('d' in Figure 4.3). This represents complete development of the failure surface and is the point of minimum arching, and the load acting on the trapdoor is constant (point 2). The development of external failure surfaces (i.e. outside the zone directly above the trapdoor) on significant trapdoor displacements of between 14 and 29% by Costa et al. (2009) was discussed in § 2.5.2.3 and illustrated in Figure 2.24.

![Figure 4.3 Typical trapdoor load displacement curve and shear band propagation, after Evans (1983)](image)

4.3.2.3 Comparison of experimental data with analytical solutions

A comparison is made in this section between the experimental values and those obtained using some of the analytical solutions discussed in Chapter 2. As the mechanism described in § 4.3.2.2 is a progressive failure, none of the solutions are able to continuously predict
the load acting on the trapdoor as displacement takes place. However, the minimum and ultimate trapdoor stress ratios are more easily predicted, and these values are compared with those obtained experimentally.

The negative value of $F_{ult}$ obtained from the solution of Bierbaumer (1913) for certain $H$ values was discussed in § 2.5.1.1. Using Eqn. 2.39, a $H_{OB}/D_{TD}$ ratio of 2.65 was obtained for the Glenview sand used in this study ($\varphi=36^\circ$). This solution is therefore not suitable as the ratio for the experimental set-up was 6.

The minimum trapdoor stress ratio obtained from the trapdoor tests is compared in Table 4.2 with those calculated from analytical solutions. The values presented for the solutions of Janssen (1895), Evans (1983) and Iglesia (1991) were obtained from Eqns. 2.23, 2.31 and 2.32, and 2.41, respectively. Where applicable, the coefficient of lateral earth pressure ($K_{ep}$) used to calculate this value is also presented. This does not apply to the methods of Bierbaumer (1913) and Evans (1983), as the equations presented by these authors are not dependent on $K_{ep}$. Although Silo Theory (Janssen, 1895) provides the closest prediction, all of the solutions significantly underpredict the minimum stress on the trapdoor, particularly that of Engesser. The amendments proposed by Iglesia (1991) to Engesser’s solution regarding $K_{ep}$ and $\varphi$ result in an improvement to the predicted value, but not to within acceptable levels.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$K$</td>
<td>N/A</td>
<td>$K_a$</td>
<td>$K_{ep}=1$</td>
<td>N/A</td>
<td>N/A</td>
<td>$K'$</td>
</tr>
<tr>
<td>$(\sigma_{TD}/\sigma_{TD,i})_{min}$ [-]</td>
<td>0.087</td>
<td>0.019</td>
<td>0.058</td>
<td>0.038</td>
<td>0.038</td>
<td>0.042</td>
</tr>
</tbody>
</table>

Table 4.2 Comparison of experimental $(\sigma_{TD}/\sigma_{TD,i})_{min}$ value with those obtained from analytical solutions

The experimental value of ultimate trapdoor stress ratio is compared with the analytical solutions in Table 4.3, in addition to the $K_{ep}$ value used in the calculations. The values presented for the solutions of Evans (1983) and Terzaghi (1936) were obtained from Eqns. 2.33 and 2.40, respectively. The former solution provides a marginally better prediction than the latter, but both significantly underpredict the ultimate stress acting on the trapdoor.
Table 4.3 Comparison of experimental \(\sigma_{TD}/\sigma_{TD,0}\text{alt}\) value with those obtained from analytical solutions

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>K</td>
<td>N/A</td>
<td>K(_{cp}=1)</td>
<td>K(_{cp}=1.2)</td>
</tr>
<tr>
<td>(\sigma_{TD}/\sigma_{TD,0}\text{alt})</td>
<td>0.199</td>
<td>0.059</td>
<td>0.061</td>
</tr>
</tbody>
</table>

4.3.2.4 Selection of coefficient of lateral earth pressure

One of the principal variables in the solutions described and used above is the coefficient of lateral earth pressure (K\(_{cp}\)). Terzaghi (1936) assumed K\(_{cp}=1\), after recording this value directly above the trapdoor in his experimental work. An average value of 1.2 was found by Whitman and Luscher (1962) above a structure undergoing active arching, and by Evans (1983). Other K\(_{cp}\) values used in the literature include Krynine’s coefficient of earth pressure (K’), Eqn. 2.29, the coefficient of earth pressure at-rest (K\(_{0}\)) and the coefficient of active earth pressure (K\(_{a}\)), where:

\[
K_0 = 1 - \sin \phi \quad \text{Eqn. 4.1}
\]
\[
K_a = \frac{1 - \sin \phi}{1 + \sin \phi} \quad \text{Eqn. 4.2}
\]

A comparison is made in Table 4.4 of the minimum trapdoor stress ratio for various K\(_{cp}\) values using the solutions which are dependent on the K\(_{cp}\) value selected, i.e. those of Engesser (1882), Janssen (1895) and Iglesia (1991). It is immediately evident that the choice of K\(_{cp}\) results in significantly different stress values being obtained, and therefore highlights the importance of choosing the appropriate value. Even considering a broad range of K\(_{cp}\) values, the solutions of Engesser (1882) and Iglesia (1991) still underpredict the measured value of the minimum trapdoor stress ratio from the trapdoor tests, although the latter is much closer than the former. The variation in K\(_{cp}\) for Janssen’s Silo Theory (1895) results in a much wider range of values. In order to match the measured value of \(\sigma_{TD}/\sigma_{TD,0}\text{min}\) of 0.087, K\(_{cp}\) values of 5.09, 0.66 and 2.15 are necessary for the solutions of Engesser (1882), Janssen (1895) and Iglesia (1991), respectively. The variation in these values is considerable, whilst the necessary value of K\(_{cp}\) for Engesser’s solution in particular is very high. The necessary K\(_{cp}\) value for Janssen’s formula is greater than K’ but lower than the values measured experimentally by Terzaghi (1936) and Evans (1983).
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<table>
<thead>
<tr>
<th>Method</th>
<th>$K_a$ (=0.260)</th>
<th>$K_0$ (=0.412)</th>
<th>$K'$ (=0.486)</th>
<th>$K_{cp}=1$ (after Terzaghi, 1936)</th>
<th>$K_{cp}=1.2$ (after Evans, 1983)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Engesser (1882)</td>
<td>0.019</td>
<td>0.021</td>
<td>0.022</td>
<td>0.030</td>
<td>0.033</td>
</tr>
<tr>
<td>Janssen (1895)</td>
<td>0.219</td>
<td>0.139</td>
<td>0.118</td>
<td>0.058</td>
<td>0.048</td>
</tr>
<tr>
<td>Iglesia (1991)</td>
<td>0.036</td>
<td>0.041</td>
<td>0.042</td>
<td>0.057</td>
<td>0.062</td>
</tr>
</tbody>
</table>

Table 4.4 Effect of $K_{cp}$ on values of $(\sigma_{TD}/\sigma_{TD,i})_{min}$

A comparison is made in Table 4.5 of the ultimate trapdoor stress ratio for various $K_{cp}$ values using the same solutions as in Table 4.3. Again, the importance of choosing an appropriate $K_{cp}$ value is reflected by the broad range of values obtained. A $K_{cp}$ value of 0.392, which lies between $K_a$ and $K_0$, is necessary for Terzaghi’s solution in order to match the average measured value of $(\sigma_{TD}/\sigma_{TD,i})_{ult}$ of 0.199. Using Evans’ solution and the $K_{cp}$ value of 1.2 measured in his experimental work to calculate $(\sigma_{TD}/\sigma_{TD,i})_{ult}$ results in a significant under-prediction of the measured value. However, the measured value is predicted exactly using $K'$ in the solution.

<table>
<thead>
<tr>
<th>Method</th>
<th>$K_a$ (=0.260)</th>
<th>$K_0$ (=0.412)</th>
<th>$K'$ (=0.486)</th>
<th>$K_{cp}=1$ (after Terzaghi, 1936)</th>
<th>$K_{cp}=1.2$ (after Evans, 1983)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Terzaghi (1936)</td>
<td>0.320</td>
<td>0.187</td>
<td>0.150</td>
<td>0.059</td>
<td>0.048</td>
</tr>
<tr>
<td>Evans (1983)</td>
<td>0.390</td>
<td>0.243</td>
<td>0.199</td>
<td>0.076</td>
<td>0.061</td>
</tr>
</tbody>
</table>

Table 4.5 Effect of $K_{cp}$ on values of $(\sigma_{TD}/\sigma_{TD,i})_{ult}$

It is evident from the predictions of $(\sigma_{TD}/\sigma_{TD,i})_{min}$ and $(\sigma_{TD}/\sigma_{TD,i})_{ult}$ that the appropriate coefficient of lateral stress for use in the analytical calculations is not obvious, and that there is considerable variation in the $K_{cp}$ values used in the solutions. However, the use of finite element (FE) analysis may be useful in this regard.

4.4 Stress acting directly above trapdoor

4.4.1 Introduction

The stress within the soil mass was measured during tests CTD002-CTD007 using a number of miniature earth pressure cells (EPCs) placed within the soil mass. The purpose of this section is to present and discuss the results of tests CTD002-CTD004, in which the
vertical stress was measured primarily above the active trapdoor. The positions of the EPCs in these tests are presented in Table 3.5.

The variation in vertical stress due to the displacement of the active trapdoor is quantified in terms of the vertical stress ratio, which is defined as the ratio of the vertical stress at a point to the initial vertical stress prior to any trapdoor displacement occurring \( (\sigma_v/\sigma_{v,i}) \). Although the results obtained from the miniature EPCs in tests CTD002-CTD004 were generally good, there were on occasion functionality issues with some of them. The data from all EPCs placed on the base of the test chamber were omitted, as discussed in § 4.3. In the case of a small number of EPCs (generally one or two per test), no change in the voltage output was measured after placing and subsequent filling and trapdoor displacement. This may be due to placement effects, as discussed in § 3.6.1.2. These data have been omitted.

4.4.2 Presentation of results

The results of the tests investigating the arching effect above the active trapdoor were broadly similar. As expected, there is a decrease in the stress acting above the displacing trapdoor due to the arching effect. To illustrate this, the vertical stress ratios obtained from test CTD003 are shown in Figure 4.4. These data are typical of tests CTD002-CTD004.

![Figure 4.4 Vertical stress ratio at various distances directly above displacing trapdoor from CTD003](image_url)

The results from the EPCs placed in close proximity to the trapdoor are compared to those obtained from the load cell placed underneath it in Figure 4.5 and Table 4.6.
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Figure 4.5 Comparison of load cell and EPC results from test CTD003

<table>
<thead>
<tr>
<th>Measurement</th>
<th>( \frac{\sigma_v}{\sigma_{v,i}} ) (_{min} ) [-]</th>
<th>( \frac{\delta}{D_{TD}} ) (_{min} ) [%]</th>
<th>( \frac{\sigma_v}{\sigma_{v,i}} ) (_{ult} ) [-]</th>
<th>( \frac{\delta}{D_{TD}} ) (_{ult} ) [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>EPC at 0.25D(_{TD})</td>
<td>0.056</td>
<td>3.18</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>EPC at 0.5D(_{TD})</td>
<td>0.042</td>
<td>3.67</td>
<td>0.067</td>
<td>7.02</td>
</tr>
<tr>
<td>Load cell</td>
<td>0.065</td>
<td>0.80</td>
<td>0.167</td>
<td>6.6</td>
</tr>
</tbody>
</table>

Table 4.6 Comparison of minimum and ultimate vertical stress ratios and corresponding displacements from test CTD003

4.4.3 Discussion of results

4.4.3.1 General behaviour

It is clear from Figure 4.4 that the zone directly above the trapdoor centreline is a negative arching zone. At all points there is a reduction in the vertical stress on initial trapdoor displacement. The rate at which this reduction occurs decreases as the distance from the trapdoor increases. From the data presented in the figure, it is also evident that the magnitude of the arching effect decreases as the distance from the soil surface decreases - the cells positioned at 1D\(_{TD}\), 3D\(_{TD}\), and 4D\(_{TD}\) reach vertical stress ratios of approximately 0.36, 0.76 and 0.85 respectively at a trapdoor displacement of 13%. This may be explained by consideration of the length of the shear bands which develop in the soil mass. As the vertical distance from the trapdoor increases, the shear planes through which stress transfer is achieved (as discussed in § 4.3.2.2) are shorter than those at greater depths, due to their intersection with the soil surface. Therefore as the shear plane reduces in length, the magnitude of stress which can be transferred along it reduces correspondingly.

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It is considered that the cell at 3TD not appearing to reach an ultimate value does not signify that total development of the failure mechanism has not occurred. This behaviour is more likely due to placement effects, as this cell also appears to show an initial greater decrease in the vertical stress ratio than at 4TD, which itself reaches a constant value at a trapdoor displacement of approximately 6%.

Figure 4.6 shows the vertical stress ratios from CTD003 previously presented in Figure 4.4 on initial trapdoor displacement in more detail. After a very small amount of normalised trapdoor displacement (of the order of 0.2%), even cells at considerable distance from the trapdoor (for example, at 4TD), experience a decrease in stress. As stated above, the fact that the cell at 4TD experiences a greater initial decrease in stress than the one at 3TD is thought to be due to placement effects rather than due to any specific mechanism occurring. After a displacement of 3.4%, the stress relationship between these two positions becomes more intuitively correct, as the stress decrease for 3TD becomes greater than that at 4TD.

![Figure 4.6](image)

**Figure 4.6** Detailed view of vertical stress ratio at distances directly above displacing trapdoor from CTD003

### 4.4.3.2 Comparison of EPC data with load cell data

The vertical stress ratios from the cells placed in close proximity to the trapdoor (illustrated in Figure 4.5) show a similar trend to the load cell (LC) placed underneath the trapdoor on initial trapdoor displacement; that is, a sharp decrease of stress to a minimum. For the EPC positioned at 0.25TD from the base, the stress was approximately constant after the minimum value was reached. The behaviour of the EPC at 0.5TD was similar to that of
the load cell after this minimum value, i.e. there was a subsequent small increase in the vertical stress to a constant value.

From the data presented in Table 4.6, the minimum vertical stress ratio at each of the three locations is broadly similar. It is clear that the trapdoor displacement at which this stress occurs is much greater for the EPCs than for the load cell, i.e. there appears to be a “lag”. This is probably due to the development of the failure mechanism through the sand mass as trapdoor yield continues.

4.4.3.3 Stress-transfer mechanisms
As illustrated in Figure 4.5, the EPC at \(0.25D_{TD}\) did not record an increase in stress to a constant after the minimum value. This may be explained as follows. As the soil began to behave plastically (during the process described in § 4.3.2.2), the soil mass bounded by the shear band ‘a’ in Figure 4.3 began to dilate. This resulted in an increase in the stress acting on the trapdoor, and an increase at \(0.5D_{TD}\), which was probably near the vertical extent of the shear band, and was therefore bounded for a time by the unyielding soil mass above it. As \(0.25D_{TD}\) was within the dilating soil mass itself rather than at its extents, it only experienced a decrease in stress. This may also explain why the ultimate stress measured by the load cell was greater than that measured by the EPC at \(0.5D_{TD}\). Prior to the soil mass above \(0.5D_{TD}\) failing, an increase in stress was recorded. However, as the soil mass above \(0.5D_{TD}\) began to fail, the increase stopped as the EPC was then within a dilating zone.

4.4.3.4 Repeatability of tests
Other than the positions of some of the miniature EPCs varying slightly (as shown in Table 3.5), tests CTD002-CTD004 were configured and performed in an identical fashion. To ensure repeatability between tests, the vertical stress ratios obtained from EPCs placed in the same position in different tests were compared. Figure 4.7 and Figure 4.8 show the results from EPCs placed directly above the centre of the trapdoor at distances of \(0.5D_{TD}\) and \(3D_{TD}\) from the test chamber base, respectively. These results are typical of the data collected. The legends in these figures identify the test from which the data is from and the name of the EPC used.

Figure 4.7 shows very good repeatability between the cells placed at \(0.5D_{TD}\). Although the repeatability for cells placed at \(3D_{TD}\) (Figure 4.8) is not to the same level, it is still deemed
to be within acceptable limits. It is proposed that the small differences in stress between the cells are due to placement effects and small variations in their positioning, despite the considerable efforts made to ensure that the cells were placed accurately and in the same way.

![Comparison of vertical stress ratio at 0.5D_{TD} from test chamber base](image)

**Figure 4.7** Comparison of vertical stress ratio at 0.5D_{TD} from test chamber base

![Comparison of vertical stress ratio at 3D_{TD} from test chamber base](image)

**Figure 4.8** Comparison of vertical stress ratio at 3D_{TD} from test chamber base

### 4.5 Stress acting adjacent to trapdoor

The purpose of this section is to investigate the arching effect adjacent to the trapdoor. To this end, the results of tests CTD005-CTD007 are presented and discussed, in which the soil mass adjacent to the trapdoor was primarily instrumented with miniature EPCs in order to measure the vertical stress. The positions of the cells in these tests are presented in

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Table 3.6. In the same way as for tests CTD002-CTD004, some EPCs failed to provide useful results, in which case the data obtained have been omitted.

4.5.1 Presentation of results

The results of tests CTD005-CTD007 were broadly similar. The results of test CTD005 are presented in Figure 4.9 and Figure 4.10, which show the vertical stress ratios at various distances from the test chamber base at a distance of $0.7D_{TD}$ (105mm) and $1.2D_{TD}$ (175mm) from the tunnel centreline, i.e. 30mm and 100mm from the edge of the trapdoor, respectively. These data are typical of those obtained in tests CTD005-CTD007.

![Figure 4.9 Vertical stress ratio $0.7D_{TD}$ from trapdoor centreline measured in test CTD005](image)

![Figure 4.10 Vertical stress ratio $1.2D_{TD}$ from trapdoor centreline measured in test CTD005](image)
4.5.2 Discussion of results

4.5.2.1 General behaviour

It is evident from Figure 4.9 that on initial trapdoor yielding, miniature EPCs placed 0.7D\textsubscript{TD} from the trapdoor centreline (i.e. 30mm from the trapdoor edge) show a decrease in stress. The largest reduction in the vertical stress ratio occurs at 0.5D\textsubscript{TD} from the base, as a gradual reduction to a minimum value of 0.82 is observed. A similar minimum ratio of approximately 0.85 is experienced at 1D\textsubscript{TD} and 2D\textsubscript{TD} from the base, although the final ratio is marginally higher at the former than at the latter.

At 1.2D\textsubscript{TD} from the trapdoor centreline, there is an increase in the vertical stress ratio, as illustrated in Figure 4.10. Despite some variability in the recorded ratio on initial trapdoor displacement at 1D\textsubscript{TD} from the base, there is a general trend of the ratio gradually increasing with trapdoor displacement. The magnitude of the arching effect increases with the distance from the base. A maximum value is reached at approximately 10% trapdoor displacement, before reducing from this peak as displacement continues.

4.5.2.2 Stress-transfer mechanisms

The behaviour which is evident in Figure 4.9 clearly does not concur with the suggestion in the literature (Costa \textit{et al.}, 2009) that external failure surfaces only develop on significant trapdoor displacement. If the failure surfaces developed as shown in Figure 2.24, the stresses at the points shown in Figure 4.9 should increase, rather than decrease. In order to explain a decrease in stress, an external shear band must develop, somewhat similar to the curved failure surface proposed by Terzaghi (1936). However, this was proposed for the ultimate state (i.e. significant trapdoor yielding), rather than the small trapdoor displacement seen in this situation.

From the results shown in Figure 4.10, it is evident that the EPCs at the points presented are initially outside the extent of any failure surfaces, and therefore stress from adjacent zones is transmitted to them. However, as the extent of the external failure surfaces develop, these points are then transferring stress to adjacent zones which are at a greater distance from the trapdoor edge, i.e. these points are part of a negative arching zone. This is reflected in the reduction in stress at a relatively large normalised trapdoor displacement of the order of 10%.
4.5.2.3 Repeatability of tests

The repeatability of tests CTD005-CTD007 was examined by comparing the results from EPCs placed at the same positions. Two such positions are shown in Figure 4.11 and Figure 4.12, showing the results from EPCs placed at distances of $0.5D_{TD}$ and $4D_{TD}$ from the test chamber base and $0.7D_{TD}$ and $1.2D_{TD}$ from the trapdoor centreline, respectively. The legends in both figures identify the test and the EPC used. These results are typical of the data collected.

The data in Figure 4.11 can be considered to compare reasonably well. Although very similar values are evident from the different sources presented in Figure 4.12, they show differing trends, as cell 7 from test CTD006 shows a marginal decrease in stress, rather than the marginal increase measured in the other two tests. Therefore, it is clear that although the EPCs may provide very useful results, there are some limitations with their consistency, and this should be considered when assessing the results of the tests.

Figure 4.11 Comparison of vertical stress ratio at $0.5D_{TD}$ from test chamber base and $0.7D_{TD}$ from trapdoor centreline
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4.6 General behaviour of arching zone

The typical failure mechanism suggested in the literature consists of internal failure surfaces (i.e. directly above the trapdoor) on initial trapdoor displacement followed by the development of external failure surfaces (i.e. beyond the vertical boundaries of the trapdoor) at much larger displacements. The results which were obtained using the miniature EPCs presented in the preceding sections are contrary to this. A combination of internal and external failure surfaces appear to develop immediately upon trapdoor yielding. The primary internal failure surface is evident from the reduction in stress at 0.5D_{TD} from the trapdoor in Figure 4.4. As a subsequent increase in stress occurs due to dilation, this point appears to be at the vertical extent of the failed zone. However, immediately upon trapdoor yielding, a reduction in stress is also experienced by the EPCs placed outside the vertical extent of the trapdoor (as presented in Figure 4.9). This shows the development of an external failure surface, similar to that shown in Figure 2.24. The development of this failure surface is evident from the cells placed 100mm from the trapdoor edge by the small decrease in stress following the initial increase, as shown in Figure 4.10, as the soil in this zone begins to transfer stress to adjacent parts.

4.7 Summary of chapter

The stress ratio-trapdoor displacement curve obtained from the load cell placed underneath the trapdoor (Figure 4.2) concurs with those from similar trapdoor tests in the literature,
such as that of Terzaghi (1936). The development of shear bands identified by others (for example Evans, 1983) could be inferred from these data. The values of \((\sigma_{\text{TD}}/\sigma_{\text{TD,i}})_{\text{ult}}\) and \((\sigma_{\text{TD}}/\sigma_{\text{TD,i}})_{\text{ult}}\) measured by the load cell compared poorly with those obtained from a number of different analytical solutions. This is considered to be due to the many inherent assumptions implied in their derivations. The solution of Janssen (1895) was found to provide the best prediction of the \((\sigma_{\text{TD}}/\sigma_{\text{TD,i}})_{\text{ult}}\), but this still underestimated the measured value by 33%. Evans’ solution (1983) provided a marginally better prediction of \((\sigma_{\text{TD}}/\sigma_{\text{TD,i}})_{\text{ult}}\) than that of Terzaghi (1936), although it was underpredicted by 70%.

The dependency of \((\sigma_{\text{TD}}/\sigma_{\text{TD,i}})_{\text{ult}}\) and \((\sigma_{\text{TD}}/\sigma_{\text{TD,i}})_{\text{ult}}\) on the coefficient of lateral earth pressure \((K_{\text{ep}})\) was highlighted. The difficulty in selecting the appropriate \(K_{\text{ep}}\) value to use in the analytical solutions was also identified. Very high and possibly unrealistic \(K_{\text{ep}}\) values of 5.09 and 2.15 were required in the solutions of Engesser (1882) and Iglesia (1991) to match the experimental value \((\sigma_{\text{TD}}/\sigma_{\text{TD,i}})_{\text{ult}}\). A value of 0.392 was required for the solution of Terzaghi (1936) to match the measured value of \((\sigma_{\text{TD}}/\sigma_{\text{TD,i}})_{\text{ult}}\), whilst that of Evans (1983) matched it exactly using Krynine’s coefficient of earth pressure \((K')\) value \((=0.486)\), which was considerably lower than the value of 1.2 measured in the corresponding experimental investigation.

The arching effect on the vertical stresses acting directly above the trapdoor centreline and its relationship to the degree of trapdoor displacement has been quantified. Minimum ultimate stress ratios ranging from 0.056 to approximately 0.85 were recorded from \(0.25D_{\text{TD}}\) to \(4D_{\text{TD}}\) above the yielding trapdoor, respectively. The behaviour of the miniature EPCs placed in close proximity to the trapdoor compared well with that of the load cell measuring the stress acting on the trapdoor itself, thereby verifying the performance of the miniature EPCs. The difference in behaviour between the cells placed at \(0.25D_{\text{TD}}\) and \(0.5D_{\text{TD}}\) from the trapdoor was explained by consideration of a zone of dilating soil directly above the trapdoor.

Negative arching was observed at all points at \(0.7D_{\text{TD}}\) from the trapdoor centreline, with the maximum arching effect occurring at \(0.5D_{\text{TD}}\) from the base. Positive arching was observed at all points at \(1.2D_{\text{TD}}\) from the trapdoor centreline, where the maximum arching was recorded at \(2D_{\text{TD}}\) from the base. This was followed by subsequent negative arching as the failure mechanisms extended from the trapdoor on significant trapdoor displacement.
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The experimentally observed behaviour of the arching zone is inconsistent with the accepted view in the literature of the development of failure mechanisms around an active trapdoor. Rather than the development of external failure surfaces after significant trapdoor displacement, the results from the miniature EPCs suggest that both internal and external surfaces develop immediately upon trapdoor yielding.

The miniature EPCs used in the active trapdoor tests generally performed well. There were however some issues with their functionality and repeatability. Although their use involves some difficulties, generally due to placement effects probably because of their small dimensions, it has been demonstrated throughout this chapter that they can provide very useful information on the arching effect and the mechanisms occurring.
5. Miniature TBM tests

5.1 Introduction

The results of the series of tests which were carried out primarily to investigate the arching effect due to the progression of a purpose-built miniature tunnel boring machine (MTBM) through a sand mass are presented and discussed in this chapter. The stresses acting within the sand were recorded using miniature earth pressure cells (EPCs), whilst the pressure acting on the face of the MTBM and both surface and sub-surface settlements were also measured. Further details of the tunnelling equipment and instrumentation used are presented in Chapter 3.

A series of six MTBM tests were carried out, namely TBM001 to TBM006. Full details of the configurations of the instrumentation used in these tests are provided in § 3.9.2. Various problems were encountered during some of these tests, which in addition to their impacts, are summarised below. In test TBM001, the placement of the LVDTs on the sand surface outside their working range led to no surface settlement data being recorded, whilst a problem with the data logger of the load cell used to measure the face pressure led to the loss of these data. Incorrect configuration of the cutting face and the bar for forward advancement of the lining (see Figure 3.11) led to poor face pressure data being obtained in tests TBM004 and TBM006. Finally, in test TBM006, a mechanical problem developed with the connection between the threaded bar for forward movement of the carriage and the motor which rotates it (Figure 3.10), which meant that forward movement of the MTBM was not possible. This resulted in an MTBM advancement of only 0.546m, in comparison with an average distance of 0.76m in the five other tests in the series. These problems are mentioned where relevant throughout the chapter.

Many of the results in this chapter are presented in terms of normalised distances. The distances from the tunnel face, tunnel centreline and tunnel crown are normalised by the diameter of the cutting face (Df). The distance from the surface to some point below the surface (z) is normalised by the distance to the tunnel axis (z0). This has been carried out in order to allow a comparison with the data presented in the literature.
Chapter 5 Miniature TBM tests

The purpose of this chapter is to present the results obtained from the series of MTBM tests, in order to develop the understanding of the settlements and associated arching effects. Initially, the relationship between MTBM parameters (such as excavation ratios and face pressure) and settlements are discussed. Following this, the surface and subsurface settlements as well as the corresponding settlement trough parameters are presented. Finally, the stress transfer mechanisms occurring within the soil mass are discussed.

5.2 Excavation and advancement rates

5.2.1 Introduction

The MTBM consists of a rotary cutting face and a lining. Three cutting blades are incorporated in the cutting face, through which the excavated soil passes into the tapered section of the lining behind the cutting face, as discussed in § 3.4.3.2. The spoil is then removed using a vacuum. After each drive, the mass of the soil which was excavated was measured and converted to a weight. The excavation rate (Q) was then calculated as this weight divided by the length excavated during the drive under consideration. The drive length was measured by a rotary potentiometer (§ 3.4.3.3). The advancement rate of the MTBM (v) was also calculated for each drive as the ratio of the length of the drive under consideration to the time taken to complete it.

The ideal theoretical excavation rate \( Q_{\text{theor}} \) was calculated from the average weight of the sand times the area of the face of the MTBM \( (A_f) \). The over-excavation ratio \( Q' \), which under ideal tunnelling conditions will equal one, is defined in Eqn. 5.1:

\[
Q' = \frac{Q}{Q_{\text{theor}}} \quad \text{Eqn. 5.1}
\]

The number of cutting blades in operation at the cutting face and the face pressure were used to control the excavation ratio. When a cutting blade was not used during a test, it was sealed on both the inside and outside of the cutting face. When inspected after each test, there was no evidence of any damage to these seals which might have allowed sand ingress.
5.2.2 Presentation of results

The average excavation rates for each test are presented in Table 5.1. The over-excavation ratio and the number of cutting blades used during each test are also presented. The standard deviations of excavation rates for each test, obtained from the individual drive excavation rates, are shown in Table 5.2, and are also expressed as a percentage of the average excavation rate.

<table>
<thead>
<tr>
<th>Test name</th>
<th>Total weight of excavated soil [N]</th>
<th>Total excavated length [m]</th>
<th>Average excavation rate, Q_{avg} [N/m]</th>
<th>Over-excavation ratio, Q' [-]</th>
<th>Cutting blades used</th>
</tr>
</thead>
<tbody>
<tr>
<td>TBMOOl</td>
<td>139.38</td>
<td>0.783</td>
<td>178.01</td>
<td>2.02</td>
<td>3</td>
</tr>
<tr>
<td>TBM002</td>
<td>111.08</td>
<td>0.755</td>
<td>147.12</td>
<td>1.67</td>
<td>3</td>
</tr>
<tr>
<td>TBM003</td>
<td>99.86</td>
<td>0.753</td>
<td>132.61</td>
<td>1.50</td>
<td>2</td>
</tr>
<tr>
<td>TBM004</td>
<td>87.29</td>
<td>0.784</td>
<td>113.34</td>
<td>1.29</td>
<td>1</td>
</tr>
<tr>
<td>TBM005</td>
<td>77.30</td>
<td>0.737</td>
<td>104.88</td>
<td>1.19</td>
<td>1</td>
</tr>
<tr>
<td>TBM006</td>
<td>60.78</td>
<td>0.546</td>
<td>111.31</td>
<td>1.26</td>
<td>1</td>
</tr>
</tbody>
</table>

Table 5.1 Details of spoil removal and cutting face configuration

<table>
<thead>
<tr>
<th>Test name</th>
<th>Standard deviation, Q_{std.dev.} [N/m]</th>
<th>Q_{std.dev.}/Q_{avg} [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>TBMOOl</td>
<td>66.69</td>
<td>37.5</td>
</tr>
<tr>
<td>TBM002</td>
<td>49.58</td>
<td>33.7</td>
</tr>
<tr>
<td>TBM003</td>
<td>40.71</td>
<td>30.7</td>
</tr>
<tr>
<td>TBM004</td>
<td>34.40</td>
<td>27.8</td>
</tr>
<tr>
<td>TBM005</td>
<td>32.41</td>
<td>30.9</td>
</tr>
<tr>
<td>TBM006</td>
<td>45.86</td>
<td>41.2</td>
</tr>
</tbody>
</table>

Table 5.2 Variability in MTBM excavation rates

The effect of altering Q by varying the number of cutters used during each test is presented in Figure 5.1, which shows the average over-excavation ratio (Q'_{avg}) achieved when a specified cutting face configuration was used. The Q'_{avg} value corresponding to the use of three cutters, for example, was calculated by averaging the Q' values measured in tests TBMOOl and TBM002 which are presented in Table 5.1.
The relationship between $Q'_{\text{avg}}$ and the maximum surface volume loss measured during each test ($V_{l,\text{max}}$), is illustrated in Figure 5.2, where $V_{l}$ is as defined in Eqn. 2.4. The surface trough width parameter ($K_s$), which is necessary to calculate $V_{l}$, could not be determined in tests TBM002-TBM004 due to the configuration of the LVDTs placed on the sand surface to measure the surface settlement (the positions of which are presented in Table 3.10). The $K_s$ value used to calculate $V_{l}$ presented in Figure 5.2 was therefore an average value of those measured in tests TBM005 and TBM006 (presented in Table 5.5). The method of determining the volume loss from the surface settlements measured by the LVDTs on the sand surface is discussed in further detail in § 5.4.2.
The average MTBM advancement rates ($v_{\text{avg}}$) and the standard deviations of advancement rate ($v_{\text{st.dev.}}$) for each test, obtained from the individual drive rates, are presented in Table 5.3. The standard deviations are also expressed as a percentage of $v_{\text{avg}}$.

<table>
<thead>
<tr>
<th>Test name</th>
<th>Average MTBM advancement rate, $v_{\text{avg}}$ [mm/s]</th>
<th>Standard deviation, $v_{\text{st.dev.}}$ [mm/s]</th>
<th>$v_{\text{st.dev.}}/v_{\text{avg}}$ [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>TBM001</td>
<td>0.317</td>
<td>0.191</td>
<td>60.3</td>
</tr>
<tr>
<td>TBM002</td>
<td>0.489</td>
<td>0.184</td>
<td>37.6</td>
</tr>
<tr>
<td>TBM003</td>
<td>0.756</td>
<td>0.249</td>
<td>33.0</td>
</tr>
<tr>
<td>TBM004</td>
<td>0.590</td>
<td>0.132</td>
<td>22.3</td>
</tr>
<tr>
<td>TBM005</td>
<td>0.411</td>
<td>0.082</td>
<td>19.9</td>
</tr>
<tr>
<td>TBM006</td>
<td>0.331</td>
<td>0.193</td>
<td>58.3</td>
</tr>
</tbody>
</table>

Table 5.3 Details of MTBM advancement rate

5.2.3 Discussion of results

Despite many preliminary tests being performed prior to the MTBM test series (which are not discussed in further detail), a decrease in the ratio of $Q_{\text{st.dev.}}/Q_{\text{avg}}$ after TBM001 from 37.5% to approximately 30% for tests TBM002-TBM005 is evident in the data presented in Table 5.2. This is due to increasing familiarity with the operation of the MTBM by the author. The $Q_{\text{st.dev.}}/Q_{\text{avg}}$ ratio of approximately 30% is interrupted by the value of 41.2% in test TBM006, which occurred due to the problem described in § 5.1 coupled with continued rotation of the cutting face during the final drive, resulting in a high excavation rate. By excluding this drive from the standard deviation calculation, a much lower $Q_{\text{st.dev.}}/Q_{\text{avg}}$ value of 15.2% is found.

A linear relationship between $Q'_{\text{avg}}$ and the number of cutters used at the face is evident from Figure 5.1. A linear relationship between the maximum volume loss and $Q'_{\text{avg}}$ is shown in Figure 5.2. The surface settlements resulting from the progression of the MTBM through the sand mass is discussed in further detail in § 5.4. The high values of $V_L$ presented in Figure 5.2 are due in part to incorrect specification of the face rotation motor, which was unable to provide sufficient torque to rotate the face whilst also keeping adequate face pressure. The $V_L$ values recorded in this series of tests are significantly greater than those experienced in the field (§ 2.3.2.2), but are comparable to those recorded in other experimental tests, for example Lee et al. (2006a), in which volume losses of up to 120% were realised. There is also evidence in the literature of deeper tunnels experiencing failure at greater values of $V_L$ (Marshall, 2009).
The ratios of $\frac{\text{V}_{\text{st.dev}}}{\text{V}_{\text{avg}}}$ presented in Table 5.3 show a significant reduction after test TBM001, and continue to decrease until test TBM006. This is a similar trend to the $\frac{\text{Q}_{\text{st.dev}}}{\text{Q}_{\text{avg}}}$ values presented in Table 5.2, and is also explained by consideration of the increasing familiarity of the author with the operation of the MTBM. The increase in the ratio for test TBM006 is once again due to the mechanical problem discussed above.

5.3 Face pressure

5.3.1 Introduction

The configuration which allowed the measurement of the pressure acting on the face of the MTBM was discussed in § 3.4.3.4. In short, it consisted of allowing a very small displacement of the face rotation motor when a force was imposed at the face, which caused an increase in the load acting on a load cell placed between the motor and a reaction plate (see Figure 3.12). The purpose of measuring the pressure acting on the cutting face was to ensure that adequate support was provided to the excavated face as the MTBM progressed through the sand mass.

A careful balance between the positions of the cutting face and the bar for lining forward advancement (see Figure 3.11) was necessary to obtain accurate face pressure results. The success of the set-up was evaluated after the sand pouring process had been completed by comparing the measured with the theoretical face pressure. The initial start point of the cutting face of the MTBM prior to the first drive was just inside (approximately 10mm) the front face of the test chamber. As the sand level within the test chamber increased above the height of the cutting face, it was possible to calculate the theoretical face pressure using the density of the soil mass, the height of sand, the diameter of the cutting face and Jaky's Law (Eqn. 6.6). Despite the difficulties in configuring the MTBM accurately so that the appropriate movement at the face was achieved, useful results were obtained using this method.

From the results of preliminary tests, a considerable amount of scatter was evident in the face pressure measured during MTBM drives. A very small degree of movement in the connection between the cutting face rotation shaft and the cutting face rotation motor was highlighted as the cause of the problem. This movement was of the order of 0.30mm perpendicular to the tunnel axis and of 0.20mm parallel to the tunnel axis, and although
steps taken prior to the tests proper succeeded in reducing it to some extent, they did not eliminate it altogether.

5.3.2 Presentation of results

The measured face pressures after sand pouring was completed, as well as the difference from the theoretical pressure at the centre of the face, are presented in Table 5.4 for tests TBM002-TBM006. The theoretical pressure was calculated from the vertical pressure using Jaky’s Law (Eqn. 6.6), which itself was calculated from the average weight of the placed sand and the depth from the surface to the centre of the face.

<table>
<thead>
<tr>
<th>Test name</th>
<th>Measured final pressure [kPa]</th>
<th>Difference from theoretical pressure [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>TBM002</td>
<td>5.42</td>
<td>-6.2</td>
</tr>
<tr>
<td>TBM003</td>
<td>5.60</td>
<td>-16.1</td>
</tr>
<tr>
<td>TBM004</td>
<td>1.07</td>
<td>78.9</td>
</tr>
<tr>
<td>TBM005</td>
<td>4.28</td>
<td>16.2</td>
</tr>
<tr>
<td>TBM006</td>
<td>1.68</td>
<td>67.2</td>
</tr>
</tbody>
</table>

Table 5.4 Details of face pressure after sand pouring during tests TBM002-TBM006

Figure 5.3 illustrates the average face pressure measured during each MTBM drive during tests TBM002, TBM003 and TBM005, and also includes the initial theoretical face pressure.
5.3.3 Discussion of results

The difficulty in configuring the set-up precisely is evident from the data in Table 5.4, as it is clear that the theoretical and measured face pressures differ significantly for two out of the five tests (tests TBM004 and TBM006). As stated in § 5.1, this occurred due to incorrect configuration of the cutting face and the bar for lining forward advancement. These tests are therefore omitted from the remainder of § 5.3. The comparison may be considered acceptable for tests TBM003 and TBM005, and is good for test TBM002.

5.4 Surface settlements

5.4.1 Introduction

The surface settlements in the MTBM tests were measured using linear variable differential transformers (LVDTs). The positions at which these were placed are presented in Table 3.10. In summary, the settlement was measured at three positions along the tunnel axis (i.e. directly above the tunnel axis) during tests TBM001-TBM004, and at three positions in the transverse direction to the tunnel axis in tests TBM005 and TBM006.

5.4.2 Transverse surface settlement profile

5.4.2.1 Presentation of results

The transverse surface settlement troughs obtained from tests TBM005 and TBM006 are presented in Figure 5.4 and Figure 5.5, respectively. The various series show the settlement trough as the MTBM moves towards (positive integers) and past (negative integers) the plane of measurement. The plane of measurement in this case refers to the vertical plane perpendicular to the MTBM face in which the LVDTs were placed. The first series in each of these figures corresponds to the position of the MTBM prior to any tunnelling occurring. Each subsequent series corresponds to the distance from the MTBM after each drive. It should be noted that Figure 5.4 and Figure 5.5 are presented in this section with regard to the transverse trough width parameters only. An examination of the longitudinal settlement trough is presented in § 5.4.3.
5.4.2.2 Discussion of results

As discussed in Chapter 2, a Gaussian distribution is used to approximate the settlement trough above and ahead of a tunnel constructed in a “greenfield” environment (Martos 1958, Peck 1969). The Gaussian distribution was fitted to the settlement data presented in Figure 5.4 and Figure 5.5 by a non-linear least squares regression analysis using the computer software Matlab, as carried out by Farrell (2010). This method determines the best fit parameters for a specific equation fitted to that data (in this case Eqn. 2.1), based on those which give the highest coefficient of determination ($R^2$). This value indicates the quality of the equation’s fit to the data, where 1 indicates a perfect fit. In this way, the
value of the point of inflection (i) was obtained, which could then be used to obtain the surface trough width parameter $K_s$ using Eqn. 2.6, which in turn could be used to calculate the volume loss using Eqn. 2.4.

The settlement trough width parameters calculated from the data recorded in tests TBM005 and TBM006 and presented in Figure 5.4 and Figure 5.5 respectively are presented in Table 5.5. The variation of $K_s$ with $V_L$ is illustrated in Figure 5.6.

<table>
<thead>
<tr>
<th>Test name</th>
<th>Distance from plane of measurement [$D_i$]</th>
<th>Surface trough width parameter, $K_s$ [-]</th>
<th>Point of inflection, i [mm]</th>
<th>Volume loss, $V_L$ [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>TBM005</td>
<td>4.3</td>
<td>0.47</td>
<td>403.1</td>
<td>12.3</td>
</tr>
<tr>
<td></td>
<td>3.0</td>
<td>0.46</td>
<td>393.7</td>
<td>34.6</td>
</tr>
<tr>
<td></td>
<td>1.9</td>
<td>0.43</td>
<td>363.0</td>
<td>49.3</td>
</tr>
<tr>
<td></td>
<td>0.8</td>
<td>0.41</td>
<td>351.0</td>
<td>65.8</td>
</tr>
<tr>
<td></td>
<td>-0.4</td>
<td>0.40</td>
<td>342.5</td>
<td>86.4</td>
</tr>
<tr>
<td></td>
<td>-1.6</td>
<td>0.38</td>
<td>326.2</td>
<td>105.2</td>
</tr>
<tr>
<td></td>
<td>-2.6</td>
<td>0.37</td>
<td>313.4</td>
<td>119.2</td>
</tr>
<tr>
<td></td>
<td>-3.1</td>
<td>0.34</td>
<td>289.5</td>
<td>126.3</td>
</tr>
<tr>
<td>TBM006</td>
<td>4.4</td>
<td>0.42</td>
<td>357.5</td>
<td>20.2</td>
</tr>
<tr>
<td></td>
<td>3.2</td>
<td>0.41</td>
<td>348.5</td>
<td>52.3</td>
</tr>
<tr>
<td></td>
<td>2.0</td>
<td>0.40</td>
<td>338.5</td>
<td>70.7</td>
</tr>
<tr>
<td></td>
<td>0.9</td>
<td>0.41</td>
<td>352.9</td>
<td>90.8</td>
</tr>
<tr>
<td></td>
<td>-0.3</td>
<td>0.39</td>
<td>334.0</td>
<td>113.9</td>
</tr>
<tr>
<td></td>
<td>-1.0</td>
<td>0.37</td>
<td>312.3</td>
<td>136.7</td>
</tr>
<tr>
<td></td>
<td>-1.1</td>
<td>0.35</td>
<td>301.5</td>
<td>139.2</td>
</tr>
</tbody>
</table>

Table 5.5 Surface trough width parameters obtained from tests TBM005 and TBM006
Various methods of calculating the trough width parameter have been presented in § 2.3.2.3. O'Reilly and New (1982), based on a survey of tunnelling data from the UK, suggested a $K_s$ value of 0.25 for tunnels constructed in sands and gravels with depths to axis level in the range of 6-10m. Mair and Taylor (1997) identified from a review of tunnelling data in the literature that the $K_s$ values obtained from tunnels constructed in sands exhibited a greater amount of scatter than those constructed in clays, and found that the value for tunnels in sands lay in the range of 0.25-0.45, with a mean of 0.35.

The $K_s$ values presented in Table 5.5 lie approximately within the range suggested by Mair and Taylor (1997). Although the resulting volume losses are significantly higher than those experienced in practice, useful trends may still be deduced from the data. The $R^2$ values obtained from the least squares analysis were very high (>0.98), indicating that the fit of the data to the Gaussian distribution was good.

There is reasonable agreement between the values obtained from the two tests presented in Figure 5.6, which shows the approximately linear trend of reducing $K_s$ with increasing volume loss. This pattern has been described in the literature, for example by Vorster (2005) and Marshall (2009). Farrell (2010) identified a non-linear decrease with increasing volume loss for $V_i < 2\%$, followed by a linear decrease for $V_i > 2\%$. This reduction in $K_s$ signifies the development of the chimney failure mechanism, which was identified from centrifuge tests in sands by Chambon and Corte (1994) and is discussed in § 2.3.4. In summary, the term “chimney failure mechanism” refers to the failure type observed above
tunnels constructed in sands, where there is a narrow “chimney” which propagates vertically, in contrast to the much wider failure mechanism observed for tunnels constructed in clays (Fig. 2.7).

Although not relevant for many of the methods of modelling tunnel construction discussed in § 2.5.3.1, the possibility of the development of a non-symmetrical settlement trough was identified as a risk for the method adopted in this series of tests due to the rotation of the cutting face. The LVDT configuration adopted in test TBM006 allowed the investigation of this phenomenon. It was concluded, however, from the transverse settlement troughs recorded in that test which are presented in Figure 5.5, that there is a high degree of symmetry about the tunnel centreline and that it is therefore unnecessary to consider this effect further.

5.4.3 Longitudinal surface settlement profile

The purpose of this section is to present and discuss the results obtained from tests TBM002-TBM004 regarding the longitudinal surface settlement profile.

5.4.3.1 Presentation of results

The measured vertical settlements from tests TBM002-TBM004 are presented in Figure 5.7. These curves were fitted with a third order polynomial, the derivatives of which were then found. This allowed the variation in vertical surface settlement as the MTBM face progressed towards and past the point of measurement to be compared. These data are presented in Figure 5.8.

5.4.3.2 Discussion of results

The differences in the magnitudes of the maximum surface settlements recorded by the LVDTs during each of the tests were consistent throughout tests TBM002-TBM004: there is a pattern of larger settlements being recorded by the LVDT closest to the rear face of the test chamber, as shown in Figure 5.7. Although this may be related to the different rates of excavation as the tunnel progresses (as discussed in § 5.2), it is more likely that the boundary conditions were influencing the surface settlements to some extent.
Figure 5.7 Measured vertical settlements: (a) test TBIM002 (b) test TBIM003 (c) test TBIM004
Figure 5.8 Slope of vertical settlement profiles from tests TBM002-TBM004 from LVDTs at (a) front (b) middle (c) back
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The influence of the boundary conditions is also evident from the slopes of the troughs presented in Figure 5.8, in which it is illustrated that the behaviour of the middle LVDT lies between that of the front and back LVDTs. This suggests that as the distance from the front face of the test chamber increases, the influence of the boundary conditions on the surface settlement decreases. This may be related to the considerable depth of the tunnel, as it has a $C/D_f$ ratio of 9.2, where $C$ is the distance from the soil surface to the crown of the tunnel. Finally, Figure 5.8 also shows the relatively good agreement between the behaviour of the front, middle and back LVDTs from each of the three tests.

In addition to the distance from the point of measurement to the front face of the test chamber, the distance from the tunnel crown to the horizontal plane in question is also a function of the extent to which the boundary conditions will affect the settlement. For example, on the initial drive of the MTBM, the settlements in close proximity to the tunnel crown will not be influenced by the boundary conditions, but those directly above the tunnel at the soil surface will as they will have been unable to develop in the same way. This is related to the deep position of the tunnel in these tests ($C/D_f=9.2$).

The results presented in Figure 5.7 illustrate that none of the LVDTs recorded a constant vertical surface settlement after the MTBM had passed the point of measurement. This is true even for the final data recorded by the front LVDT, despite the fact that the MTBM and end of the tapered section of the MTBM lining had progressed approximately $4D_f$ and $2.9D_f$ past the point of measurement, respectively.

The method of predicting the longitudinal settlement trough due to tunnel construction proposed by Attewell and Woodman (1982) is discussed in § 2.3.3. An integral step in solving the equation proposed in this work is the calculation of the volume of the settlement trough ($V_s$) using the trough width parameters at a transverse profile after the MTBM has progressed a distance of $z_0$ past that plane. This was not possible due to the boundary conditions imposed by the dimensions of the test chamber. Instead, 2.5 times the maximum vertical settlement value recorded by the back LVDT when the face of the MTBM was directly below that point was the value used to calculate $V_s$. This was chosen because Attewell and Woodman (1982) found from a number of case histories (albeit on tunnels constructed in clays) that the average ratio of the settlement directly above the tunnel face to the maximum final settlement was 40%.
A comparison is made in Figure 5.9 of the theoretical longitudinal settlement troughs using this method with those measured in test TBM004. The qualitative results of this test are representative of the other tests in the series. In this figure, the points 0.0m and 0.8m on the horizontal axes represent the front and rear faces of the test chamber, respectively. The results obtained from the theoretical solution are presented in a different form in Figure 5.10, where they are compared with the settlements measured by the LVDT positioned nearest the rear face of the test chamber in tests TBM002-TBM004. This allows a comparison of the full LVDT data set with the theoretical solution.

It is evident from Figure 5.9 that when the tunnel face is at 0.1m (sub-figure ‘a’), the settlement trough is well predicted by the solution of Attewell and Woodman (1982). However, generally speaking, the quality of the prediction obtained from the solution is not acceptable. The absolute values as well as the slope of the measured trough, are very different to the measured data, other than at the position of the LVDT placed nearest the rear face of the test chamber. This may be due to the influence of the boundary conditions on the surface settlements measured using the other two LVDTs, as discussed in § 5.4.3.2. However, when the entire data set from the LVDT placed nearest the rear face of the test chamber is compared to the theoretical solution, as shown in Figure 5.10, it is evident that the solution predicts the settlement reasonably well. Generally speaking, the settlement is under-predicted when the tunnel face is greater than approximately 3D_f from the point of measurement, and over-predicted when the tunnel face is less than approximately 3D_f.

It should be noted that the typical shape of the longitudinal settlement trough, as presented in Figure 2.6, is obtained from the solution of Attewell and Woodman (1982) when the face of the tunnel is at considerable distance from the start point. As the face of the MTBM in this instance is relatively close to the start point, the typical longitudinal settlement trough is not evident in Figure 5.9.
Figure 5.9 Comparison of longitudinal settlement troughs from test TBM004 as MTBM progresses through sand mass with those obtained from the solution of Attewell & Woodman (1982)
Figure 5.10 Comparison of vertical settlements with those obtained from the solution of Attewell & Woodman (1982): (a) test TBM002 (b) test TBM003 (c) test TBM004
5.5 Sub-surface settlements

5.5.1 Introduction

The sub-surface settlements were measured using a standard off-the-shelf digital camera and the Particle Image Velocimetry (PIV) software GeoPIV (White et al., 2003). Further details of this are presented in § 2.6 and § 3.6.2. The theoretical field-of-view of the PIV for each test is presented in Table 5.6 in terms of \(D_f\). Data from the entire field-of-view was not available after PIV analysis, due to erroneous results at the periphery of the image, referred to by White et al. (2003) as “wild vectors”, which have been omitted. Wild vectors may be caused by a number of issues, including scratches on the rear Perspex face of the test chamber.

It should be noted that the field-of-view did not extend to the surface of the sand mass as it was limited in size. This was due to restrictions in the distance from the camera to the rear face of the test chamber imposed by concerns regarding the resolution of the system, based on observations made during the calibration tests (as discussed in § 3.6.2). The plane of measurement, which is referred to throughout § 5.5 with regard to PIV, is the rear face of the test chamber from which the camera captured the images of the sand mass.

<table>
<thead>
<tr>
<th>Test name</th>
<th>TBM001</th>
<th>TBM002</th>
<th>TBM003</th>
<th>TBM004</th>
<th>TBM005</th>
<th>TBM006</th>
</tr>
</thead>
<tbody>
<tr>
<td>Above crown</td>
<td>5.3</td>
<td>6.2</td>
<td>6.5</td>
<td>6.1</td>
<td>5.6</td>
<td>5.8</td>
</tr>
<tr>
<td>Below crown</td>
<td>1</td>
<td>1.9</td>
<td>2.2</td>
<td>1.8</td>
<td>1.8</td>
<td>2.6</td>
</tr>
<tr>
<td>LHS of tunnel CL</td>
<td>2.5</td>
<td>3</td>
<td>3.2</td>
<td>3.9</td>
<td>3.5</td>
<td>2.6</td>
</tr>
<tr>
<td>RHS of tunnel CL</td>
<td>3</td>
<td>4.2</td>
<td>3.1</td>
<td>0.9</td>
<td>1.0</td>
<td>2.5</td>
</tr>
</tbody>
</table>

Table 5.6 Extent of PIV field-of-view for MTBM tests, expressed as a multiple of \(D_f\)

5.5.2 Transverse sub-surface settlement trough

5.5.2.1 Presentation of results

Typical results obtained from the PIV analyses are presented in Figure 5.11, which shows the vertical settlement at various distances above the tunnel crown when the face of the MTBM is 0.5\(D_f\) behind the plane of measurement (i.e. the face of the MTBM has not yet reached the plane of measurement), as measured in test TBM002.
Figure 5.11 Vertical settlements from tests TBM002 at various distances above tunnel crown when MTBM face is 0.5D, from plane of measurement

5.5.2.2 Discussion of results

General behaviour:

It is worth noting from Figure 5.11 that, from the rear face of the test chamber, the magnitude of the settlement increases as the vertical distance above the crown increases. This is due to the development of the longitudinal settlement trough ahead of the advancing MTBM, as shown in Figure 2.6. This is in contrast with the settlements observed when tunnel construction simulation methods are used which do not account for the progression of the face of the tunnel, for example, the methods adopted by Chambon and Corte (1994) and Atkinson and Potts (1977), in which the maximum settlements are observed at the tunnel crown. An example of the settlements recorded using the latter method is shown in Figure 5.12, from Farrell (2010). This is equivalent to the results obtained from this series of tests when the cutting face of the MTBM is at the rear face of the test chamber. The surface and sub-surface longitudinal settlement troughs measured during this test series are discussed further in § 5.4.3 and § 5.5.3, respectively.

It proved very difficult to quantify the effect of various excavation rates during drives on the sub-surface settlements measured using PIV. This highlights the added complexities involved in the modelling of the advancement of the MTBM over those methods which do not account for this.
Assessment of suitability of Gaussian distribution:

The use of the Gaussian distribution (Martos 1958, Peck 1969) to approximate the settlement due to the construction of a tunnel in a “greenfield” environment have been discussed in Chapter 2 and in § 5.4.2.2. The purpose of this section is to investigate the suitability of the Gaussian distribution to describe the vertical settlement. Correspondingly, the Gaussian distribution was fitted to the transverse sub-surface settlement data using the non-linear least squares regression analysis previously discussed in § 5.4.2.2 (i.e. by matching Eqn. 2.1 to find the point of inflection \( i \), which can then be used to find the sub-surface trough width parameter, \( K_{ss} \)).

Vorster (2005) reported that the Gaussian method underestimates the settlement at the shoulders of the trough. Although analysing the \( R^2 \) values obtained from the regression analysis gives an indication of the quality of the match of the Gaussian solution to the measured data, it does not specifically give an indication of the match at the trough shoulders. The suitability of applying the Gaussian distribution to the measured data was therefore assessed in two ways: by a visual comparison of the shoulders of the measured and calculated settlement troughs, and by analysing the \( R^2 \) values obtained from the non-linear least squares regression analysis. The results of tests TBM004 and TBM005 were used to assess the suitability of the Gaussian distribution because the lateral extent of the camera’s field-of-view was greatest in these tests (as shown in Table 5.6), and therefore provided more information at the shoulders of the trough than the other tests where the field-of-view was concentrated more so on the central part of the trough.

A comparison between the settlement trough measured in test TBM005 at various heights above the tunnel crown when the MTBM is 0.6\( D_f \) from the plane of measurement and the
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calculated trough is made in Figure 5.13. A summary of the $R^2$ values from the same test is presented in Table 5.7. These data are typical of tests TBM004 and TBM005. There is a general pattern of increasing $R^2$ values as the MTBM face progresses towards the plane of measurement. This is due to the development of a very flat settlement trough at low volume losses when the MTBM face is at a considerable distance from the plane of measurement (when displacements are very small), which is not well approximated by either the Gaussian distribution or the solution of Vorster (2005).

![Figure 5.13 Measured and calculated settlement troughs from test TBM005 when MTBM is 0.6Df from plane of measurement at various heights from tunnel crown: (a) 1Df (b) 2Df (c) 3Df (d) 4Df](image)

Figure 5.13 Measured and calculated settlement troughs from test TBM005 when MTBM is 0.6Df from plane of measurement at various heights from tunnel crown: (a) 1Df (b) 2Df (c) 3Df (d) 4Df
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### Table 5.7 Coefficients of determination from least squares regression analysis of test TBM005, where positions are expressed as multiples of $D_f$

<table>
<thead>
<tr>
<th>Distance from plane of measurement [$D_f$]</th>
<th>Height above tunnel crown [$D_f$]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
<td>6.7</td>
<td>0.905</td>
</tr>
<tr>
<td>5.6</td>
<td>0.884</td>
</tr>
<tr>
<td>4.5</td>
<td>0.950</td>
</tr>
<tr>
<td>3.3</td>
<td>0.951</td>
</tr>
<tr>
<td>2.2</td>
<td>0.960</td>
</tr>
<tr>
<td>1.1</td>
<td>0.962</td>
</tr>
<tr>
<td>0.6</td>
<td>0.970</td>
</tr>
</tbody>
</table>

It is evident from Figure 5.13 that the Gaussian distribution matches the measured data reasonably well. Although the settlement at the shoulders of the trough was underestimated by the Gaussian distribution at 1$D_f$ above the crown, this difference reduced as the distance from the tunnel crown increased, so that it was small at 2$D_f$ and practically insignificant at 3$D_f$ and 4$D_f$. The ability of the Gaussian distribution to match the measured data is evident from the relatively high values of $R^2$ in Table 5.7.

It can be concluded from the results presented above that the Gaussian distribution is a suitable method of modelling the transverse settlement trough resulting from tunnel construction, and will be used for the purposes of this work. The solution matched the data reasonably well even as the chimney failure mechanism is developing (as observed in § 5.4.2.2), which would increase the likelihood of under-prediction at the shoulders. It is therefore unnecessary to use the solution of Vorster (2005).

**Variation of trough width parameter with depth:**

The importance of the trough width parameter has been discussed in § 2.3.2.3. The variation of this parameter with depth is of considerable importance when assessing the impact of tunnel construction on adjacent geotechnical structures at various depths from the soil surface, such as foundations, services and existing tunnels. The various methods of calculating $K_{ps}$ with respect to depth which were presented in § 2.3.2.3 include the solution of Mair *et al.* (1993), which was adapted by Jacobsz (2002) to take into account the lower
values of the trough width parameter observed at the soil surface during centrifuge tests in sand.

Figure 5.14 illustrates the trend from test TBM005 of increasing $K_{ss}$ with depth as the MTBM face progresses towards the plane of measurement. The left hand side vertical axis represents the distance from the ground surface ($z$) normalised by the distance to the tunnel centreline ($z_0$). The data has been presented in this way to match the format in the literature, in order to allow a comparison. However, the $K_{ss}$ value with respect to the distance above the tunnel crown is also shown using the secondary vertical axis on the right hand side of the figure. This figure illustrates how the magnitude of $K_{ss}$, and correspondingly the width of the settlement trough, increases non-linearly as the distance from the ground surface to the tunnel increases. It also illustrates how the trough width varies with $V_L$, as the advancement of the MTBM is associated with an increase in the settlements. However, the purpose of this section is to discuss the variation with depth. The variation with $V_L$ is discussed in the following section.

A comparison between the measured $K_s$ value from test TBM005, the $K_{ss}$ values at various depths from the soil surface from tests TBM002-TBM005 and the solutions of Mair et al. (1993) and Jacobsz (2002) is illustrated in Figure 5.15. It has been described above how $K_{ss}$ varies with volume loss. As the aforementioned solutions do not take this into account, the data presented in this figure are average values of the $K_{ss}$ values at the respective height as the MTBM progressed towards the plane of measurement. For example, the $K_{ss}$
value at \( z/z_0 = 0.9 \) for TBM005 in Figure 5.15 is the average of the values at the same depth in Figure 5.14. The data show some outlying points, for example at \( z/z_0 = 0.7 \) for test TBM002. These are as a result of wild vectors (in part due to scratches on the Perspex rear face of the test chamber) and demonstrate that although the results obtained from PIV analyses are very useful, there are some limitations. Despite this, it can be concluded from the figure that the measured data follows a similar trend to those suggested by the theoretical solutions. At the surface, the calculated \( K_s \) value lies between that obtained from the two solutions. At depths of up to \( z/z_0 = 0.4 \), the two solutions provide very similar values. For \( z/z_0 \) values greater than 0.4, the solution of Jacobsz (2002) tends to overestimate \( K_{ss} \), whereas the solution of Mair et al. (1993) provides a very reasonable prediction.

![Graph](image)

**Figure 5.15 Comparison of measured and calculated variation of \( K \) with depth**

Variation of sub-surface trough width parameter with volume loss:

The variation of \( K_{ss} \) with volume loss was identified briefly in the preceding section. The purpose of this section is to discuss this phenomenon in more detail. The relationship between \( K_{ss} \) and the volume loss of the corresponding settlement trough at various heights above the tunnel crown after each MTBM drive is illustrated in Figure 5.16. \( V_L \) was calculated using Eqn. 2.4.

It is evident from this figure that the magnitude of \( K_{ss} \) is very much dependent on \( V_L \) – there is an approximately linear and different relationship between the two parameters for each height above the tunnel.
The reduction in the value with $V_L$ and with height above the tunnel is due to the development of the chimney failure mechanism. A similar relationship was observed between $K_s$ and $V_L$, as discussed in § 5.4.2.2. This highlights the fact that the solutions of Mair et al. (1993) and Jacobsz (2002) do not account for the variation of both $K_s$ and $K_{ss}$ with $V_L$.

5.5.3 Longitudinal sub-surface settlement trough

5.5.3.1 Presentation of results

This section presents some of the sub-surface settlement results obtained from the PIV analysis. Typical sub-surface settlements results obtained are shown in Figure 5.17, which show the settlements occurring as the tunnel face progresses towards the plane of measurement in test TBM002. The Matlab script which was used to produce these plots was based on one used by Farrell (2010). Sub-figures ‘a’ to ‘h’ show the displacements after MTBM drives one to eight, respectively. The horizontal and vertical axes represent the distances from the tunnel centreline and tunnel crown, respectively, which have been normalised by $D_f$. It should be noted that the scale of the contour colours in many of the sub-figures is different, and therefore a direct comparison based on colour is not possible. The vertical settlements from the same test at various distances from the tunnel crown are presented in Figure 5.18. The troughs at distances of 1-6$D_f$ were obtained from PIV analyses, while the surface trough was obtained from the LVDT placed closest to the rear face of the test chamber (discussed in § 5.4.3).
Figure 5.17 Settlements (in mm) from test TBM002 at various distances from plane of measurement:
(a) 7.9D_t (b) 6.8D_t (c) 5.6D_t (d) 4.5D_t (e) 3.4D_t (f) 2.3D_t (g) 1.1D_t (h) 0.5D_t
5.5.3.2 Discussion of results

From Figure 5.17 it is evident that as the tunnel face progressed towards the plane of measurement, the most significant settlements occurred at the top of the PIV field-of-view, i.e. at approximately $6D_f$ from the crown, as discussed in § 5.5.2.2. There was insignificant variability in the transverse direction and little settlement at the tunnel level whilst the face was up to $4.5D_f$ from the plane of measurement, as shown in sub-figures ‘a’ to ‘d’. It is not until the tunnel face was $3.4D_f$ (i.e. after drive 5) from the plane of measurement that concentration of the maximum settlements above the tunnel centreline is evident. It is clear from sub-figures ‘f’, ‘g’ and ‘h’ that as MTBM advancement continued, the zone of maximum settlement increased in size and extended from the top of the field-of-view towards the tunnel crown. The maximum settlements in the transverse direction were concentrated above the tunnel centreline.

The stationary points within the zones of settlement, which are evident in these sub-figures (particularly in sub-figures ‘f’ to ‘h’), are due to the grid of control markers (discussed in § 2.6.1) and should therefore be ignored. The small zone of higher settlement visible in the bottom left corner of the field-of-view in some of the sub-figures is due to wild vectors (discussed in § 5.5.1), and should also be ignored.

The behaviour described above is also evident from Figure 5.18. Significant settlements were evident at the surface and at $6D_f$ above the tunnel crown as soon as MTBM
progression began. Soil planes closer to the tunnel crown were not subjected to large displacements until the MTBM was much closer to the plane of measurement. For example, significant displacements began to occur at 4D_f from the tunnel crown when the face was 3D_f away, while significant displacements occurred at 1D_f from the tunnel crown when the face was only approximately 1.5D_f away. The surface settlement measured using the LVDT corresponds reasonably well with the PIV data. For example, the surface settlement (as measured by the LVDT) is correspondingly larger than that at 6D_f from the crown (as obtained from PIV) at most points. However, an exception to that is when the face is within 2D_f from the plane of measurement. This may suggest that the LVDT is affected by the boundary conditions imposed by the dimensions of the test chamber.

5.6 Soil movements obtained from settlement plates

5.6.1 Introduction

The use of slide potentiometers and settlement plates in the MTBM test series to measure the vertical soil displacements throughout the sand mass was discussed in § 3.6.3. In summary, these instruments were attached by connector rods to stainless steel settlement plates, which were positioned throughout the sand mass. The movement of the potentiometer arm caused by the movement of the plate resulted in a change in the voltage output which was converted to a settlement using a calibration factor.

5.6.2 Presentation of results

The vertical settlement results obtained from the potentiometers were assessed by comparison with those obtained from PIV. An example of the varying quality of results is shown in Figure 5.19. Sub-figures ‘a’ and ‘b’ compare the results of PIV with the settlement recorded by a settlement plate at 2D_f and 1D_f above the tunnel crown, respectively, as recorded in test TBM002. The eight PIV data points in each series refer to the settlement after each MTBM drive.
5.6.3 Discussion of results

Despite the success of the potentiometers during the calibration tests (§ 3.6.3.1) in measuring the vertical displacement, the results obtained in the full scale tests involving the MTBM were inconsistent. The data from the settlement plate placed at 2D_f from the tunnel crown match those obtained from PIV analyses very well (Figure 5.19 sub-figure ‘a’). Conversely, the data from the settlement plate placed at 1D_f do not match the PIV data, other than the last PIV data point, which corresponds to the displacement after the last MTBM drive (Figure 5.19 sub-figure ‘b’). It should be noted that the positions of these settlement plates are only presented for completeness – there is no apparent relationship
between the quality of the match of the potentiometer data to the PIV data and the position at which they were placed.

Although there was significant variability in the results obtained from the potentiometers, it was still possible to calculate trough width parameters. By comparison of the movements recorded by settlement plates at the same distance from the tunnel crown, it was possible to remove erroneous data, and Eqn. 2.1 was then applied to the remaining data points using least squares analysis. The transverse sub-surface settlement trough parameters obtained in this manner showed approximately the same trends as those obtained from PIV analyses. However, the $R^2$ values obtained from the least squares analysis carried out to match the Gaussian distribution to the potentiometer data were low. This is a reflection of the variability in the data, which is exacerbated by the few data points available at a soil plane. For example, the maximum number of settlement plates at $1D_f$ and $2D_f$ from the tunnel crown was five.

An advantage of the use of potentiometers over PIV is that they allowed the behaviour of the sand mass after the passing of the MTBM to be observed. Despite the quantitative inconsistencies in the results, some useful trends may be observed from the qualitative results of the tests. The settlement plates at heights of $2D_f$ and $4D_f$ above the tunnel crown were placed at distances of $3.7D_f$ and $4.5D_f$ from the front face of the test chamber, respectively. This allowed the settlement behaviour at these planes to be observed after the MTBM had progressed up to $5.4D_f$ and $4.6D_f$ past them, respectively. The settlement at these points as recorded during test TBM002 is presented in Figure 5.20. From this figure, it is evident that approximately constant vertical settlements only developed after the MTBM face had progressed $4.2D_f$ from the point of measurement. At this point, the rear of the tapered section of the lining was $3.1D_f$ from the point of measurement. Constant vertical settlements appeared to be developing for the plate at a height of $4D_f$ after the MTBM had progressed $4.2D_f$ from the point of measurement. However, there was a lag in the development of constant vertical settlements as the height from the tunnel crown increased. This may explain why constant vertical settlements were not measured by the LVDT placed nearest the front face of the test chamber, as shown in Figure 5.7.
It is considered that the potentiometer installation process, which is discussed in § 3.7.2, is the principal cause of the inconsistent sub-surface settlement results obtained from the potentiometers and settlement plates. The transfer of the connector rod from the wooden dowels to the potentiometers and the attempt to develop the appropriate tensile force prior to testing in order to avoid the loss of the first 0.2mm of movement (as experienced in the calibration tests, discussed in § 3.6.2.3) may have caused disturbances in the soil mass. Although these disturbances influenced the settlement measured by the potentiometer, the effect of this on the overall behaviour of the sand mass as a whole is believed to be negligible.

The purpose of using potentiometers in the test series was to supplement the vertical sub-surface settlement data obtained from the PIV analyses. While the results obtained from the potentiometers were inconsistent, the results of the PIV analyses which were presented above in § 5.5 were very useful in describing and understanding the behaviour of the soil mass due to the progression of the MTBM. Therefore, despite the difficulties in the use of the potentiometers, there is sufficient information from the PIV analyses to describe the settlement behaviour of the sand.
5.7 Arching effect due to tunnel construction

5.7.1 Introduction

An array of miniature EPCs placed within the sand mass in a vertical plane situated 475mm from the front face of the test chamber (i.e. the plane of measurement for the purposes of this section) was used to quantify the arching effect resulting from the progression of the MTBM through the sand mass. Technical details of the cells and their calibration are provided in § 3.6.1.1 and § 3.6.1.3, respectively. The positions at which these were placed in the test series are illustrated in Figure 3.40 and presented in Tables 3.7 and 3.8. The results obtained from this instrumentation describe the change in the vertical stress acting in the soil mass, which allows the development of a more detailed understanding of the arching effect.

The magnitude of change of the vertical stress is expressed through the use of the vertical stress ratio, which is defined as the ratio of the vertical stress at a point to the initial vertical stress prior to any MTBM advancement \((\sigma_v/\sigma_{v,i})\). The maximum, minimum and final vertical stress ratios \((\sigma_v/\sigma_{v,i})_{\text{max}}\), \((\sigma_v/\sigma_{v,i})_{\text{min}}\) and \((\sigma_v/\sigma_{v,i})_{f}\), respectively) are used throughout this section to describe the behaviour of the soil due to arching. The normalised distances from the tunnel face to the plane of measurement that correspond to the maximum, minimum and final vertical stress ratios are \(y_{\text{max,norm}}\), \(y_{\text{min,norm}}\) and \(y_{f,norm}\), respectively. The maximum vertical stress ratio refers to the maximum value of the vertical stress from prior to the first drive until it has just passed the plane of measurement. These variables are defined in Figure 5.21.

It was acknowledged in Chapter Four that although the cells provide very useful results, there are some difficulties inherent in their use which may be related to placement effects. Accordingly, this should be kept in mind when interpreting the results presented below.
5.7.2 Behaviour directly above tunnel centreline

5.7.2.1 Presentation of results

Typical results of the miniature EPCs placed directly above the tunnel centreline from the test series are illustrated in Figure 5.22, which shows the vertical stress ratio from test TBM002 at various distances above the tunnel crown as the MTBM progresses towards (positive integers) and past (negative integers) the plane of measurement. The primary points from this figure are presented in Table 5.8 below.
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Table 5.8 Summary of vertical stress ratios and corresponding settlements from test TBM002
(distances above tunnel crown expressed as multiples of $D_f$)

<table>
<thead>
<tr>
<th>Distance above tunnel crown [$D_f$]</th>
<th>$(\sigma_v/\sigma_{v,i})_{\text{max}}$</th>
<th>$y_{\text{max,norm}}$</th>
<th>$(\sigma_v/\sigma_{v,i})_{\text{min}}$</th>
<th>$y_{\text{min,norm}}$</th>
<th>$(\sigma_v/\sigma_{v,i})_f$</th>
<th>$y_{f,norm}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.4</td>
<td>1.103</td>
<td>2.50</td>
<td>0.078</td>
<td>-1.03</td>
<td>0.498</td>
<td>-3.24</td>
</tr>
<tr>
<td>3.2</td>
<td>1.176</td>
<td>1.94</td>
<td>0.353</td>
<td>-1.44</td>
<td>0.574</td>
<td></td>
</tr>
<tr>
<td>4.1</td>
<td>1.073</td>
<td>3.16</td>
<td>0.462</td>
<td>-1.46</td>
<td>0.815</td>
<td></td>
</tr>
<tr>
<td>5.5</td>
<td>1.068</td>
<td>4.07</td>
<td>0.635</td>
<td>-1.42</td>
<td>0.906</td>
<td></td>
</tr>
</tbody>
</table>

The results obtained from the EPCs positioned at $D_f$ and $3D_f$ from the tunnel crown in tests TBM003 and TBM004 are illustrated in Figure 5.23 and Figure 5.24, respectively. These data are compared to allow the consistency of the results to be examined. It is not suggested that results from all tests should be identical, because the settlements varied from test to test. It is expected, however, that common trends will be evident between tests.

Figure 5.23 Comparison of arching effect at $1D_f$ above tunnel crown from tests TBM003 and TBM004
Figure 5.24 Comparison of arching effect at 3Df above tunnel crown from tests TBM003 and TBM004

5.7.2.2 Discussion of results

The patterns evident in Figure 5.22 and Table 5.8 above may be explained by consideration of the progression of the longitudinal settlement trough resulting from the movement of the MTBM through the sand mass. The miniature EPC placed at 2.4Df above the tunnel crown is initially used to explain this behaviour.

As the MTBM began to move from the front face of the test chamber towards the rear, the advancing settling soil mass transferred stress via shear planes which developed within the soil to the soil mass directly ahead of and beside it. This is evident in Figure 5.22 from the increase in the vertical stress ratio as the MTBM face moved from its starting position until it was approximately 1.2Df from the plane of measurement. Terzaghi (1943) described arching as the “transfer of pressure from a yielding mass of soil onto adjoining stationary parts”. This is clearly inconsistent with the results presented above for the following reasons: the settlement of the soil body ahead of the advancing MTBM has been presented previously in Figure 5.18, the data in which show that considerable settlement was occurring when the MTBM face was 1.2Df from the plane of measurement. The soil in the vicinity of the miniature EPC was both non-stationary and receiving stress from the adjacent soil mass. Arching must therefore be related to the relative displacement between adjacent soil masses, rather than the transfer of pressure from a yielding to a stationary mass.
As the zone of soil in which the miniature EPC was placed began to settle to a greater extent, its behaviour changed from a positive to a negative arching zone. This is illustrated by the reduction in the vertical stress ratio after the MTBM was closer than approximately 1.2Df from the plane of measurement. The arching effect increased (i.e. the vertical stress ratio decreased) as the MTBM continued to progress through the sand mass and the resulting settlements increased in size. A minimum vertical stress ratio of 0.078 was reached for the EPC placed at 2.4Df when the MTBM face had progressed 1.03Df past the plane of measurement. This corresponds to the point at which approximately 90% of the tapered section of the MTBM had passed, and was followed by a subsequent constant value of vertical stress ratio, until the MTBM face and rear of the tapered section were approximately -1.24Df and -0.10Df from the plane of measurement, respectively.

Following this constant value, the vertical stress ratio then began to increase once more, as the zone of soil in the vicinity of the EPC received stress from the adjacent soil zone above and ahead of the MTBM. The displacement of the soil mass as the MTBM passes was discussed in § 5.6.2, where it was stated that constant vertical settlements only developed when the MTBM and end of the tapered section were -4.2Df and -3.1Df from the point of measurement, respectively. It may therefore be concluded that the soil 2.4Df above the tunnel crown is still settling after the face has progressed 1.36Df from the point of measurement. The magnitude of the settlement of the soil in the vicinity of the miniature EPC after the MTBM and tapered lining have passed was less than that of the soil mass which was above and ahead of the tunnel face and tapered lining at that time, resulting in a transfer of stress from the latter zone to the former.

Soil nearer the crown experienced a greater degree of arching than that which is experienced at greater distances away. This is despite the fact that, as shown from the PIV results presented in Figure 5.17 and Figure 5.18, the soil at lower \( \frac{z}{z_0} \) ratios (i.e. further from the tunnel crown) experienced greater displacements than those which are experienced at higher \( \frac{z}{z_0} \) ratios (i.e. closer to the tunnel crown) as the MTBM approached, and higher arching effects are usually associated with zones of soils that are displacing significantly. This is explained by consideration of the length of the shear planes. As the ratio of \( \frac{z}{z_0} \) decreased, the shear planes were more likely to intersect the soil surface, thereby reducing their length in comparison to those at greater \( \frac{z}{z_0} \) ratios. It is through these shear planes that soil is transferred from one soil mass to the next. Consequently as the shear plane reduced in length, the magnitude of stress which could be transferred along
these reduces correspondingly. This explains why \((\sigma_v/\sigma_{v,1})_{\text{min}}\) was greater for cells placed at lower \(z/z_0\) ratios than those placed at deeper positions within the sand mass.

The EPC placed at 2.4\(D_f\) from the tunnel crown experienced the minimum vertical stress ratio when the MTBM face was -1.14\(D_f\) from the plane of measurement. The other cells all reached the minimum vertical stress ratio at approximately the same time. This corresponded to the MTBM face having travelled 1.4\(D_f\) past the plane of measurement. The reason for this is uncertain, and may be due to the orientation of the shear planes which develop.

The point described above regarding the length of the shear planes also explains the differences in the \((\sigma_v/\sigma_{v,1})_{\text{max}}\) values, which arose as the MTBM face moved from its starting position until it was approximately 1.2\(D_f\) from the plane of measurement, i.e. when the soil around the EPCs were part of a positive arching zone. Greater displacements are normally associated with an increase in the degree of arching. However, as the shear planes at smaller \(z/z_0\) ratios where the settlements were larger were intersected by the soil surface, the degree of arching was in fact less than those experienced at higher \(z/z_0\) ratios, where the amount of settlement occurring was smaller but the shear planes were longer.

Although it cannot be stated conclusively that the vertical stresses at the positions above the tunnel crown presented in Figure 5.22 reached a distinct constant ultimate value, it appears as though a constant value was being approached when MTBM advancement stopped when the face was -3.2\(D_f\) from the plane of measurement. As stated in § 5.6.3, constant vertical settlements only developed when the MTBM was -4.2\(D_f\) from the plane of measurement. However the rate of increase in the settlements was very low when the face was -3.2\(D_f\) away from the EPC, as shown in Figure 5.20. It is the relative movement between soil masses which causes the arching effect to occur; therefore the reduction in the variation of the arching effect is well matched by the corresponding reduction in the rate of settlement.

It is evident from Figure 5.23 and Figure 5.24 that there is reasonable repeatability in the results obtained from the EPCs which were placed directly above the tunnel centreline from different tests. The general behaviour as well as the minimum and final vertical stress ratios all compare well.
5.7.3 Behaviour adjacent to tunnel centreline

5.7.3.1 Presentation of results
The results obtained from test TBM002 are presented in Figure 5.25. This figure illustrates the vertical stress ratio at lateral distances of 1\(D_f\) and 1.5\(D_f\) from the tunnel centreline, at heights of 1.25\(D_f\) and 2.4\(D_f\) above the tunnel crown. The minimum and final vertical stress ratios from this figure are presented in Table 5.9. These results are typical of those obtained from the various tests in the series. The values obtained from the cell at 2.4\(D_f\) directly above the tunnel crown are also presented in this table to allow a comparison.

<table>
<thead>
<tr>
<th>Distance from tunnel centreline [(D_f)]</th>
<th>Distance above tunnel crown [(D_f)]</th>
<th>((\sigma_v/\sigma_{v,0})_{\text{min}}) [-]</th>
<th>((\sigma_v/\sigma_{v,0})_f) [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>2.4</td>
<td>0.078</td>
<td>0.498</td>
</tr>
<tr>
<td>1</td>
<td>1.25</td>
<td>0.300</td>
<td>0.931</td>
</tr>
<tr>
<td>1</td>
<td>2.4</td>
<td>0.382</td>
<td>0.989</td>
</tr>
<tr>
<td>1.5</td>
<td>1.25</td>
<td>0.596</td>
<td>0.987</td>
</tr>
<tr>
<td>1.5</td>
<td>2.4</td>
<td>0.680</td>
<td>0.918</td>
</tr>
</tbody>
</table>

Table 5.9 Effect of distance from tunnel crown and centreline on vertical stress ratio from test TBM002

5.7.3.2 Discussion of results
The vertical stress ratios at lateral distances of 1\(D_f\) and 1.5\(D_f\) from the tunnel centreline are qualitatively very similar to those directly above the tunnel centreline. Although the common trends have been described in § 5.7.2.2 above in detail, they are summarised
briefly below. As the MTBM progressed from its initial position near the front face of the test chamber until it was approximately 1.2Dₚ from the plane of measurement, the advancing longitudinal settlement trough caused an increase in the vertical stress ratio, as stress was transferred from the displacing soil mass to the relatively stationary mass around the EPCs. As the soil in the vicinity of the cells underwent vertical displacement, stress was transferred from this area to the surrounding areas which were undergoing less settlement, thereby resulting in the reduction in the vertical stress. This was then followed by an increase in the vertical stress ratio due to the advancement of the MTBM past the plane of measurement, as the soil mass near the EPCs received stress from other soil masses which are settling to a greater extent.

It is evident from the data presented in Table 5.9 that the magnitude of arching is dependent on the depth below the surface considered. In the same way as for cells placed directly above the tunnel centreline, lower minimum vertical stress ratios are experienced at greater ratios of z/z₀ than at lower z/z₀ ratios. This has been explained in § 5.7.2.2.

Similarly, the minimum vertical stress ratio increases (i.e. the arching effect decreases in magnitude) as the distance from the tunnel centreline increases. This is due to the variation in the degree of settlement in the transverse direction due to the chimney mechanism, which was identified by Chambon and Corte (1994) and is discussed in § 2.3.4. In summary, the chimney mechanism refers to the failure type observed above tunnels constructed in sands, where there is a narrow “chimney” which propagates vertically. The development of this chimney in the current study was identified from the results of PIV analysis as illustrated in Figure 5.14, which shows the reduction in the trough width parameter with volume loss. This mechanism leads to significant variations in the magnitude of the vertical settlement over a relatively short distance from the tunnel centreline. The soil above the tunnel centreline transfers a significant proportion of the stress acting in this zone to adjacent zones which are displacing less, i.e. at 1Dₚ from the tunnel centreline, which is in turn transferring stress to zones further from the centreline.

The final vertical stress ratios presented in Table 5.9 illustrate the extent of the stress transfer to these zones after the MTBM has passed the plane of measurement. A (σᵥ/σᵥ,i)ₚ ratio of approximately 0.5 was reached above the tunnel crown at a height of 2.4Dₚ. The soil zones at distances of 1Dₚ and 1.5Dₚ from the tunnel centreline, on the other hand,
showed a remarkable recovery in vertical stress to almost return to their initial values. The
difference in the behaviour between these zones may be due to the differences in
settlements which they have been subjected to. The soil masses adjacent to the centreline
have been subjected to lower displacements than the soil above the centreline, which may
have yielded. This allows the soil which has been subjected to lower displacements to
realise a greater increase in the vertical stress, resulting in a higher \((\sigma_v/\sigma_{v,i})_f\) value.

5.7.4 General behaviour of instrumented zone

5.7.4.1 Introduction
The results from tests TBM002-TBM005 are compared in this section in order to identify
trends occurring throughout the entire instrumented area of soil. As stated in § 5.7.2.1, the
results from the different tests are not expected to yield identical results, due to the
variation in excavation ratios and soil displacements. However, the collation of the data in
this way allows the behaviour within the zone of arching to be investigated. While trends
similar to those in test TBM002 were observed, the purpose of this section is to develop an
understanding of the arching zone as a whole. Due to the problems encountered during
tests TBM001 and TBM006 which were discussed in § 5.1, only the results from tests
TBM002-TBM005 are presented in this section.

5.7.4.2 Presentation of results
The maximum, minimum and final vertical stress ratios obtained from tests TBM002-
TBM005 as the MTBM moves towards the plane of measurement are presented below in
Figure 5.26. In these plots, the relationship between the stress ratios, the distance from the
tunnel crown and the distance from the tunnel centreline are investigated. The various
series refer to the lateral distance from the tunnel centreline. It should also be noted that as
this is an amalgamation of data from tests TBM002-TBM005, there may be multiple points
at the same distance from the tunnel position, for example, directly above the tunnel at a
distance of 5Df above the tunnel crown.
Figure 5.26 Vertical stress ratios in instrumented zone obtained from tests TBM002-TBM005: (a) maximum, (b) minimum, (c) final.
It should be noted that there is only one data point at a lateral distance of 2Df from the tunnel centreline. The data at a lateral distance of 2Df from the tunnel centreline and at a height of 2Df above the crown from test TBM003 (cell ‘5’) were removed from the data set, due to poor functionality of the EPC placed at this point. The remaining point at this distance is from the cell positioned at 0.5Df below the tunnel crown in test TBM003.

5.7.4.3 Discussion of results
There is considerable scatter in the data illustrated in the sub-figures within Figure 5.26. This is most likely due to a number of reasons, including the differences in excavation ratios and settlements between tests. Hence what follows is a discussion of the general patterns which are evident from these data.

There do not appear to be any significant trends in the relationship between $(\sigma_v/\sigma_{v,i})_{max}$ and either the lateral distance from the tunnel centreline or the vertical distance above the tunnel crown, as presented in Figure 5.26 sub-figure ‘a’. The majority of the maximum vertical stress ratio data points lie in the range of 1.1-1.2. It is interesting to note that significant increases in the vertical stress ratio were observed at and below the tunnel crown level at relatively large distances (2Df-3Df) from the tunnel centreline.

A number of trends may be observed from Figure 5.26 sub-figure ‘b’ above regarding the minimum stress ratio. For miniature EPCs placed directly above the centreline (0Df), there is a general pattern of increasing $(\sigma_v/\sigma_{v,i})_{min}$ values as the distance from the tunnel crown increases, due to the development of shorter shear bands, which was described in § 5.7.2.2. Although this pattern is also observed at 1Df from the tunnel centreline, it appears to lessen as the distance from the tunnel centreline increases. The relationship between the distance from the tunnel crown and the minimum vertical stress ratio is almost vertically linear at distances from the tunnel centreline of 2.5Df and 3Df. It is likely that this is due to the development of the chimney mechanism, which was also used to explain the same effect for test TBM002 above – the soil at these points lie outside the extent of the narrow chimney failure mechanism.

The data presented in Figure 5.26 sub-figure ‘c’ shows an approximate trend of increasing $(\sigma_v/\sigma_{v,i})_f$ as the distance from the tunnel centreline increases. Closer to the tunnel centreline, for example at 0Df and 1Df, there is also an approximate trend of increasing
(σv/σv0)F with the normalised distance from the tunnel centreline. This appears to reduce to a vertically linear relationship at distances of 1.5DF and 2.5DF. At a distance of 3DF from the tunnel centreline, the final vertical stress ratios which were measured at three different levels increase to values approximately equal to one. These data appear to suggest that the soil at this distance from the tunnel centreline is approaching the extent of the arching zone. The results from this test (test TBM003) are therefore analysed in more detail, and are presented in Figure 5.27.

![Figure 5.27 Vertical stress ratios at a lateral distance of 3DF from the tunnel centreline from test TBM003 at various distances from tunnel crown](image)

In the same way as for all data recorded by the miniature EPCs in this test series, the advance of the longitudinal trough through the sand mass causes an initial increase in the vertical stress ratio at 3DF from the tunnel centreline, as shown in Figure 5.27. Following this, the vertical stress ratio decreases to a minimum value which is approximately the same for all heights from the tunnel crown. After the MTBM has advanced a significant distance past the plane of measurement (of the order of 2DF), the ratio subsequently rises above a value of 1.0. Although the magnitudes of the maximum vertical stress ratios in this zone are similar to those nearer to the tunnel, the minimum and final vertical stress ratios suggest that this plane is very near the lateral extent of the arching zone.

### 5.8 Summary of chapter

The results of a series of tests carried out to investigate the settlements and associated arching due to the progression of a MTBM through a sand mass have been presented and discussed. The relationships between a number of the MTBM parameters, including the
excavation rate and advancement rate, and the settlements caused by it have been illustrated.

The properties of both the transverse and longitudinal surface settlement troughs resulting from the advancement of the MTBM through the sand mass have been investigated through the measurements obtained from an array of LVDTs. The surface trough width parameter ($K_s$) values which were calculated from these settlements and presented in this chapter show good agreement with those in the literature. The Gaussian distribution was found to provide a good prediction of the vertical settlements. The trend of reducing $K_s$ as volume loss increased was identified, as the values ranged from approximately 0.45 at lower volume losses to 0.35 at higher values. It was highlighted that this was due to the development of the chimney failure mechanism. The slope and magnitude of the longitudinal settlement trough was generally poorly predicted by the solution proposed by Attewell and Woodman (1982). However, this may have been due to the settlements recorded by two of the LVDTs being affected by the dimensions of the test chamber. The full data set of the LVDT placed nearest the rear face of the test chamber compared well with the results obtained from the solution.

The use of PIV and the associated software GeoPIV (White et al., 2003) has been used with great success to obtain information on both the magnitude of settlements and the shape of the settlement troughs in the transverse and longitudinal directions occurring at the rear Perspex face of the test chamber. The Gaussian distribution provided a good prediction of the sub-surface settlements in the transverse direction, despite some minor under-prediction at the shoulders of the trough which was related to the development of the chimney failure mechanism. The dependency of the trough width parameter on the depth from the soil surface and on volume loss was highlighted. It was concluded that the solution of Mair et al. (1993) generally provided a better prediction of the increase in $K$ with depth than that of Jacobsz (2002). The sub-surface longitudinal settlement trough was described and the settlement results obtained from the use of potentiometers and settlement plates were found to be unreliable quantitatively but useful qualitatively.

The complex three-dimensional interactions between the arching effect and the progression of the MTBM have been described. The transition of soil points from positive to negative to positive arching zones was observed: on approaching the zone in which the miniature
EPCs were placed, an increase in the vertical stress ratios was measured due to the advancement of the longitudinal settlement trough, with values up to 1.32 recorded. Following this, there was a reduction in the vertical stress ratio as the soil mass in which the cells were placed transferred stress via shear planes to adjacent soil masses. The most significant arching effect was directly above the tunnel centreline in close proximity to the crown, where ratios of less than 0.2 were recorded. Finally, a subsequent increase in the vertical stress to a final value was observed.

The dependency of the arching effect on both the relative magnitude of settlement between adjacent soil masses and shear planes was identified. The pattern of increasing minimum vertical stress ratio at 0Df and 1Df from the tunnel centreline as the depth from the soil surface decreased was observed, as was the increase in the final vertical stress ratio as the distance from the tunnel centreline increased. The extent of the arching zone was identified at approximately 3Df from the tunnel centreline.

The points made above highlight the importance of accurately modelling the three-dimensional nature of tunnelling through the simulation of the forward advancement of the MTBM and therefore the advancement of the resulting longitudinal settlement trough. If the progression of the MTBM is not modelled in three dimensions, the zone directly above the tunnel crown simply transfers stress to adjacent soil zones, resulting in similar stress-displacement curves to those obtained from cells placed directly above the trapdoor as presented in Chapter 4. This is in stark contrast to the rather complex variation in the actual arching behaviour of the soil observed in the MTBM tests and described above.
6. Numerical investigation

6.1 Introduction

A series of finite element (FE) analyses were carried out in order to investigate the arching effect in a sand mass due to an active trapdoor and the advancement of a miniature tunnel boring machine (MTBM). The aim of the FE investigation was to identify the patterns of stress-transfer in the soil mass surrounding the two problems, in order to develop the understanding of the mechanisms which occur in the arching zone, and to use this knowledge to interpret the findings from the physical modelling discussed in Chapters 4 and 5.

Two-dimensional (2D) analyses of the trapdoor and MTBM problems were carried out using a commercially available FE software program, PLAXIS version 9.2. However, the three-dimensional (3D) nature of the arching effect caused by the advancement of the MTBM has been highlighted in Chapter 5. Therefore, a 3D FE study of the problem has also been undertaken using PLAXIS 3D Tunnel version 2.4.

The theory of the soil models used in the numerical investigation is discussed, in addition to their parameters and how they were obtained. The results of the laboratory tests which were carried out on the Glenview sand used in the physical tests and used to obtain some of the FE parameters are presented, and are then compared with those obtained numerically. Following this, the results of the numerical modelling of both problems are presented, and the patterns of stress-transfer are discussed.

6.1.1 Finite element analysis

The FE Method is based on the principle of virtual work. An external load/displacement is applied incrementally, resulting in internal loads in a FE mesh. Equilibrium is achieved by equating the external and internal forces for each increment (Eqn. 6.1):

\[ K^i \Delta v^i = f_{ex}^i - f_{in}^{i-1} \]  

Eqn. 6.1

where \( K \) is a stiffness matrix, \( \Delta v \) is the incremental displacement vector, \( f_{ex} \) is the external force vector, \( f_{in} \) is the internal reaction factor, and \( i \) refers to the step number. As the
relationship between stress and strain increments is usually non-linear, the stiffness matrix is not known beforehand, meaning that an iterative procedure is necessary to satisfy both equilibrium and the constitutive relation. After taking into account the boundary conditions and ensuring compatibility between elements, a solution of the system of equations can be found.

6.2 Description of soil models

6.2.1 Mohr-Coulomb model

6.2.1.1 Theory of Mohr-Coulomb model

The Mohr-Coulomb model is a simple linear elastic perfectly plastic model. In order to distinguish between elastic and plastic strain, the full Mohr-Coulomb yield condition consists of six yield functions, \( f \), which are a function of the principal stresses. An example of one of these is shown in Eqn. 6.2:

\[
    f = (\sigma_1 - \sigma_3) - \sin \varphi (\sigma_1 + \sigma_3) - 2c \cos \varphi = 0
\]

Eqn. 6.2

where \( \sigma_1 \) and \( \sigma_3 \) are the major and minor principal stresses, respectively, \( c \) is the cohesion of the soil and \( \varphi \) is the angle of shearing resistance of the soil. The yield functions together form a fixed hexagonal cone in principal stress space, as shown in Figure 6.1.

![Figure 6.1 Mohr-Coulomb yield surface in principal stress space with c=0, from PLAXIS (2010)](image)

Six plastic potential functions, \( g \), are also defined for the model in terms of the principal stresses, one of which is given in Eqn. 6.3:

\[
    g = \frac{1}{2} (\sigma_1 - \sigma_3) + \frac{1}{2} (\sigma_1 + \sigma_3) \sin \psi
\]

Eqn. 6.3

where \( \psi \) is the angle of dilation of the soil.
6.2.1.2 Parameters of Mohr-Coulomb model

The use of the Mohr-Coulomb model requires the input of five parameters. Young’s Modulus (E) and Poisson’s ratio (v) describe the behaviour of the material in the elastic region, while the behaviour of the soil in the plastic region is described by c, φ and ψ.

The secant Young’s Modulus at 50% strength, $E_{50}$, is the basic stiffness value used in a Mohr-Coulomb simulation, although in some special cases, the use of the initial tangent modulus $E_0$ is recommended (Figure 6.2). The Young’s Modulus of the Glenview sand used for the purposes of this study was determined from a series of drained triaxial tests performed by the author, the results of which are presented in Figure 6.7 and Figure 6.8.

![Figure 6.2 Definition of $E_0$ and $E_{50}$ for standard drained triaxial test results, from PLAXIS (2010)](image)

The Poisson’s ratio is defined according to the following equation:

$$\frac{\sigma'_{xx}}{\sigma'_{yy}} = \frac{v}{(1 - v)} \quad \text{Eqn. 6.4}$$

In elastic computations of one-dimensional compression, its value is given by:

$$v = \frac{K_0}{(1 + K_0)} \quad \text{Eqn. 6.5}$$

where $K_0$ is the coefficient of lateral earth pressure at rest. Therefore, $v$ was found using the angle of shearing resistance and Jaky’s Law:

$$K_0 = 1 - \sin \varphi \quad \text{Eqn. 6.6}$$

As dry sand is used in this study, the cohesion is zero. However it is recommended by PLAXIS (2010) that $c > 0.2$ kPa is entered to avoid complications in some options within the program.
\( \varphi \) determines the shear strength of the soil. It was determined for the Glenview sand used in the study from both direct shear and triaxial tests. PLAXIS (2010) recommends the use of \( \varphi_c \) (i.e. the critical angle of shearing resistance) rather than \( \varphi_p \) (the peak angle).

The dilatancy of sand depends on both its density and \( \varphi \). PLAXIS (2010) suggests that the magnitude of the angle of dilation (\( \psi \)) can be found from:

\[
\psi \approx \varphi - 30^\circ \tag{Eqn. 6.7}
\]

The magnitude of the angle of dilation of the sand used in this study was found from drained triaxial tests. The rate of dilation can be represented by the parameter \( b \), where \( b \) is the slope of the \( e_{vol} \) versus \( \varepsilon_1 \) curve, where \( e_{vol} \) and \( \varepsilon_1 \) are the volumetric strain and axial strain, respectively. The maximum slope, \( b_{peak} \), occurs at the axial strain where the stress-strain curve peaks. The parameter \( b \) can then be related to the angle of dilation by:

\[
\sin \psi = \frac{b}{2 + b} \tag{Eqn. 6.8}
\]

### 6.2.2 Hypoplastic model

#### 6.2.2.1 Introduction

Hypoplasticity is a non-linear constitutive model whose framework was first given by Kolymbas (1977), although similar models were investigated by others. It was developed during the 1980’s and 1990’s at the University of Karlsruhe, Germany, and the first application of it to granular materials specifically was reported by Kolymbas (1985). Gudehus (1996) extended the basic model by allowing for the influence of stress level (barotropy) and the influence of density (pyknotropy). This model was adapted by von Wolfferdorff (1996) to include the Matsuoka-Nakai critical state stress condition, and von Wolfferdorff’s model is now generally considered the standard hypoplastic model for granular materials. The author applied this version in this study. It is able to predict many of the important features of soil behaviour, including dependency of the peak strength on soil density, non-linear behaviour and dependency of the soil stiffness on the loading direction.

#### 6.2.2.2 Theory of hypoplastic model

The hypoplastic constitutive equation determines the stress rate tensor \( \dot{T} \) as a function of the Cauchy effective stress \( T \), the granulate stretching rate \( D_g \) and the void ratio \( e \):

\[
\dot{T}_s = F(T, e, D_g) \tag{Eqn. 6.9}
\]
where $D_g$ is the symmetric part of the deformation gradient $L_d$:

$$D_g = \left( \frac{L_d + L_d^T}{2} \right)$$  \hspace{1cm} \text{Eqn. 6.10}

where $L_d^T$ is the transpose of $L_d$, and where

$$L_d = \frac{\delta v(x,t)}{\delta x}$$  \hspace{1cm} \text{Eqn. 6.11}

where $v$ is the velocity vector of the continuum representing the grain skeleton at a point $x$.

The rate of change of the void ratio, $\dot{e}$, assuming incompressibility of the soil grains, is given by:

$$\dot{e} = (1 + e) \text{tr}D_g$$  \hspace{1cm} \text{Eqn. 6.12}

where $\text{tr}D_g$ is the trace of the tensor $D_g$. In von Wolfferdorf’s version of the model, the constitutive equation takes the following form:

$$\dot{T}_s = f_b f_e \frac{1}{\text{tr}\overline{T}^2} \left[ P^2 D_g + a^2 \text{tr} (\nabla D_g) + f_d a F \left( \overline{T} + \overline{T}^* \sqrt{\text{tr}D_g^2} \right) \right]$$  \hspace{1cm} \text{Eqn. 6.13}

where $T$ is the dimensionless granular stress ratio tensor, $a$ is a parameter related to $\varphi_c$ and $F$ is a stress function. The parameters $f_e$ and $f_d$ take into account the pyknotropy of the material, and $f_b$ takes into account the barotropy of the material. These parameters are defined below:

$$\overline{T} = \frac{T}{\text{tr}T}$$  \hspace{1cm} \text{Eqn. 6.14}

$$\overline{T}^* = \overline{T} - \frac{1}{3} I$$  \hspace{1cm} \text{Eqn. 6.15}

where $I$ is the unit matrix.

$$a = \frac{\sqrt{3}(3 - \sin \varphi_c)}{2\sqrt{2} \sin \varphi_c}$$  \hspace{1cm} \text{Eqn. 6.16}

$$F = \sqrt{8} \tan^2 \psi + \frac{2 - \tan^2 \psi}{2 + \sqrt{2} \tan \psi \cos 3\theta} - \frac{1}{2\sqrt{2}} \tan \psi$$  \hspace{1cm} \text{Eqn. 6.17}

$$\tan \psi = \sqrt{3 \text{tr}\overline{T}^2}$$  \hspace{1cm} \text{Eqn. 6.18}

$$\cos 3\theta = -\sqrt{6} \frac{\text{tr}\overline{T}^*^3}{[\text{tr}\overline{T}^*^2]^2}$$  \hspace{1cm} \text{Eqn. 6.19}

where $\theta$ is an angle relating the principal stresses.

$$f_b = \frac{h_s}{n} \left( \frac{e_{i0}}{e_{c0}} \right)^{\mu} \left( \frac{1 + e_i}{h_s} \right)^{1-n} \left[ 3 + a^2 - a \sqrt{3} \left( \frac{e_{i0} - e_{d0}}{e_{c0} - e_{d0}} \right)^{\alpha} \right]^{-1}$$  \hspace{1cm} \text{Eqn. 6.20}

where $h_s$ is the granulate hardness, $p_0$ is the mean pressure, and $e_{i0}$, $e_{c0}$ and $e_{d0}$ are the maximum, critical and minimum void ratios at zero pressure, respectively. The parameters
n, α and β are parameters related to the compression characteristics, θp, and shear stiffness, respectively. These parameters are discussed in further detail below. It should be noted that hs is not a measure of the hardness of individual grains, but is used as a reference pressure.

\[ f_d = \left( \frac{e - e_d}{e_c - e_d} \right) ^\alpha \]  
Eqn. 6.21

where \( e_c, e_d \) and \( e_i \) are the characteristic void ratios.

\[ f_e = \left( \frac{e_c}{e} \right) ^\beta \]  
Eqn. 6.22

The characteristic void ratios \( e_i, e_c \) and \( e_d \) decrease with increasing mean skeleton pressure (\( p_s \)), according to the compression law of Bauer (1996), as follows:

\[ \frac{e_i}{e_i^0} = \frac{e_c}{e_c^0} = \frac{e_d}{e_d^0} = e^\left( \frac{3p_s}{h_s} \right)^n \]  
Eqn. 6.23

This relationship is shown in Figure 6.3 below, where shaded zones show impossible states for simple grain skeletons.

![Figure 6.3 Relationship between \( e_i, e_c, e_d \) and \( p_s \) in logarithmic scale, from Herle and Gudehus (1999)](image)

6.2.2.3 Method of determination of hypoplastic parameters

The hypoplastic model requires the input of eight material parameters for the modelling of granular materials: \( \phi_c, h_s, e_{d0}, e_{i0}, e_{c0} \), and the exponents \( n, \alpha \) and \( \beta \). Both Herle and Gudehus (1999) and Mašin (2010) describe how these parameters can easily be determined from simple laboratory tests.

A simple estimation of \( \phi_c \) may be found from the angle of repose of a loose pile of dry soil which has been subjected to toe excavation (Cornforth, 1973). However, for the Glenview sand, it was found from drained triaxial tests and compared with values obtained from direct simple shear tests.
Chapter 6 Numerical investigation

The granulate hardness \((h_s)\) and the exponent \(n\) were determined from the loading curve obtained from an oedometer test on a very loose sample of the Glenview sand, the results of which are presented in § 6.3.2.2. The sample may be either completely dry or saturated. \(n\) takes into account the pressure sensitivity of the grain skeleton. It allows for a non-proportional increase of the incremental stiffness with increasing \(p_s\), and is found from:

\[
 n = \frac{\ln \left( \frac{e_{p1}}{e_{p2}} \frac{C_{c2}}{C_{c1}} \right)}{\ln \left( \frac{ps2}{ps1} \right)}
\]

Eqn. 6.24

where \(p_{s1}\) and \(p_{s2}\) are mean stresses calculated from axial stresses using the Jáky formula, and \(e_{p1}\) and \(e_{p2}\) are the void ratios corresponding to \(p_{s1}\) and \(p_{s2}\). Compression indices \(C_{c1}\) and \(C_{c2}\) can be calculated as the slopes of tangents to the loading curve at \(p_{s1}\) and \(p_{s2}\), as shown in Figure 6.4.

\[h_s\] may then be calculated from:

\[
h_s = 3p_s \left( \frac{ne_p}{C_c} \right)^{\frac{1}{n}}
\]

Eqn. 6.25

However, various values of \(h_s\) and \(n\) may be found, depending on the values of \(p_{s1}\) and \(p_{s2}\) that are chosen. Herle and Gudehus (1999) therefore proposed an empirical relationship between the coefficient of uniformity of the sample \(\left( C_U = \frac{D_{60}}{D_{10}} \right)\), the mean grain size \(D_{50}\) and the exponent \(n\):

\[
n = 0.366 - 0.0341x_s
\]

Eqn. 6.26

where

\[
x_s = \frac{C_U}{\left( \frac{D_{50}}{D_0} \right)^3}, \quad \text{with } D_0 = 1mm
\]

Eqn. 6.27
The minimum void ratio $e_d$ of a granular material is reached by cyclic shearing with small amplitude under constant pressure. The minimum void ratio $e_{\text{min}}$ is usually found in the laboratory using methods such as that described in BS 1377-4 (BSI, 1990). However, these are not as effective as cyclic shearing, and therefore the value of $e_d$ obtained is lower than the value of $e_{\text{min}}$. Youd (1973) proposed a relationship between $e_d$, $C_u$ and the angularity of the grains. Herle and Gudehus (1999), after a comparison of the minimum void ratio at zero pressure ($e_{d0}$) with $e_{\text{min}}$ for several sands, concluded that it can be assumed that $e_{d0}$ is approximately equal to $e_{\text{min}}$.

Herle and Gudehus (1999) describe the maximum void ratio at zero pressure ($e_{i0}$) as the maximum void ratio of a simple grain skeleton which is reached during isotropic consolidation of a grain suspension in a gravity-free space. The aforementioned authors conclude that it is almost impossible to determine experimentally, but that from a comparison of experimental $e_{\text{max}}$ values with theoretical $e_{i0}$ values, $e_{i0}/e_{\text{max}} \approx 1.2$ can be assumed for identical spheres, and 1.3 for cubes.

The critical void ratio at zero pressure ($e_{c0}$) may be found from undrained triaxial tests on loose samples. However in this study, $e_{c0} \approx e_{\text{max}}$ was assumed, as proposed by Herle and Gudehus (1999).

The exponent $\alpha$ controls $\varphi_p$ and was found in this study by means of a drained triaxial test. By considering a peak state in a triaxial test, $\alpha$ can be found from the following equation:

$$
\alpha = \frac{\ln \left[ \frac{(2 + K_p)^2 + a^2K_p(K_p - 1 - \tan \psi_p)}{a(2 + K_p)(5K_p - 2)} \right]}{\ln r_e}
$$

where $K_p$ is the coefficient of passive earth pressure and is defined as

$$
K_p = \frac{1 + \sin \varphi_p}{1 - \sin \varphi_p}
$$

The peak dilatancy angle, $\psi_p$, may be found from the following equation:

$$
\tan \psi_p = 2 \frac{K_p - 4 + 5AK_p^2 - 2AK_p}{(5K_p - 2)(1 + 2A)} - 1
$$
Chapter 6 Numerical investigation

\[ A = \frac{a^2}{(2 + K_p)^2} \left[ 1 - \frac{K_p(4 - K_p)}{5K_p - 2} \right] \]  

Eqn. 6.31

\[ r_e = \frac{e - e_d}{e_c - e_d} \]  

Eqn. 6.32

The exponent \( \beta \) controls the shear stiffness (as a function of density and pressure) and is only significant for dense soils. Although according to Herle and Gudehus (1999) it may be assumed for most natural sands that \( \beta = 1 \), it was calculated from data from oedometer tests on both loose and dense samples (as presented in Figure 6.9). \( \beta \) can be found from:

\[ \beta = \frac{\ln \left( \frac{\beta_0 E_2}{E_1} \right)}{\ln \left( \frac{e_1}{e_2} \right)} \]  

Eqn. 6.33

where \( e_1 \) and \( e_2 \) are the void ratios of oedometer tests on loose and dense samples respectively, \( E_1 \) and \( E_2 \) are the stiffness moduli obtained from oedometer tests on loose and dense samples respectively, and \( \beta_0 \) is a parameter defined as:

\[ \beta_0 = \frac{3 + a^2 - a\sqrt{3}f_{d1}}{3 + a^2 - a\sqrt{3}f_{d2}} \]  

Eqn. 6.34

where \( f_{d1} \) and \( f_{d2} \) are defined as:

\[ f_{d1} = \left( \frac{e_1 - e_d}{e_c - e_d} \right)^{\alpha} \]  

Eqn. 6.35

\[ f_{d2} = \left( \frac{e_2 - e_d}{e_c - e_d} \right)^{\alpha} \]  

Eqn. 6.36

6.2.3 Hardening Soil model

6.2.3.1 Introduction

The Hardening Soil model, as described by Schanz (1998) and Schanz et al. (1999), is a combination of the Duncan-Chang Model (Duncan and Chang, 1970), but uses the theory of plasticity rather than elasticity. Unlike the Mohr-Coulomb model, its yield surface is not fixed but can expand due to plastic straining. Two types of hardening are distinguished, namely shear hardening and compression hardening, and soil dilatancy and a yield cap are also incorporated. The main characteristics of the model, as discussed by PLAXIS (2010), are:

- Stress dependent stiffness according to a power law
- Hyperbolic relationship between deviatoric stress and strain
- Distinction between primary deviatoric loading, primary compression and unloading/reloading
- Failure according to the Mohr-Coulomb model, defined by $c$, $\varphi$ and $\psi$

This model has previously been used to predict the settlements due to open-face tunnelling (Lochaden et al., 2008) and the deformations around two excavations (Lawler et al., 2011).

6.2.3.2 Theory of Hardening Soil model

On initial drained triaxial shearing, the relationship between the vertical strain ($\varepsilon_1$) and the deviatoric stress ($q$) can be written as:

$$-\varepsilon_1 = \left(\frac{1}{E_i}\right) \left(\frac{q}{q_a}\right), \quad \text{for } q < q_f$$  \hspace{1cm} \text{Eqn. 6.37}

where $q_a$ is the asymptotic value of shear strength (Figure 6.5), $q_f$ is the ultimate deviator stress, $E_i$ is the initial stiffness, which is related to the secant stiffness at 50% strength in standard drained triaxial tests ($E_{50}$) by:

$$E_i = \frac{2E_{50}}{2 - R_f}$$  \hspace{1cm} \text{Eqn. 6.38}

where $R_f$ is the failure ratio defined as:

$$R_f = \frac{q_f}{q_a}$$  \hspace{1cm} \text{Eqn. 6.39}

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{figure6.5.png}
\caption{Hyperbolic stress-strain relationship for a standard drained triaxial test, from PLAXIS (2010)}
\end{figure}

$E_{50}$ is given by:

$$E_{50} = E_{50}^{ref} \left(\frac{c \cos \varphi - \sigma_1 \sin \varphi}{c \cos \varphi + p^{ref} \sin \varphi}\right)^m$$  \hspace{1cm} \text{Eqn. 6.40}

where $E_{50}^{ref}$ is the secant stiffness in standard drained triaxial tests corresponding to the reference pressure $p^{ref}$ and $m$ is a power for the stress-level dependency of stiffness.
For oedometer conditions of stress and strain, the following relationship applies:

$$E_{oed} = E_{oed}^{ref} \left( \frac{c \cos \varphi - \sigma_1 \sin \varphi}{c \cos \varphi + p^{ref} \sin \varphi} \right)^m$$  \hspace{1cm} \text{Eqn. 6.41}

where $E_{oed}$ is the tangent oedometer stiffness and $E_{oed}^{ref}$ is the tangent stiffness for primary oedometer loading at $p^{ref}$.

The unload/reload stiffness is defined in terms of the unload/reload stiffness at $p^{ref}$ ($E_{ur}^{ref}$) as:

$$E_{ur} = E_{ur}^{ref} \left( \frac{c \cos \varphi - \sigma'_1 \sin \varphi}{c \cos \varphi + p^{ref} \sin \varphi} \right)^m$$ \hspace{1cm} \text{Eqn. 6.42}

$E_{50}^{ref}$ controls the shear yield surface and $E_{oed}^{ref}$ controls the cap yield surface. The yield surfaces in principal stress space are shown in Figure 6.6.

![Figure 6.6 Hardening Soil yield surface in principal stress space with c=0, after PLAXIS (2010)](image)

6.2.3.3 Method of determination of Hardening Soil parameters

The methods of determining c, $\varphi_c$, and $\psi$ are discussed in § 6.2.1.2. The reference secant stiffness ($E_{50}^{ref}$) was obtained from standard drained triaxial tests, as shown in Figure 6.5. It is the secant stiffness at 50% strength corresponding to $p^{ref}$. Although PLAXIS (2010) suggests a default value of $0.8E_{50}^{ref}$ may be used for the reference tangent stiffness ($E_{oed}^{ref}$), this value was found from the $\sigma_1-\varepsilon_\alpha$ curves obtained from oedometer tests, where $\sigma_1$ is the axial stress. Although a default value for the reference unloading/reloading stiffness ($E_{ur}^{ref}$) of $3E_{50}^{ref}$ is suggested by PLAXIS (2010), this value was also found from the $\sigma_1-\varepsilon_\alpha$ curves obtained from oedometer tests.
The stress-strain data from drained triaxial tests is replotted on a transformed vertical axis of $\varepsilon_1/(\sigma_1-\sigma_3)$ to determine the value of $E_i$ and $q_a$, which are inversely proportional to the intercept and slope of the data respectively. This allows the calculation of $R_f$ using Eqn. 6.39. The value of $m$ may then be obtained from the slope of the line through the $\log E_i-\sigma_3$ data. PLAXIS (2010) suggests a default value of $m=0.5$ for sands. This proved to be a more suitable figure for simulating the response of the soil in drained triaxial and oedometer tests, and was therefore used in place of the one calculated using the method above.

6.3 Laboratory tests

6.3.1 Introduction

A series of laboratory tests consisting of drained triaxial and oedometer tests were performed on the Glenview sand used in the physical tests, in order to obtain some of the parameters used in the FE material models. A series of drained shear box tests were also carried out, the results of which are not presented but referred to when appropriate.

6.3.2 Presentation of results

6.3.2.1 Triaxial tests

Four triaxial tests were carried out at confining stress intervals of 50kPa in a range from 50kPa-200kPa. The initial void ratio of each sample was calculated from its dimensions, mass, and the specific gravity of the sand. Large samples (with diameters of approximately 100mm) were tested, as the calculation of the initial void ratio is highly influenced by inaccuracies in the measurement of the dimensions, which are reduced by using larger samples. The samples were loaded until an axial strain of 20% was reached, before being unloaded to the confining pressure. The specimens were prepared by pluviating dry sand from a funnel through water, and the funnel was lifted so that the drop height was kept constant. The axial deformation of the specimen was measured by a displacement transducer at the top of the sample. The results of these tests are shown in Figure 6.7 and Figure 6.8.
6.3.2.2 Oedometer tests

Three oedometer tests were carried out on loose and dense samples to quantify the loading and unloading characteristics of the material under confined conditions. The loose and dense samples, with average initial void ratios of 0.821 and 0.556 respectively, were loaded to a maximum vertical stress of 600kPa and 400kPa respectively. The results obtained are shown in Figure 6.9.
Figure 6.9 Oedometer test results on Glenview sand: (a), (b) loose sample (c), (d) dense sample
6.3.3 Discussion of results

6.3.3.1 Triaxial tests

$\varphi_c$ and $\varphi_p$ were found from the data presented in Figure 6.7 as $37^\circ$ and $46.7^\circ$, respectively. These values compare well to those obtained from direct shear box tests ($36.2^\circ$ and $46.5^\circ$, respectively).

The values of $b_{\text{peak}}$ obtained from the plots of volumetric strain versus axial strain presented in Figure 6.8 are plotted against $\sigma_3$ in Figure 6.10. The corresponding angles of dilatancy obtained using Eqn. 6.8 are presented in Table 6.1.

![Figure 6.10 Relationship between stress and dilatancy from triaxial tests](image)

<table>
<thead>
<tr>
<th>Confining stress, $\sigma_3$ [kPa]</th>
<th>Angle of dilatancy [$^\circ$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>16.4</td>
</tr>
<tr>
<td>100</td>
<td>16.7</td>
</tr>
<tr>
<td>150</td>
<td>13.4</td>
</tr>
<tr>
<td>200</td>
<td>13.1</td>
</tr>
</tbody>
</table>

Table 6.1 Relationship between stress and dilatancy from triaxial tests

6.3.3.2 Oedometer tests

A power law was applied to the data of $\varepsilon_1$ versus $\sigma_1$ which was presented in Figure 6.9, as proposed by Janbu (1963). This was then differentiated in order to find an expression for $E_{\text{oed}}$, as:

$$E_{\text{oed}} = \frac{\delta \sigma_1}{\delta \varepsilon_1}$$  
Eqn. 6.43
This led to the relationship plotted in Figure 6.11 and shown in Table 6.2.

![Figure 6.11 Relationship between oedometer stiffness and confining stress](image)

<table>
<thead>
<tr>
<th>Sample state</th>
<th>Loading condition</th>
<th>Stiffness, $E_{\text{ood}}$ [kPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loose</td>
<td>Loading</td>
<td>$4906\sigma_3^{0.356}$</td>
</tr>
<tr>
<td></td>
<td>Unloading</td>
<td>$1736\sigma_3^{0.919}$</td>
</tr>
<tr>
<td>Dense</td>
<td>Loading</td>
<td>$11151\sigma_3^{0.364}$</td>
</tr>
<tr>
<td></td>
<td>Unloading</td>
<td>$4929\sigma_3^{0.734}$</td>
</tr>
</tbody>
</table>

Table 6.2 Relationship between oedometer stiffness and axial stress

The stiffness parameter $E$ necessary for use in the Mohr Coulomb model was related to $E_{\text{ood}}$ using Poisson’s ratio ($v$):

$$E = \frac{(1 + v)(1 - 2v)}{1 - v} E_{\text{ood}}$$

Eqn. 6.44

This allowed the relationship between $E$ and $\sigma_3$ to be found, as shown in Table 6.3.

<table>
<thead>
<tr>
<th>Sample state</th>
<th>Loading condition</th>
<th>Stiffness, $E$ [kPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loose</td>
<td>Loading</td>
<td>$3795\sigma_3^{0.356}$</td>
</tr>
<tr>
<td></td>
<td>Unloading</td>
<td>$1343\sigma_3^{0.919}$</td>
</tr>
<tr>
<td>Dense</td>
<td>Loading</td>
<td>$9826\sigma_3^{0.364}$</td>
</tr>
<tr>
<td></td>
<td>Unloading</td>
<td>$4343\sigma_3^{0.734}$</td>
</tr>
</tbody>
</table>

Table 6.3 Relationship between stiffness and axial stress

### 6.3.4 Calculated parameters for soil models

#### 6.3.4.1 Mohr-Coulomb model

The Mohr-Coulomb parameters used for the simulation of triaxial and oedometer tests (discussed in § 6.4) are listed in Table 6.4. For the oedometer tests, $\sigma_3$ was calculated from the mean vertical stress using $K_0$. $E$ and $v$ were calculated from the relationships presented...
in Table 6.3 and Eqn. 6.5, respectively. The relationship given in Figure 6.10 was used to calculate the angle of dilation for the dense samples. Eqn. 6.7 was used to calculate $\psi$ for the loose sample.

<table>
<thead>
<tr>
<th>Test</th>
<th>$\sigma$ [kPa]</th>
<th>E [kPa]</th>
<th>$\nu$ [-]</th>
<th>$c$ [kPa]</th>
<th>$\phi$ [$^\circ$]</th>
<th>$\psi$ [$^\circ$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Triaxial (dense)</td>
<td>50</td>
<td>40810</td>
<td>0.214</td>
<td>0.5</td>
<td>46.7</td>
<td>18.0</td>
</tr>
<tr>
<td>Triaxial (dense)</td>
<td>100</td>
<td>52530</td>
<td>0.214</td>
<td>0.5</td>
<td>46.7</td>
<td>15.6</td>
</tr>
<tr>
<td>Oedometer (loose)</td>
<td>123.66</td>
<td>21090</td>
<td>0.292</td>
<td>0.5</td>
<td>36</td>
<td>6.0</td>
</tr>
<tr>
<td>Oedometer (dense)</td>
<td>54.44</td>
<td>42100</td>
<td>0.214</td>
<td>0.5</td>
<td>46.7</td>
<td>16.7</td>
</tr>
</tbody>
</table>

Table 6.4 Mohr-Coulomb parameters for simulation of triaxial and oedometer tests

### 6.3.4.2 Hypoplastic model

Table 6.5 shows the hypoplastic model parameters used for the simulation of the laboratory tests, which were found using the methods described in § 6.2.2.3.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>$\phi_c$ [$^\circ$]</th>
<th>$h_s$ [MPa]</th>
<th>$n$ [-]</th>
<th>$e_{do}$ [-]</th>
<th>$e_{so}$ [-]</th>
<th>$\epsilon_0$ [-]</th>
<th>$\alpha$ [-]</th>
<th>$\beta$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Value</td>
<td>36</td>
<td>330</td>
<td>0.479</td>
<td>0.520</td>
<td>0.846</td>
<td>1.016</td>
<td>0.093</td>
<td>2.19</td>
</tr>
</tbody>
</table>

Table 6.5 Hypoplastic model parameters for simulation of triaxial and oedometer tests

### 6.3.4.3 Hardening Soil model

The Hardening Soil model parameters used for the simulation of the laboratory tests are shown in Table 6.6. An asterisk denotes that the default value has been used.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Loose oedometer test</th>
<th>All other tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_{50}^{ref}$ [kPa]</td>
<td>18980</td>
<td>18980</td>
</tr>
<tr>
<td>$E_{oed}^{ref}$ [kPa]</td>
<td>16500</td>
<td>38500</td>
</tr>
<tr>
<td>$E_{ur}^{ref}$ [kPa]</td>
<td>90000</td>
<td>90000</td>
</tr>
<tr>
<td>$c$ [kPa]</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>$\phi_c$ [$^\circ$]</td>
<td>36</td>
<td>46.7</td>
</tr>
<tr>
<td>$\psi$ [$^\circ$]</td>
<td>6</td>
<td>16.7</td>
</tr>
<tr>
<td>$m$ [-]</td>
<td>0.5*</td>
<td>0.5*</td>
</tr>
<tr>
<td>$R_f$ [-]</td>
<td>0.672</td>
<td>0.672</td>
</tr>
<tr>
<td>$p_{ur}^{ref}$ [kPa]</td>
<td>100*</td>
<td>100*</td>
</tr>
<tr>
<td>$v_{ur}$ [-]</td>
<td>0.2*</td>
<td>0.2*</td>
</tr>
</tbody>
</table>

Table 6.6 Hardening Soil parameters for simulation of triaxial and oedometer tests
The measured $E_{oed}^{ref}$ value of 16500 kPa for the loose sample compared reasonably well with the default value using the relationship presented in § 6.2.3.3 of 15200 kPa. This default value is significantly lower however than the measured $E_{oed}^{ref}$ value of 38500 kPa for the dense sample. The measured $E_{ur}^{ref}$ value of 90000 kPa is significantly higher than the default value of 57000 kPa (also from the relationship presented in § 6.2.3.3).

6.4 Comparison of laboratory and numerical data

6.4.1 Introduction

Two oedometer tests (loose and dense) and two of the triaxial tests which were carried out in the laboratory were modelled numerically to investigate the performance of the three soil models, namely the Mohr-Coulomb, Hardening Soil and Hypoplastic soil models (abbreviated as MC, HS, and HYP respectively in the figures below).

6.4.2 Oedometer tests

6.4.2.1 Presentation of results

The numerical model for the oedometer tests consisted of a unit sample of soil with a very coarse mesh. The bottom and vertical boundaries were fixed in the horizontal and vertical directions. The sample was then loaded vertically to the maximum stress applied in the laboratory tests. Both test series carried out in the laboratory (i.e. on loose and dense samples) were simulated, the results of which are shown in Figure 6.12 and Figure 6.13.
6.4.2.2 Discussion of results

The qualitative behaviour of the models is similar for both the loose and dense samples. The Mohr-Coulomb model is stiffer than the other two on initial loading. The curve also follows the same path for loading and unloading, which clearly is not representative of the actual behaviour of the sample. The Hardening Soil model very accurately predicts the behaviour of the soil. However, it should be kept in mind that the parameters for this model were obtained from the results of the oedometer test, and therefore this is not a true prediction as the parameters were matched to this data. The hypoplastic model, although simulating the non-linear stress-strain behaviour of the soil in both loading and unloading, shows a much stiffer response than that observed in the laboratory.

6.4.3 Triaxial tests

6.4.3.1 Presentation of results

The numerical model for the triaxial tests consisted of a unit sample of soil with a very coarse mesh. The bottom boundary was fixed in the horizontal and vertical directions. The horizontal boundary on the left side of the sample (i.e. the axis of symmetry of the sample) was fixed in the horizontal direction. The triaxial tests with confining stresses of 50kPa and 100kPa carried out in the laboratory were modelled numerically and compared with the measured results. The results of these analyses are shown in Figure 6.14.
Figure 6.14 Comparison of experimental and numerical results for (a), (b) triaxial test with $\sigma_3=50\text{kPa}$ and (c), (d) triaxial test with $\sigma_3=100\text{kPa}$
6.4.3.2 Discussion of results

The Mohr-Coulomb model does not capture the non-linear stress-strain behaviour. The initial stiffness of the sample is overestimated and the peak strength is not accurately predicted. For $\sigma_3=50\text{kPa}$, the dilation of the soil is overestimated. However, for $\sigma_3=100\text{kPa}$, the model provides a very good prediction of the dilation.

The Hardening Soil model captures the initial stiffness of the sample very well. For $\sigma_3=50\text{kPa}$, the peak strength is very close to that measured in the laboratory, but is overestimated for $\sigma_3=100\text{kPa}$. Typical results of a triaxial test on a loose and dense are shown in Figure 6.15. As the Hardening Soil model is a hyperbolic model, it is unable to predict the post-failure behaviour of the dense soil tested in the triaxial tests performed in the laboratory. However, it must be assumed that the model would provide more accurate results for loose samples, as are used in the trapdoor and MTBM test chamber experiments, as the results would take the shape of a hyperbola (Figure 6.15). The dilatancy behaviour of the soil is underestimated by the Hardening Soil model, particularly for $\sigma_3=100\text{kPa}$, as shown in Figure 6.14 sub-figure ‘d’.

![Figure 6.15 Typical triaxial test results on loose and dense sand samples, from Craig (2004)](image)

The hypoplastic model provides a much stiffer response to that measured in the laboratory. The peak strength is overestimated in both tests. The strain softening behaviour of the sample is simulated by the model, although the final deviator stress at 20% axial strain is still significantly higher than that measured. The model predicts the dilatancy of the sample reasonably well, particularly for $\sigma_3=50\text{kPa}$.
6.4.4 Problem with hypoplastic model

During initial simulations of both the trapdoor and MTBM tests using the Hypoplastic model, tensile stresses were predicted directly above both structures. This was due to the fact that, as specified in Mašín (2010), the model is undefined in the tensile region and artificial cohesion is introduced in the model. However, despite considerable efforts, it was not possible to obtain acceptable results using this model.

6.4.5 Conclusions

The Mohr-Coulomb model and Hardening Soil model were chosen to carry out further investigations into the trapdoor and MBTM tests, based on the results presented in § 6.4.2, § 6.4.3 and § 6.4.4. The Mohr-Coulomb model was used as it is a simple model which provides a very quick approximation of the problem using relatively basic soil parameters. The Hardening Soil model was chosen as it models the stiffness of the soil under various loading conditions very accurately, and although it did not accurately simulate the post-failure behaviour of the dense sample, it is expected that it would provide satisfactory results for a loose sample, due to that fact that it is a hyperbolic model. The hypoplastic model was not used due to the reasons specified in § 6.4.4.

6.5 Numerical modelling of active trapdoor problem

6.5.1 Introduction

This section presents and discusses the results of 2D FE analyses carried out to investigate the arching effect above and adjacent to an active trapdoor.

6.5.2 Preliminary analysis

6.5.2.1 Introduction

A preliminary 2D FE analysis was carried out on an active trapdoor problem to determine the influence of different variables on the results and to obtain the most suitable model configuration for further analyses. As the problem is symmetric, only half of it was modelled. Therefore, the dimensions used were 0.4m x 0.9m (width x height). The bottom and vertical boundaries were fixed in the horizontal and vertical directions. The lowering of the trapdoor was simulated by a prescribed displacement across a length of 0.075m at the base. The factors investigated were the mesh coarseness and the use of interface
elements at the trapdoor edge. All soil model parameters were obtained from the relationships presented in § 6.3. A screenshot of the model with an interface length of 0.5$D_{TD}$ is shown in Figure 6.16, where $D_{TD}$ is the diameter of the trapdoor.

![Figure 6.16 Screenshot of trapdoor FE model with interface length of 0.5$D_{TD}$](image)

Prior to FE calculations being performed, the geometry must be divided into a mesh of finite elements. The mesh generator requires a parameter which represents the average element size, $l_e$, as defined in Eqn. 6.45:

$$l_e = \sqrt{\frac{(x_{max} - x_{min})(y_{max} - y_{min})}{n_c}}$$

Eqn. 6.45

where $x_{max}$, $x_{min}$, $y_{max}$ and $y_{min}$ are the outer geometry dimensions, and $n_c$ is a parameter which depends on the degree of mesh coarseness chosen by the user. A mesh should generally be finer in areas where large deformations are expected, to improve accuracy. The effect of three different mesh settings was tested in the preliminary analysis: fine, medium and coarse. These correspond to $n_c$ values of 200, 100 and 50, respectively.

PLAXIS (2010) recommends the use of an interface placed perpendicularly to the transition zone between the fixity beside the prescribed displacement to the prescribed displacement itself, as shown in both Figure 6.16 and Figure 6.17. Four different interface element configurations were investigated: no interface, and an interface with lengths of 0.5$D_{TD}$, 1$D_{TD}$ and 2$D_{TD}$. The strength properties of the interface element were the same as the surrounding soil.
The effect of the mesh coarseness and use of interface elements was investigated by comparing the vertical stress ratio (as defined in § 4.4.1) acting on the centre of the trapdoor and directly above the trapdoor centreline at a height of \( 0.5D_{TD} \). Table 6.7 and Table 6.8 present a summary of the tests carried out to establish the effect of the mesh size and interface elements respectively.

**Table 6.7 Summary of FE trapdoor tests carried out to establish effect of mesh coarseness**

<table>
<thead>
<tr>
<th>Soil model</th>
<th>Test name</th>
<th>Mesh coarseness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mohr-Coulomb</td>
<td>FE-TDCAL-001</td>
<td>Fine</td>
</tr>
<tr>
<td></td>
<td>FE-TDCAL-002</td>
<td>Medium</td>
</tr>
<tr>
<td></td>
<td>FE-TDCAL-003</td>
<td>Coarse</td>
</tr>
<tr>
<td>Hardening Soil</td>
<td>FE-TDCAL-004</td>
<td>Fine</td>
</tr>
<tr>
<td></td>
<td>FE-TDCAL-005</td>
<td>Medium</td>
</tr>
<tr>
<td></td>
<td>FE-TDCAL-006</td>
<td>Coarse</td>
</tr>
</tbody>
</table>

**Table 6.8 Summary of FE trapdoor tests carried out to establish effect of interface length**

<table>
<thead>
<tr>
<th>Soil model</th>
<th>Test name</th>
<th>Interface length ([D_{TD}])</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mohr-Coulomb</td>
<td>FE-TDCAL-001a</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>FE-TDCAL-001b</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>FE-TDCAL-001c</td>
<td>2</td>
</tr>
<tr>
<td>Hardening Soil</td>
<td>FE-TDCAL-005a</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>FE-TDCAL-005b</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>FE-TDCAL-005c</td>
<td>2</td>
</tr>
</tbody>
</table>

**6.5.2.2 Effect of mesh coarseness**

The results of tests FE-TDCAL-001 to FE-TDCAL-003 are presented in Figure 6.18. The fine mesh appears to give the most appropriate results when using the Mohr-Coulomb model. It is unlikely that a stress ratio of zero on the trapdoor base would be encountered in
practice, as observed when a medium or coarse mesh is used. The fine mesh also shows more appropriate stress ratios at $0.5D_{TD}$ above the trapdoor, as it is higher than that experienced on the trapdoor itself. This is in comparison with the medium and coarse meshes, which show approximately the same stress ratio on the trapdoor as at $0.5D_{TD}$ above the trapdoor. For these reasons, a fine mesh is used for all trapdoor simulations using the Mohr-Coulomb soil model. It should be noted that the unexpected results (for example, $0.5D_{TD}$ above the trapdoor using the coarse mesh) may be as a result of numerical errors due to proximity to the boundaries of the soil cluster.

![Graph](a)

![Graph](b)

Figure 6.18 Effect of mesh coarseness for Mohr-Coulomb model (a) at base (b) $0.5D_{TD}$ above trapdoor

The results of tests FE-TDCAL-004 to FE-TDCAL-006 are presented in Figure 6.19. In this case, the fine mesh does not appear to give good results. There is a sudden peak in the stress ratio at the base of the trapdoor after a normalised trapdoor displacement of
approximately 2%. This may be as a result of numerical errors due to the proximity to the boundaries of the soil cluster. The medium and coarse meshes also show some variability in the stress ratio, and none of the meshes can be considered to provide completely satisfactory results. However, a medium mesh was used for all trapdoor simulations using the Hardening Soil model, as it appears to give marginally more consistent results than the other two configurations.

![Diagram](image)

**Figure 6.19** Effect of mesh coarseness for Hardening Soil model (a) at base (b) 0.5D<sub>TD</sub> above trapdoor

### 6.5.2.3 Effect of interface elements

Having established that the fine and medium meshes were most suitable for the Mohr-Coulomb and Hardening Soil models respectively, these configurations were then used to study the effect of the interface elements. The results of FE-TDCAL-001 to FE-TDCAL-001c are shown in Figure 6.20. Other than an interface length of 1D<sub>TD</sub>, the behaviour of the
stress acting on the trapdoor is not significantly influenced by the presence and/or length of the interface (sub-figure ‘a’). The stress acting at 0.5D_{TD} above the trapdoor is influenced by both the presence of the interface and its length. However, there is no evident pattern in these results.

As discussed in § 6.5.2.1, the strength properties of the interface were the same as the surrounding soil. The interface elements should therefore not influence the failure mechanism to occur in a specific location or direction. However, on comparing the shear strains when an interface is and is not used, it is evident that the interface is influencing the direction, as shown in Figure 6.21 for a normalised trapdoor displacement of 2%. Therefore, interface elements were not used for future trapdoor analyses using the Mohr-Coulomb model.
The results of FE-TDCAL-005 to FE-TDCAL-005c are shown in Figure 6.20. The stress acting on the trapdoor and at a height of $0.5D_{TD}$ above the trapdoor is influenced by the presence of the interface but not by its length. However, a similar effect to that observed when the Mohr-Coulomb model was used on the shear strains was observed for this scenario. Therefore, an interface was not used for all further analyses using the Hardening Soil model.

6.5.3 Presentation of results of detailed analysis

6.5.3.1 Introduction
The stress transfer mechanisms above and adjacent to the trapdoor were investigated by examining the variation in the vertical and horizontal stress ratios at various points above and adjacent to the trapdoor. The exact positions of these are presented in Table 6.9.
Figure 6.22 Effect of interface for Hardening Soil model (a) at base (b) 0.5D\(_{TD}\) above trapdoor

<table>
<thead>
<tr>
<th>Horizontal distance from trapdoor centreline [D(_{TD})]</th>
<th>Vertical distance from base of test chamber [D(_{TD})]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
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<td>2</td>
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<tr>
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<td>4</td>
</tr>
<tr>
<td>0.7, 1.2</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>2</td>
</tr>
</tbody>
</table>

Table 6.9 Positions at which stresses were examined for purposes of trapdoor FE analyses
6.5.3.2 Behaviour directly above trapdoor centreline

The vertical and horizontal stress ratios directly above the trapdoor centreline obtained from the numerical analysis at various distances from the base of the test chamber for the Mohr-Coulomb and Hardening Soil models are presented in Figure 6.23.

Figure 6.23 ‘a’ shows a significant arching zone for the Mohr-Coulomb model up to $1D_{TD}$ from the trapdoor, with minimum vertical stress ratios of 0.03 and 0.5 occurring at $0D_{TD}$ and $1D_{TD}$, respectively. This value increases significantly to approximately 0.9 at $2D_{TD}$ from the base. There is minimal change in the vertical stress at distances greater than $3D_{TD}$ from the trapdoor.

The Hardening Soil model shows very similar quantitative behaviour to that described above, as is evident from Figure 6.23 ‘b’. The vertical stress ratio reduces to almost zero on the trapdoor, whilst at $1D_{TD}$ from the base, the value reaches a value of approximately 0.2 at a normalised trapdoor displacement of 0.75%, and subsequently increases to a final value of 0.3. The arching effect continues to reduce considerably as the distance from the trapdoor increases.

The variation in the horizontal stress obtained from the Mohr-Coulomb model is presented in Figure 6.23 ‘c’. The ratio at the trapdoor decreases to almost zero, and subsequently increases to a final value of 0.2. The changes at the other distances presented in sub-figure ‘c’ are very small, where an immediate increase on trapdoor displacement occurs. The horizontal stress ratio at $1D_{TD}$ increases to approximately 1.08, and then decreases to a minimum value. The increase in the horizontal stress ratio decreases as the distance from the trapdoor increases.

The horizontal stress ratio at the trapdoor obtained from the Hardening Soil model, as shown in Figure 6.23 ‘d’, is very similar to that obtained from the Mohr-Coulomb model. A decrease to approximately 0.08 occurs, followed by an increase to a final value of 0.3. At $1D_{TD}$ from the trapdoor, an increase to approximately 1.1 is evident followed by a decrease to a constant value of 1.05. For distances from the trapdoor greater than $1D_{TD}$, the horizontal stress ratio increases to a maximum after initial trapdoor displacement, the magnitude of which decreases with increasing distance from the base.
Chapter 6 Numerical investigation

Figure 6.23 Stress ratios directly above centreline at various distances from test chamber base for (a) (c) Mohr-Coulomb model (b) (d) Hardening Soil model
6.5.3.3 Behaviour adjacent to trapdoor centreline

The vertical and horizontal stress ratios at $0.7D_{TD}$ from the trapdoor centreline obtained from the numerical analysis of an active trapdoor at various distances from the base of the test chamber using the Mohr-Coulomb and Hardening Soil models are presented in Figure 6.24.

Figure 6.24 sub-figure 'a' illustrates the results of the vertical stress ratios obtained from the Mohr-Coulomb model. At the base of the test chamber, there is an initial increase in vertical stress ratio to a maximum after a trapdoor displacement of 0.15%, before decreasing sharply to a minimum value of 0.56 at a displacement of 2%. It then increases gradually with continuing trapdoor displacement, and appears to be approaching an ultimate value at a displacement of 20%. At $0.5D_{TD}$ from the base, there is a sharp increase in the vertical stress ratio to approximately 1.14, which then continues to increase slightly to a final value of 1.24. There is marginal variation in the vertical stresses at $1D_{TD}$ and $2D_{TD}$ from the base of the test chamber, the stress ratio at these points decreasing to a constant value of approximately 0.95 on trapdoor displacement of 0.2%.

The changes in the vertical stress obtained from the Hardening Soil model are presented in Figure 6.24 sub-figure ‘b’. The vertical stress ratio at the base of the test chamber increases significantly to a final value of 6.0. The inset in this sub-figure shows the vertical stress ratios at the other distances from the base in more detail. After decreasing on initial trapdoor displacement, the ratio at $0.5D_{TD}$ then increases on continuing trapdoor yielding. At $1D_{TD}$ and $2D_{TD}$ from the base, the vertical stress decreases on trapdoor yielding, to values of 0.85 and 0.92, respectively. The stress at $1D_{TD}$ then increases gradually on continued trapdoor yielding to a final value of approximately 0.91, whilst that at $2D_{TD}$ remains relatively constant.

The changes in the horizontal stresses using the Mohr-Coulomb model are shown in Figure 6.24 sub-figure ‘c’. A sharp increase on initial trapdoor yielding is evident at the base of the test chamber. This continues to increase with trapdoor displacement, reaching a final value of approximately 4.5. The horizontal stress ratio at $0.5D_{TD}$ decreases sharply, and following a small increase, subsequently decreases to a minimum of approximately 0.84. At $1D_{TD}$, the horizontal stress ratio decreases to a constant minimum of 0.82 on initial trapdoor displacement. There is no arching at a distance from the base of $2D_{TD}$.
The horizontal stresses at the base of the trapdoor using the Hardening Soil model show an increase with trapdoor displacement to a final value of 4.2, as illustrated in Figure 6.24 sub-figure ‘d’. At $0.5D_{TD}$ from the base, the magnitude of the horizontal stress ratio is small but highly variable, with minimum and maximum values of 0.97 and 1.11, respectively. The stress ratio is greater at $2D_{TD}$ than at $1D_{TD}$, as maximum values of 1.12 and 1.05 are reached on initial trapdoor yielding, respectively.

The vertical stress ratios at $1.2D_{TD}$ from the trapdoor centreline obtained from the Mohr-Coulomb model (shown in Figure 6.25 sub-figure ‘a’) show a very small increase from the initial value of 1.0. As the distance from the base of the test chamber increases, there is a corresponding decrease in the maximum vertical stress ratio. There is an insignificant increase at $2D_{TD}$ on initial trapdoor yielding. After the maximum values of 1.037 and 1.034 are reached at $0D_{TD}$ and $0.5D_{TD}$ respectively, there is a subsequent decrease to final values of approximately 1.027 and 1.025, respectively. This is in contrast to the behaviour at $1D_{TD}$, which reaches a constant maximum value of 1.028 after initial trapdoor yielding.

Very similar patterns in the vertical stress ratios to the Mohr-Coulomb model are found using the Hardening Soil model, illustrated in Figure 6.25 sub-figure ‘b’. The main differences between the behaviour observed are that the maximum values are higher for the latter model and the vertical stress at $1D_{TD}$ decreases gradually after reaching the maximum value, rather than reaching a constant value. It is not clear as to why there is a sudden variation in both the vertical and horizontal stress ratios at a trapdoor displacement of approximately 14.5%.
Figure 6.24 Stress ratios 0.7D₁₀ from trapdoor centreline at various distances from base for (a) (c) Mohr-Coulomb model (b) (d) Hardening Soil model
Figure 6.25 Stress ratios $1.2D_{TD}$ from trapdoor centreline at various distances from base for (a) (c) Mohr-Coulomb model (b) (d) Hardening Soil model
The horizontal stresses at $0D_{TD}$ and $0.5D_{TD}$ using the Mohr-Coulomb model show a small increase from the initial value, whilst a small decrease occurs at $1D_{TD}$ and $2D_{TD}$ (Figure 6.25 sub-figure 'c'). A maximum value of 1.04 on initial trapdoor displacement is reached at the base, which decreases marginally to a final value. After increasing gradually with trapdoor displacement, a final value of 1.07 is reached at $0.5D_{TD}$. The horizontal stress at $1D_{f}$ from the crown increases gradually with continuing trapdoor displacement to a final value of approximately 0.97, after an initial decrease to a minimum. The ratio at $2D_{f}$ from the crown decreases on initial trapdoor yielding to an ultimate minimum value of approximately 0.97.

The behaviour in the horizontal stresses at $0D_{TD}$ and $0.5D_{TD}$ from the base of the test chamber using the Hardening Soil model are also very similar to those observed in the Mohr-Coulomb model, as shown in Figure 6.25 sub-figure 'd', although the magnitude of the arching effect is greater in the former. The behaviour at $1D_{TD}$ and $2D_{TD}$ is different to that predicted using the Mohr-Coulomb model. The ratio at $1D_{TD}$ increases with trapdoor displacement to a final value of approximately 1.08, while that at $2D_{TD}$ reaches a maximum constant value of 1.02 on initial trapdoor yielding.

6.5.4 Discussion of results of detailed analysis

6.5.4.1 General behaviour

The vertical stress ratios obtained from both soil models in the region of sand directly above the trapdoor centreline were generally as expected. At all points, a decrease to a minimum on initial trapdoor displacement was observed. The magnitude of the vertical stress ratio increased as the distance from the trapdoor increased, and greater arching effects were observed using the Hardening Soil model. This model was able to capture very accurately the gradual increase after the minimum value at $1D_{TD}$ from the base which was also observed in the physical test results presented in Figure 4.5 and discussed in § 4.4.3.3.

A reduction in the horizontal stress ratios was observed on the trapdoor itself. However, there was an increase at $1D_{TD}$ from the initial value. The peak in the ratio observed at this point after initial trapdoor displacement (Figure 6.23 sub-figures 'c' and 'd') may be due to the development of the arch above the trapdoor, and the subsequent reduction due to the
fact that the arch failed due to the continued yielding of the trapdoor, as discussed by Sadrekarimi and Abbasnejad (2010). As the vertical distance increases from $1D_{TD}$ above the base, the magnitude of the positive arching effect with respect to horizontal stress decreases.

Significant variations in the horizontal and vertical stress ratios were observed at $0.7D_{TD}$ from the trapdoor centreline (Figure 6.24). For example, at the base using the Hardening Soil model, vertical and horizontal stress ratios of 6.0 and 4.5, respectively, were found. These values, coupled with the seemingly erratic behaviour at $0.5D_{TD}$ but not at greater heights, suggest that there may be numerical errors in this area due to the proximity to the transition from the fixity beside the prescribed displacement to the prescribed displacement itself. At $1D_{TD}$ and $2D_{TD}$ from the base, contrasting horizontal stress ratios were obtained from the two models. The Mohr-Coulomb model shows a reduction to 0.85 at $1D_{TD}$ and no arching at $2D_{TD}$, whilst the Hardening Soil model shows a small increase at both points, with the greater of the increases at $2D_{TD}$.

Small variations in the stress ratios were observed at $1.2D_{TD}$ from the trapdoor centreline, although greater arching was observed in the horizontal stresses than the vertical stresses. The most significant arching was concentrated at the base of the model. Contrasting results were predicted from the two models in the horizontal stresses at $1D_{f}$ and $2D_{f}$ from the base. Using the Mohr-Coulomb model, negative arching to ultimate values of approximately 0.98 were predicted, whereas using the Hardening Soil model, positive arching to ultimate values of approximately 1.04 were predicted.

6.5.4.2 Comparison of results obtained from soil models

The patterns of vertical stress transfer obtained from the Mohr-Coulomb and Hardening Soil models were broadly similar. The exception to this was at $0.7D_{TD}$ from the centreline, where erratic results were obtained from both models. The magnitude of the arching effect (both positive and negative) was greatest in the results obtained from the Hardening Soil model.

The patterns in the horizontal stress ratios obtained from the two models showed considerable differences. This was highlighted in Figure 6.25 sub-figures ‘c’ and ‘d’, where the Mohr-Coulomb model predicted a decrease in the stress ratios at $1D_{f}$ and $2D_{f}$ from the crown, whereas the Hardening Soil model predicted the opposite effect.
6.5.4.3 Coefficient of lateral earth pressure, $K_{ep}$

The difficulty in choosing an appropriate value of $K_{ep}$ and the potential use of FE analyses in doing so was discussed in § 4.3.2.4. The values obtained from the analyses described above are presented in Figure 6.26. It is evident that $K$ at the trapdoor varies considerably with the degree of trapdoor yielding. The magnitude of $K$ decreases with increasing distance from the trapdoor. From a comparison of the values obtained from the two soil models, it is clear that there are significant differences, with those obtained from the Hardening Soil model significantly larger than those from the Mohr-Coulomb model. It can be concluded that the appropriate value of $K$ to use in the theoretical models is not obvious from the results of the FE analyses. It should be noted that the significant variability in the $K$ value at the base of the model using the Hardening Soil model is as a result of the very small vertical stresses recorded at this point at trapdoor displacements of 0-2% and 15%, as shown in Figure 6.23 sub-figure ‘b’.

![Figure 6.26 Coefficient of lateral stress above active trapdoor from FE analyses for (a) Mohr-Coulomb model and (b) Hardening Soil model](image-url)
6.5.4.4 Comparison of numerical and laboratory results

The purpose of this section is to compare the numerical results obtained using the MohrCoulomb and Hardening Soil models with the experimental results presented in Chapter 4. As only the vertical stresses were measured in the laboratory tests, the comparison is restricted to vertical stress ratios only. The results directly above the centreline, at $0.7D_{TD}$ and at $1.2D_{TD}$ from the centreline are compared in Figure 6.27, Figure 6.28 and Figure 6.29, respectively.

The stress acting on the trapdoor is underpredicted by both soil models, as shown in Figure 6.27 sub-figure ‘a’. A sharper decrease on initial trapdoor displacement is also clear from the numerical results than was measured in the laboratory tests. Although the rate of reduction is poorly predicted, sub-figure ‘b’ shows the reasonable prediction of the final vertical stress ratio by the Hardening Soil model at $1D_{TD}$ from the base. At $3D_{TD}$ and $4D_{TD}$ from the base, both models significantly underpredict the extent of the arching effect (sub-figures ‘c’ and ‘d’).

The Hardening Soil model predicts reasonably the vertical stress ratio acting at $0.7D_{TD}$ from the trapdoor centreline at a height of $1D_{TD}$ from the base (Figure 6.28 sub-figure ‘a’), although the rate of reduction is poorly predicted. The Mohr-Coulomb underpredicts the arching effect at this position. As shown in Figure 6.28 sub-figure ‘b’, the vertical stress ratio is overpredicted by both soil models at a height of $2D_{TD}$ from the base. It should be noted that due to the inconsistencies in the numerical results at $0.5D_{TD}$ from the base (as discussed in § 6.5.4.1), the comparison is limited to the ratios at $1D_{f}$ and $2D_{f}$.

It can be seen from Figure 6.29 that at $1.2D_{TD}$ from the centreline, both soil models provide a reasonable prediction of the final vertical stress ratio at distances of $0.5D_{TD}$ and $1D_{TD}$ from the base. At $2D_{TD}$ from the base, the magnitude of the arching effect is significantly underpredicted. However, generally speaking, the slopes of the curves are poorly predicted.
Figure 6.27 Comparison of numerical results and test CTD003 directly above trapdoor at various distances from base: (a) 0D_{TD} (b) 1D_{TD} (c) 3D_{TD} (d) 4D_{TD}
Figure 6.28 Comparison of numerical results and test CTD005 at 0.7D_TD from trapdoor centreline at various distances from base: (a) 1D_TD (b) 2D_TD
Figure 6.29 Comparison of numerical results and test CTD005 at 1.2D_TD from trapdoor centreline at various distances from base: (a) 0.5D_TD (b) 1D_TD (c) 2D_TD
6.6 2D numerical modelling of MTBM tests

6.6.1 Introduction
The purpose of this section is to present and discuss the results of the 2D FE analysis carried out to investigate the arching effect caused by tunnel construction. Although the 3D nature of the arching effect has been highlighted, the 2D analysis is carried out to identify stress-transfer patterns for this simplification of the laboratory tests, prior to the more complex 3D analysis.

6.6.2 Preliminary analysis
A preliminary 2D FE analysis was carried out on the tunnel construction problem to determine the influence of the mesh coarseness on the results and to obtain the most suitable model configuration for further analyses. As the test chamber used in the laboratory tests was symmetrical, only half of it was modelled. Therefore, the dimensions used were 0.625m x 1.25m (width x height). The bottom and vertical boundaries were fixed in the horizontal and vertical directions. The contraction method was used to simulate the volume loss due to the construction of the tunnel. This contraction is applied to the tunnel lining which simulates a reduction in the cross-sectional area of the tunnel. It is expressed as the ratio of the reduction in the area to the original tunnel area. All soil model parameters were obtained from the relationships presented in § 6.3. A screenshot of the model used in the analysis is shown in Figure 6.30.

Figure 6.30 Screenshot of 2D FE MTBM model
The effect of three different mesh configurations (fine, medium, and coarse, as discussed in § 6.5.2.1) was investigated by comparing the vertical and horizontal stresses acting at a height of $1D_f$ above the tunnel crown, where $D_f$ is the diameter of the tunnel face. Table 6.10 presents a summary of the tests carried out to establish the effect of this parameter.

<table>
<thead>
<tr>
<th>Soil model</th>
<th>Test name</th>
<th>Mesh coarseness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mohr-Coulomb</td>
<td>FE-TBMCAL-001</td>
<td>Fine</td>
</tr>
<tr>
<td></td>
<td>FE-TBMCAL-002</td>
<td>Medium</td>
</tr>
<tr>
<td></td>
<td>FE-TBMCAL-003</td>
<td>Coarse</td>
</tr>
<tr>
<td>Hardening Soil</td>
<td>FE-TBMCAL-004</td>
<td>Fine</td>
</tr>
<tr>
<td></td>
<td>FE-TBMCAL-005</td>
<td>Medium</td>
</tr>
<tr>
<td></td>
<td>FE-TBMCAL-006</td>
<td>Coarse</td>
</tr>
</tbody>
</table>

Table 6.10 Summary of 2D FE MTBM tests carried out to establish effect of mesh coarseness

6.6.2.1 Effect of mesh coarseness

The results of tests FE-TBMCAL-001 to FE-TBMCAL-003 are presented in Figure 6.31. It is evident from sub-figure 'a' that the three mesh configurations provide very similar vertical stress ratios at $1D_f$ above the tunnel crown. The fine to coarse meshes provide the lowest to highest ratios, respectively. The horizontal stress ratios at $1D_f$ above the tunnel crown (sub-figure 'b') show that the fine and medium configurations provide almost the same ratios. The results from the coarse mesh differ significantly from these. As large deformations were encountered in the modelling of the MTBM tests proper, and considering calculation times were not significantly increased by a more accurate mesh while using the Mohr-Coulomb model, a fine mesh was used for all further analyses involving this soil model.
Figure 6.31 Effect of mesh coarsenes for Mohr-Coulomb model at IDf above tunnel crown: (a) vertical stress ratio (b) horizontal stress ratio

The results of tests FE-TBMCAL-004 to FE-TBMCAL-006 are presented in Figure 6.32. It is evident from sub-figure ‘a’ that the three mesh configurations provide very similar vertical stress ratios. The fine to coarse mesh show the lowest to highest ratios, respectively. The same trend may be observed in the horizontal stress ratios in sub-figure ‘b’, that is, an increase in the ratio with the mesh coarseness. Despite the fact that calculation times are increased by a less coarse mesh with the Hardening Soil model, a fine mesh was used for all further analyses involving this soil model due to the large deformations which were encountered in the modelling of the MTBM tests proper.
6.6.3 Presentation of results of detailed analysis

The stress transfer mechanisms around the tunnel were investigated by examining the variation in the vertical and horizontal stress ratios at various points above and adjacent to it, the exact positions of which are presented in Table 6.11. The stress transfer was caused by an applied tunnel contraction of 20%, which is significantly lower than the values measured during the laboratory tests and presented in Table 5.5. As the behaviour and extent of the arching zone is dependent on the degree of volume loss and as the contraction used in the numerical analysis is considerably smaller than the volume losses measured in the laboratory tests, a direct comparison between the FE results and laboratory results is
not possible. The purpose of the 2D analyses was therefore to investigate generally the stress transfer patterns occurring due to the volume loss at the tunnel.

<table>
<thead>
<tr>
<th>Horizontal distance from tunnel centreline [D_t]</th>
<th>Vertical distance from tunnel crown [D_t]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>2</td>
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<tr>
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<td>3</td>
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<td></td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>2</td>
</tr>
</tbody>
</table>

Table 6.11 Positions at which stresses were examined for purposes of 2D FE MTBM analyses

6.6.3.1 Behaviour directly above tunnel centreline

The vertical and horizontal stress ratios obtained from the numerical analysis at various distances from the tunnel crown directly above the tunnel centreline are presented in Figure 6.33.

Although there are differences in the magnitude of the vertical stress ratios between the Mohr-Coulomb and Hardening Soil models (as presented in Figure 6.33 sub-figure ‘a’ and ‘b’, respectively), the qualitative behaviour is very similar. At all heights directly above the tunnel centreline there is a decrease in the vertical stress. The magnitude of the arching effect decreases with increasing distance from the tunnel crown, and increases with tunnel contraction. The Hardening Soil model predicts a greater arching effect than the Mohr-Coulomb model, for example, the final vertical stress ratios at 1D_t from the tunnel crown using the former is 0.11, compared to 0.21 using the latter.

At 1D_t from the tunnel crown, the horizontal stress ratio obtained from the Mohr-Coulomb model shows an initial increase to a maximum value of 1.2 after a tunnel contraction of 0.75%, before decreasing with increasing tunnel contraction to a constant value of 1.1 after a contraction of 12%, as illustrated in Figure 6.33 sub-figure ‘c’. The remaining distances above the crown show a gradual increase to a maximum constant value. The magnitude of this value decreases as the distance from the crown increases.
Figure 6.33 Stress ratios directly above tunnel centreline at various distances from crown for (a) Mohr-Coulomb model (b) Hardening Soil model
The behaviour obtained from the Hardening Soil model (Figure 6.33 sub-figure ‘d’) is considerably different. In the same way as was observed for the Mohr-Coulomb model, the horizontal stresses at $1D_t$ from the crown increase to a maximum of 1.33 after a contraction of approximately 3.5% before decreasing slowly with further contraction to the initial value. However, the remaining points above the crown are subjected to an increase in horizontal stress ratio, the magnitude of which increases with the distance from the tunnel crown.

### 6.6.3.2 Behaviour adjacent to tunnel centreline

The stress ratios obtained from the 2D FE analysis at various distances above the tunnel crown at $1D_t$ from the tunnel centreline are presented in Figure 6.34. The vertical stress ratios at $1D_t$ from the tunnel centreline using the Mohr-Coulomb model are shown in Figure 6.34 sub-figure ‘a’. On initial tunnel contraction, there is an increase in the vertical stresses acting at $-0.5D_t$ and $0D_t$ from the tunnel crown, followed by a subsequent decrease to a minimum value. The magnitude of the arching effect is greater at the former point than at the latter. At $1D_t$ and $2D_t$ above the crown, there is a gradual decrease to a minimum ultimate value. The magnitude of the final vertical stress ratio increases with increasing distance from the springline of the tunnel, where the springline is the horizontal line through the centre of the tunnel cross-section. The results obtained from the Hardening Soil model are quantitatively very similar to those obtained from the Mohr-Coulomb model, as shown in Figure 6.34 sub-figure ‘b’. Both the maximum and minimum values are smaller in magnitude than those predicted using the Mohr-Coulomb model.

On initial tunnel contraction, the horizontal stress ratios at $-0.5D_t$ and $0D_t$ from the crown show significant variation, as they alternate from ratios below to above the initial value (Figure 6.34 sub-figure ‘c’). After a tunnel contraction greater than 2%, the ratios decrease gradually to final values of approximately 0.28 and 0.70, respectively. At $1D_t$ from the crown, there is a decrease to a minimum value of 0.8 after a tunnel contraction of approximately 1.5%, before increasing gradually to a final value of 0.9. A small increase to a constant value may be observed at $2D_t$. The magnitude of the final horizontal stress ratio increases as the distance from the springline increases.
Figure 6.34 Stress ratios at $1D_c$ from tunnel centreline at various distances from crown for (a) (c) Mohr-Coulomb model (b) (d) Hardening Soil model
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The behaviour of the horizontal stress ratio using the Hardening Soil model (Figure 6.34 sub-figure ‘d’) differs from that described above. The horizontal stress ratios at the springline and crown decrease to values of 0.2 and 0.54, respectively. The ratio at 1Df, after a very small initial decrease, subsequently increases to a maximum value of 1.2 before decreasing to a final value of 1.03. The ratio at 2Df increases gradually to a constant final value of 1.27.

Figure 6.35 presents the vertical and horizontal stress ratios obtained from the numerical analysis at various distances from the tunnel crown at 2Df from the tunnel centreline. The vertical stress ratios from the Mohr-Coulomb model are presented in sub-figure ‘a’. At all points, there is a gradual increase to a constant maximum value. The magnitude of this value is the same at both springline and crown level, before decreasing with increasing distance from the crown.

The vertical stress ratios from the Hardening Soil model (Figure 6.35 sub-figure ‘b’) are relatively similar to those obtained from the Mohr-Coulomb model. The final values at -0.5Df and 1Df using the two models are approximately the same, but the maximum values are reached after a smaller amount of tunnel contraction than for the Mohr-Coulomb model. A smaller increase to approximately 1.1 is observed at 1Df when using the Hardening Soil model. At 2Df, no significant arching is observed.

The horizontal stress ratio at springline and crown level increases to constant maximum values of 1.27 and 1.19, respectively, while those at 1Df and 2Df from the crown decrease to a minimum constant value of 0.89 and 0.83, as shown in Figure 6.35 sub-figure ‘c’. Different behaviour may be observed from the results of the Hardening Soil model (Figure 6.35 sub-figure ‘d’). The horizontal stress ratios at -0.5Df and 0Df from the tunnel crown increase to approximately the same maximum value of 1.3, before decreasing gradually to a final value of 1.1. The ratios at 1Df and 2Df from the crown increase gradually to final values of approximately 1.15 and 1.1, respectively, after a very small decrease on initial tunnel contraction.
Figure 6.35 Stress ratios at 2Df from tunnel centreline at various distances from crown for (a) (c) Mohr-Coulomb model (b) (d) Hardening Soil model
For distances from the tunnel centreline greater than $2D_f$, the stress ratios predicted by both the Mohr-Coulomb and Hardening Soil model are qualitatively the same as that of the Hardening Soil model at $2D_f$ from the tunnel centreline presented in Figure 6.35. Therefore, these results are not presented. The extent of the arching effect decreases as the distance from the centreline increases. This is discussed further in the following section.

6.6.4 Discussion of results of detailed analysis

6.6.4.1 General behaviour

In order to investigate the behaviour of the arching zone as a whole, the maximum and minimum vertical and horizontal stress ratios obtained from the two soil models are illustrated in Figure 6.36 and Figure 6.37. These figures contain the collated data at all points presented in Table 6.9 from the 2D numerical analysis of the MTBM problem.

The minimum vertical stress ratios obtained from the Mohr-Coulomb model are shown in Figure 6.36 sub-figure ‘a’. Directly above the centreline, the minimum ratio increases exponentially as the distance from the crown increases. At $1D_f$ from the centreline, the vertical stress ratio increases approximately linearly with distance from the springline, although the values at the tunnel crown and at $1D_f$ are very similar. There is no significant reduction in the vertical stress at distances from the tunnel centreline greater than $2D_f$, thereby suggesting that the extent of the negative arching zone with regard to vertical stresses lies between $1D_f$ and $2D_f$ from the centreline.

The reduction in the vertical stresses obtained from the Hardening Soil model (Figure 6.36 sub-figure ‘b’) show very similar trends to those observed using the Mohr-Coulomb model. The only exception to this is at $2D_f$ from the tunnel centreline at $2D_f$ from the crown, which shows a marginal reduction from the initial value of 1.0. Quantitatively, a more significant arching effect is observed from the results of the Hardening Soil model, i.e. the minimum vertical stress ratios are lower than those obtained from the Mohr-Coulomb model.
Figure 6.36 Vertical stress ratios at various distances from tunnel centreline for (a) (c) Mohr-Coulomb model and (b) (d) Hardening Soil model
Figure 6.37 Horizontal stress ratios at various distances from tunnel centreline for (a) (c) Mohr-Coulomb model and (b) (d) Hardening Soil model
It is evident from Figure 6.36 sub-figure ‘c’, which shows the maximum vertical stress ratio at various distances from the tunnel centreline obtained from the Mohr-Coulomb model, that there is no increase in the ratio acting above the tunnel centreline. The points at the springline and at the crown at a distance of $1D_f$ from the centreline show a very significant increase of up to 1.63. For heights along this plane greater than $1D_f$ from the crown, there is no increase from the initial value. At $2D_f$ from the centreline, there is also a significant increase at the springline and at the crown, which decreases to a value of approximately 1.07 at $2D_f$ from the crown. Similar trends at $3D_f$ and $4D_f$ from the centreline may be observed: the $(\sigma_v/\sigma_{c,v})_{\max}$ values from -0.5$D_f$ to $1D_f$ are approximately the same, followed by a slight decrease at $2D_f$.

The Hardening Soil model (Figure 6.36 sub-figure ‘d’) shows no increase in the vertical stress ratio above the initial value for all points directly above the tunnel centreline. The ratio decreases exponentially from the tunnel springline at $1D_f$ from the centreline. The most significant increase occurs at the springline and the crown at $2D_f$ from the centreline, before decreasing to approximately the initial value at $2D_f$ from the crown. Significant arching occurs at $3D_f$ and $4D_f$ from the centreline, although it is to a lesser extent than that experienced at $2D_f$. At heights of $2D_f$ from the crown, there is still a significant increase in the vertical stress ratio at $3D_f$ and $4D_f$ from the centreline.

The minimum horizontal stress ratios obtained from the Mohr-Coulomb model are shown in Figure 6.37 sub-figure ‘a’. There is no reduction from the initial value directly above the centreline. At $1D_f$ from the centreline, the minimum ratio increases exponentially from 0.26 at the springline to the initial value at $2D_f$ from the tunnel crown. There is a small decrease at the springline and crown at $2D_f$ from the centreline, followed by a linear decrease at distances above the crown. The reduction from the springline to $1D_f$ above the tunnel crown at $3D_f$ from the centreline is very small, but a decrease to 0.9 at $2D_f$ from the crown is evident from the figure. There is an insignificant reduction at $4D_f$ from the centreline.

Much less variability in the minimum horizontal stress ratios obtained from the Hardening Soil in comparison to those obtained from the Mohr-Coulomb model is evident in Figure 6.37 sub-figure ‘b’. The most significant reduction occurs at $1D_f$ from the centreline, where a linear increase from 0.17 at the springline to just below 1.0 at $1D_f$ from the crown is
evident. There is also a very small decrease at 1\,D_f from the crown at distances of 1\,D_f and 2\,D_f from the centreline.

The maximum horizontal stress ratio directly above the tunnel centreline from the Mohr-Coulomb model is observed at 2\,D_f from the crown, as shown in Figure 6.37 sub-figure ‘c’. Even at distances of 6\,D_f from the crown, there is a significant maximum horizontal stress ratio of 1.17. The maximum ratios observed at 1\,D_f from the centreline are smaller, with a decrease from the springline to 1\,D_f above the crown, before a subsequent increase at 2\,D_f. The maximum ratios then increase at 2\,D_f from the centreline, where a linear decrease from 1.3 at springline level to the initial value at 1\,D_f from the crown is evident from the figure. The magnitude of the ratios then decrease with increasing distance from the tunnel centreline and follow a similar trend as those observed at 2\,D_f from the centreline.

The maximum horizontal stress ratios obtained using the Hardening Soil model are shown in Figure 6.37 sub-figure ‘d’. The magnitude of the maximum horizontal stress ratio increases above the tunnel centreline with increasing distance from the crown. A similar trend may be observed at 1\,D_f from the centreline. Although there is no increase from the initial at the springline and crown, there is a linear increase above the crown. The maximum stress ratios at springline and crown level at 2\,D_f from the centreline are approximately equal at 1.34, and this value decreases linearly above the crown level. Similar patterns are observed at 3\,D_f and 4\,D_f from the centreline. Following a slight increase in the maximum ratios from the springline to the crown, the ratios then decrease linearly with increasing height from the crown. The magnitude of the arching effect is greater at crown level at 3\,D_f from the centreline, but at greater heights, the maximum ratio is greater at 4\,D_f from the centreline.

6.6.4.2 Development of arch

The predicted development of the arch directly above the tunnel centreline may be observed from the horizontal stress ratios presented in Figure 6.33 sub-figures ‘c’ and ‘d’. Although observed from the results of both the Mohr-Coulomb and Hardening Soil models, it is explained here with reference to the Mohr-Coulomb model (i.e. Figure 6.33 sub-figure ‘c’). The horizontal stress ratio at 1\,D_f above the crown increases with tunnel contraction to a peak at a tunnel contraction of 0.75%. This corresponds to the development of the arch. As tunnel contraction continues, failure of the arch occurs,
leading to the subsequent reduction in the horizontal stress ratio at this point. This is similar to the development of the arch discussed in § 6.5.4.1 in relation to the FE modelling of an active trapdoor.

6.6.4.3 Comparison of results obtained from soil models

The patterns of vertical stress ratio obtained from the Mohr-Coulomb and Hardening Soil models were broadly similar, although the extents of the arching zones varied between the two. The largest vertical stress positive arching zone was located at 1D_f and 2D_f from the centreline using the Mohr-Coulomb and Hardening Soil models, respectively. The Mohr-Coulomb and Hardening Soil models predicted the largest arching effects in the positive and negative zones, respectively.

There were considerable differences between the horizontal stress results obtained from the two models. For example, the Mohr-Coulomb model predicted a reduction in the positive arching effect above the tunnel centreline as the distance from the crown increases, in contrast to the Hardening Soil model which predicted the opposite effect. The Hardening Soil model predicted greater magnitudes of arching in both the positive and negative arching zones.

6.7 3D numerical modelling of MTBM tests

6.7.1 Introduction

Due to the nature of the arching effect caused by the advancement of the MTBM, a 3D FE study of the problem has been undertaken. The use of 3D FE analyses is associated with high computing times. In order to keep this to an acceptable length, only the Mohr-Coulomb model was used for the purposes of this section. For the same reason, a coarse mesh was used. The configuration of the 3D FE model was the same as that for the 2D analyses, except it extended to a distance of 0.8m in the longitudinal direction and a coarse mesh was used, a screenshot of which is shown in Figure 6.38.
The stress transfer mechanisms were investigated by examining the variation in the vertical and horizontal stress ratios at points in a vertical plane 0.4m from the front of the model perpendicular to the direction of progression of the MTBM. This is referred to as the plane of measurement throughout this section. The positions of these points are presented in Table 6.12.

<table>
<thead>
<tr>
<th>Horizontal distance from tunnel centreline ([D_x])</th>
<th>Vertical distance from tunnel crown ([D_y])</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>4</td>
</tr>
<tr>
<td>1, 2, 3</td>
<td>-0.5</td>
</tr>
<tr>
<td></td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>2</td>
</tr>
</tbody>
</table>

*Table 6.12 Positions at which stresses were examined for purposes of 3D FE MTBM analyses*

The application of the tunnel contraction, which is used in the FE software to simulate the soil movements around a tunnel, differed in the 3D analysis from the 2D. As stated in § 3.4.3.2, the tapered section of the tunnel lining has a length of 100mm and a reduction in area corresponding to 8.88% of the cross-sectional area of the tunnel face. Therefore, this value was applied as a contraction at a distance of 100mm from the tunnel face. In order to
match the contraction value of 20% used in the 2D analyses, the remaining contraction (11.12%) was applied at the cutting face. In this way, a total contraction of 20% was applied to the tunnel lining.

6.7.2 Presentation of results

6.7.2.1 Behaviour directly above tunnel centreline

The vertical stress ratios obtained using the Mohr-Coulomb model are presented in Figure 6.39 sub-figure ‘a’. As the MTBM begins to progress towards the plane of measurement, a slight increase in the stress ratio is evident. This increase is marginally largest at 2Df from the crown, whilst it is approximately the same at 1Df and 4Df. When the face is 1Df from the plane of measurement, there is a decrease in the vertical stress ratio. This continues at 1Df from the tunnel crown, where the arching effect is greatest, until the cutting face has advanced a distance of approximately 1Df past the plane of measurement. The reduction experienced at 2Df and 4Df from the crown is less significant, and the point at which the reduction stops appears to increase with distance from the crown. There is a slight increase in the vertical stress ratio after the minimum value has been reached at 1Df and 2Df from the crown, but not at 4Df.

Figure 6.39 sub-figure ‘b’ shows the variation in the horizontal stress ratios obtained from the 3D FE analysis. At 1Df from the crown, the ratio begins to decrease gradually immediately upon commencement of MTBM movement. A minimum value is reached when the face is 0.5Df away, before the ratio increases again when the face is at 0Df. An ultimate value of 0.82 is reached when the face has travelled 2.5Df past the plane of measurement. At 2Df from the tunnel crown, the horizontal stresses decrease slightly to a minimum value at 2Df, before increasing again at approximately 0.5Df. In similar fashion to the horizontal stresses at 1Df from the crown, an ultimate value of 1.1 is reached at a distance from the face of -2.5Df. The ratio at 4Df from the tunnel crown increases gradually at a distance of 3Df before reaching an ultimate value of 1.08, also at a distance from the face of -2.5Df.
Figure 6.39 Stress ratios directly above tunnel centreline from 3D FE analysis: (a) vertical (b) horizontal

6.7.2.2 Behaviour adjacent to tunnel centreline

The stress ratios at 1Df and 2Df from the tunnel centreline obtained from the 3D FE analysis are presented in Figure 6.40. Sub-figure ‘a’ shows the vertical stresses at 1Df from the tunnel centreline. At the springline level, the ratio decreases by a small amount until the MTBM face is 0.5Df from the plane of measurement. It subsequently peaks at a value of 1.6 when the face is at the plane, and decreases sharply to a minimum of 0.6 at a distance of -0.5Df, before a sharp increase to 0.92. Similar trends may be observed at the level of the tunnel crown, although the magnitude of the arching effect is less. The behaviour at 1Df from the tunnel crown is similar to that at 2Df. The ratios increase as the MTBM advances towards the plane of measurement, with a larger increase experienced at 1Df. At 0.75Df, the increase stops and the ratios begin to decrease to a minimum, which is
reached at a distance of -1.5D_f. The ratios then remain approximately constant, with final values of 0.85 and 0.78 at 1D_f and 2D_f from the tunnel crown, respectively.

The horizontal stress ratios at -0.5D_f and 0D_f from the tunnel crown at 1D_f from the tunnel centreline (Figure 6.40 sub-figure ‘b’) decrease marginally as the MTBM approaches the plane of measurement before increasing sharply at distances of 2.75D_f and 2.25D_f respectively, and reaching maximum values of 5.8 and 3.6 at 0D_f, respectively. The ratios then decrease sharply to minimum values when the face is 1.25D_f past the plane of measurement, before increasing gradually and reaching an ultimate value when the face has travelled 2.5D_f past the plane of measurement. At 1D_f and 2D_f from the tunnel crown, the horizontal stress ratios decrease gradually to minimum values of 0.8 and 0.9, respectively, when the cutting face is at the plane of measurement. They then increase slightly before reaching ultimate values at -2.5D_f of 0.9 and 0.95, respectively.

As the MTBM moves towards the plane of measurement, the vertical stress ratio at 2D_f from the tunnel centreline at all points increases gradually, as shown in Figure 6.40 sub-figure ‘c’. The magnitude of this increase decreases as the distance from the springline increases. The ratios at springline, crown level and 1D_f above the crown continue to increase until the face has progressed a distance of 2.5D_f past the plane of measurement, where an ultimate value is reached. When the face is approximately 1D_f from the plane of measurement, the ratio at 2D_f from the crown begins to decrease gradually. Any variation at this point after the face has progressed 1D_f past the plane of measurement is insignificant. The magnitudes of the final ratios decrease as the distance from the springline increases.
Figure 6.40 Vertical and horizontal stress ratios at various heights from tunnel crown at: (a) (b) 1D, and (c) (d) 2D from tunnel centreline
A significant increase in the horizontal stress ratios at $2D_f$ from the tunnel centreline at the springline and the crown occurs when the MTBM face is $3.5D_f$ from the plane of measurement (Figure 6.40 sub-figure ‘d’). Maximum values of 1.78 and 1.6, respectively, are reached at the springline and crown. The ratios then decrease to a minimum at -1.5$D_f$, before increasing very slightly to an ultimate value at -2.5$D_f$. At $1D_f$ from the tunnel crown, there is a gradual increase on MTBM advancement to a maximum of 1.06 at $0.75D_f$. The ratio then decreases to a minimum ultimate value of 0.88 after the face of the MTBM has progressed a distance of 2.5$D_f$ past the plane of measurement. The horizontal stress ratio at $2D_f$ from the tunnel crown is constant until the MTBM is $3D_f$ away, at which point it decreases gradually to a minimum at -1.25$D_f$. Following a small increase from this minimum value, an ultimate value of 0.88 at -2.5$D_f$ is reached.

The results at $3D_f$ from the tunnel centreline are not presented, because the changes in both the vertical and horizontal stress ratios at this plane were quantitatively the same as those at $2D_f$ from the centreline. Qualitatively speaking, the arching effect was less pronounced at $3D_f$.

6.7.3 Discussion of results

6.7.3.1 General behaviour of arching zone

The stress-transfer behaviour of the zone to a lateral extent of $3D_f$ from the tunnel centreline and to a vertical extent of $4D_f$ above the tunnel crown is illustrated in Figure 6.41, which presents the maximum, minimum and final vertical stress ratios as defined in Figure 5.21. It should be noted that the maximum ratio refers to the highest value from the point where MTBM advancement begins until it has just passed the plane of measurement.
Figure 6.41 Vertical stress ratios in arching zone obtained from 3D FE analyses: (a) maximum (b) minimum (c) final
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The maximum vertical stress ratios are illustrated in Figure 6.41 sub-figure ‘a’. A very small increase in the vertical stress ratio is observed at all distances directly above the tunnel centreline, the magnitude of which increases slightly from 1Df to 2Df, before decreasing again at 4Df. The highest values are observed at 1Df from the tunnel centreline at the springline. This decreases exponentially as the distance from the springline increases. Much smaller values are evident at a distance of 2Df from the tunnel centreline. Maximum vertical stress ratios of approximately 1.08 are observed at the springline, crown and 1Df above the crown, before decreasing at 2Df above the crown. There is a small increase at 3Df from the tunnel centreline as the miniature MTBM advances, resulting in an approximate (σv/σvi)max value of 1.05 at all heights along this plane.

Figure 6.41 sub-figure ‘b’ presents the minimum vertical stress ratios (σv/σvi)min at various lateral distances from the tunnel centreline and vertical distances above the tunnel crown. The most significant arching is experienced directly above the centreline, and the magnitude of this decreases exponentially as the distance from the tunnel crown increases. The greatest arching effect at 1Df from the tunnel centreline occurs at the springline level. The (σv/σvi)min values then increase at both the tunnel crown and at 1Df from the tunnel crown, before decreasing at 2Df. At the springline at 2Df from the tunnel centreline, there is a slight reduction from the initial value to 0.98. At all other points along this plane, and at all points at 3Df from the tunnel centreline, the settlements caused by the MTBM do not cause a reduction in the vertical stress ratio.

A comparison of the final vertical stress ratios presented in Figure 6.41 sub-figure ‘c’ with the (σv/σvi)min values presented in sub-figure ‘b’ shows the positive arching effect caused by the progression of the MTBM past the plane of measurement. At distances from the tunnel centreline of 0Df to 2Df, there is a general trend of points closer to the tunnel crown experiencing a greater vertical stress ratio increase than those situated further from the crown. Points at the same height from the tunnel crown but at different distances from the centreline show a very similar magnitude of increase after the passing of the MTBM. At 3Df, the magnitude of increase is constant regardless of distance from the crown.

The maximum, minimum and final horizontal stress ratios are illustrated in Figure 6.42. Sub-figure ‘a’ illustrates the variation in the maximum ratio throughout the zone caused by the approach of the MTBM towards the plane of measurement. Directly above the tunnel
centreline, there is no increase in the horizontal stresses in the vicinity of the tunnel crown, and a marginal increase at 4D_f from the crown. At 1D_f from the centreline, the largest variation occurs at the springline and at the crown, with ratios of 5.8 and 3.6, respectively. The ratio then decreases to the initial at 1D_f from the crown. The ratios decrease almost linearly to zero from the springline to 2D_f above the crown situated 2D_f from the centreline. A similar trend is evident at 3D_f from the centreline. At heights of 1D_f from the crown, there is an increase in the ratio as the distance from the centreline increases. Below this, the ratio decreases with distance from the centreline. These points highlight the zone of increased maximum arching as the distance from the centreline and from the crown increase.

The reduction in the horizontal ratio is greatest directly above the centreline (sub-figure 'b'). The behaviour at this plane and at 1D_f from the centreline is similar, with the ratio increasing exponentially with increasing distance from the springline. There is no decrease at the springline and crown at 2D_f from the centreline, but a decrease to approximately 0.88 at both 1D_f and 2D_f from the crown. A similar trend is evident at 3D_f from the centreline: there is no decrease up to 1D_f from the crown, followed by an increase to 0.92 at 2D_f from the crown. This identifies the zone of negative arching which spreads vertically through the sand mass as the distance from the centreline increases.

From a comparison of sub-figures 'b' and 'c', it is evident that at the majority of points there is a significant increase from the minimum to final values. However, this is not the case at 2D_f from the centreline, where at 1D_f and 2D_f from the crown the minimum ratios are equal in magnitude to the final. There is also minimal increase at 3D_f from the centreline at 2D_f from the crown.
Figure 6.42 Horizontal stress ratios in arching zone obtained from 3D FE analyses: (a) maximum (b) minimum (c) final
6.7.3.2 Comparison with 2D with 3D FE analysis

As the results from the 2D and 3D numerical analyses are of a very different form, it is not possible to compare the entire data sets. However, it is possible to compare the minimum vertical and horizontal stress ratios, as illustrated in Figure 6.43. It should be noted that the first part of each series' name refers to whether the analysis is 2D or 3D, whilst the second part refers to the distance from the tunnel centreline.

It is evident from Figure 6.43 sub-figure ‘a’ that the minimum vertical stress ratio obtained from the 3D analysis is underpredicted by the 2D analysis directly over the tunnel centreline and at 1D_f from the centreline. The exception to this is at 2D_f from the centreline at 2D_f from the crown, where the same ratio is predicted using both methods. There is no reduction in the vertical stress ratio at 2D_f and 3D_f from either the 2D or 3D analyses.

Directly above the centreline, the 2D analysis overpredicts the minimum horizontal stress ratio obtained from the 3D analysis up to 2D_f from the tunnel crown, as shown in Figure 6.43 sub-figure ‘b’. It also underpredicts it at the springline and crown level at 1D_f from the centreline, matches it at 1D_f, and overpredicts it at 2D_f from the crown. Finally, the 2D analysis underpredicts the ratio obtained from the 3D analysis at both 2D_f and 3D_f from the centreline.
6.8 Summary of chapter

6.8.1 Soil models

An investigation of the ability of three soil models, namely the Mohr-Coulomb, Hardening Soil and Hypoplastic, to simulate the behaviour of the sand used in the laboratory tests discussed in Chapters 4 and 5 has been carried out. The theory of the models, the necessary soil parameters and the methods of obtaining them have been described. In order to calculate these parameters, a series of oedometer, triaxial and shear box tests were performed. The ability of the models to simulate the behaviour of the sand was then assessed by comparison of the results of the numerical simulation of the laboratory tests.
with the measured data. It was concluded from this that the Mohr-Coulomb and Hardening Soil models were the most suitable. These models were therefore used for the subsequent FE analyses carried out in order to investigate the patterns of stress-transfer caused by an active trapdoor and MTBM.

6.8.2 Numerical modelling of active trapdoor tests

Prior to the numerical modelling of the active trapdoor tests carried out in the laboratory, a series of preliminary tests were carried out in order to identify the most appropriate FE model with regard to the use of interface elements and degree of mesh coarseness. It was concluded from this that an interface should not be used, due to its influence on the shear strain around the trapdoor, and that a fine and medium mesh should be used for the Mohr-Coulomb and Hardening Soil models, respectively.

An active trapdoor test carried out in the laboratory was modelled numerically using both the Mohr-Coulomb and Hardening Soil models in order to study the arching effects at the base of the model and throughout the sand mass, by studying the variation in the horizontal and vertical stress ratios. The trends of active and positive arching were identified and quantified. Erratic stress ratios were identified close to the base of the model at $0.7D_{TD}$ from the trapdoor centreline. It was concluded that these arose due to the transition from the fixity beside the prescribed displacement to the prescribed displacement itself.

Although the vertical stress ratio results obtained from both soil models showed similar trends, the results of the Hardening Soil model generally showed greater arching. For example, at a distance of $1D_{TD}$ above the trapdoor centreline, minimum vertical stress ratios of approximately 0.5 and 0.2 were obtained from the Mohr-Coulomb and Hardening Soil models, respectively. There were significant differences in both the trends and magnitudes of the horizontal stress ratios obtained using the two models. The Hardening Soil model was, however, able to capture the gradual increase in vertical stress at $1D_{TD}$ from the base directly above the trapdoor which was observed in the laboratory tests and discussed in § 4.4.3.3. The development and subsequent decay of the arch was also identified from the horizontal stress ratios obtained using both soil models.

The importance and difficulty of choosing the appropriate coefficient of lateral earth pressure ($K_{cp}$) to use in the theoretical models to calculate the stress acting on an active
trapdoor was highlighted in Chapter 4. Correspondingly, an investigation of the values obtained from the FE analyses was carried out. It was concluded from this that the appropriate value is not obvious as it is dependent on the soil model used, the degree of trapdoor displacement and the distance from the trapdoor.

The vertical stress ratios obtained from the FE analyses and from the laboratory tests were compared. The ability of the soil models to predict both the rate of change of the stress ratio with respect to trapdoor displacement and the magnitude of the arching measured in the laboratory tests was in most cases unacceptable. This may be due in part to the assumptions regarding soil behaviour inherent in the models which were described in § 6.2 and due to the choice of material parameters described in § 6.3. However, other than on the trapdoor itself, the Hardening Soil model generally provided a better prediction than the Mohr-Coulomb model.

6.8.3 Numerical modelling of MTBM tests

Although the 3D nature of tunnelling was highlighted in Chapter 5, a 2D FE simulation of the progression of the MTBM was carried out to identify stress-transfer patterns for this simple case, prior to the more complex 3D modelling of the problem.

After a preliminary analysis which identified the appropriate mesh configuration for each soil model, the MTBM tests carried out in the laboratory were modelled numerically. The extent of the arching zone was identified for both vertical and horizontal stresses. Although the behaviour of the vertical stresses obtained from the two models was broadly similar, the behaviour of the horizontal stresses varied considerably. Reductions in the vertical stress above the tunnel centreline of approximately 20% and 10% were obtained from the Mohr-Coulomb and Hardening Soil models, respectively, with increases in the horizontal stresses of 25% and 60%, respectively. The extent of the arching zone varied depending on the soil model used. For example the largest vertical stresses occurred at 1Df from the centreline using the Mohr-Coulomb model (≈60% increase from initial), and at 2Df using the Hardening Soil model (≈30% increase from initial). The variation of the arching behaviour at 1Df from the centreline from negative to positive depending on the degree of tunnel contraction was illustrated in Figure 6.34.
In the same way as for the trapdoor tests, the development of the arch at a distance of $1D_f$ above the tunnel crown was identified from an increase in the horizontal stress at that point. This was followed by a subsequent reduction in the stress as the arch failed, as shown in Figure 6.33 and discussed in § 6.6.4.2.

Following the 2D analysis, a 3D simulation of the MTBM was carried out. The tunnel contraction was applied in this analysis in such a way that matched the magnitude of contraction applied in the 2D analysis, yet accounted for the tapered section of the MTBM. The 3D nature of the arching effect was investigated by examining the variation in the horizontal and vertical stress ratios as the MTBM moved towards the plane of measurement.

The most significant increases in the vertical stress ratio as the MTBM approached the plane of measurement were predicted at $1D_f$ from the centreline, and decreased as the distance from the springline increased. Significant increases were also predicted at $2D_f$ from the centreline, whilst increases of the order of <5% were predicted directly above the centreline and at $3D_f$ from the centreline. The passing of the MTBM caused a considerable reduction in the vertical stress ratio directly above the centreline and at $1D_f$ from the centreline. No reduction was predicted at distances greater than $1D_f$ from the centreline. At all points, a subsequent positive arching was predicted as the MTBM continued to progress away from the plane of measurement. This continued until the face had progressed approximately $2.5D_f$ from the plane. A greater increase was predicted at points closer to the crown than for those further from the crown.

The movement of the MTBM towards the plane of measurement caused an increase in the horizontal stresses above the centreline for points greater than $1D_f$ from the crown, at $1D_f$ from the centreline at springline and crown level, at $2D_f$ from the centreline from springline level to $1D_f$ above the crown, and at all points at $3D_f$ from the centreline. Therefore, the progression of this zone of positive arching as the distances from the centreline and from the crown increased was evident (Figure 6.42 sub-figure ‘a’). Significant reductions in the horizontal stresses caused by the passing of the MTBM were predicted. This zone of negative arching was also observed to progress through the sand mass with increasing distance from both the centreline and the crown (Figure 6.42 sub-figure ‘b’). Significant positive arching was predicted at all heights directly above the
centreline and at 1D_t from the centreline due to the progression of the MTBM past the plane of measurement. Further from the centreline at distances of 2D_t and 3D_t, the magnitude of the increase from the minimum value decreased as the distance from the springline increased.

The minimum stress ratios obtained from the 2D and 3D analyses were compared. Although similar trends were observed, the magnitudes of the arching effect varied. Generally speaking, the vertical stress ratios obtained from the 2D analysis were lower than those obtained from the 3D analysis. As the same tunnel contraction was applied in both analyses, this may be due to the initial increase in the vertical stress ratio caused by the advancement of the MTBM towards the plane of measurement in the 3D analysis. Directly above the centreline and for distances below 1D_t from the crown, the horizontal stress ratios from the 3D analysis were lower than those obtained from the 2D analysis. The opposite was true at all other points. In a similar way to that described above, this is probably due to the changes in the stress ratio due to the progression of the MTBM towards the plane of measurement. The importance of modelling the 3D nature of the tunnelling problem in order to fully understand the arching effect was therefore highlighted.
7. Conclusions

7.1 Introduction

As urban areas and their populations continue to grow, there is an increasing need to provide improved infrastructure for the transport of people and provision of services, and tunnels are consistently seen as a suitable solution to this problem. Consequently, it is very important to fully understand the possible implications of their construction on adjacent surface and sub-surface structures.

This thesis describes an experimental, analytical and numerical study, the principal aim of which was to investigate the phenomenon of arching, i.e. the transfer of stress from one soil mass to another, which depends on the relative settlement between those masses. From a thorough review of the literature, it was concluded that previous experimental and analytical studies of the active trapdoor problem were concentrated primarily on the force acting on the trapdoor itself. Few details of the arching effect in the surrounding soil mass are available. Similarly, the work in the literature related to tunnel construction is generally deformation based, with limited information on the stresses occurring in the sand mass. Therefore, this study also investigated the applicability of miniature earth pressure cells (EPCs) to quantify the arching effect in reduced scale models. These were calibrated during a series of tests using a Rowe cell.

Initially, a series of tests were carried out to study arching due to an active trapdoor, in which the variation in force acting on a trapdoor and stress within the soil mass above and adjacent to it were measured using a load cell and miniature EPCs, respectively. Following this, the arching effects and settlements caused by the advancement of a purpose-built miniature tunnel boring machine (MTBM) through the sand mass were measured. Finally, a series of finite element (FE) tests were performed to simulate both problems. The purpose of this chapter is to present some of the important findings from the work, and to suggest areas where further research is warranted.
7.2 Arching due to active trapdoor

The results of six of the active trapdoor tests which were performed in the laboratory were presented and discussed in Chapter 4. All tests were performed using Glenview sand at approximately the same relative density. The principal difference between the tests was that tests CTD002-CTD004 and CTD005-CTD007 primarily involved the instrumentation of the sand mass directly above and adjacent to the trapdoor, respectively. A summary of the conclusions made from the results of these tests is presented below.

The measured average stresses acting on the trapdoor were in agreement with those presented in the literature from similar tests. The results obtained from a number of analytical solutions compared poorly to the measured values. The best predictions of the measured minimum and ultimate trapdoor stresses were provided by the solutions of Janssen (1895) and Evans (1983) respectively, although these still underpredicted them significantly.

The dependency of the values obtained from the analytical solutions on the coefficient of lateral earth pressure ($K_{ep}$) was highlighted. Several of the solutions were unable to predict the minimum trapdoor stress to within acceptable levels even considering the broad range of $K_{ep}$ values which are presented in the literature. In order to match the minimum stress using the various solutions, a broad range of $K_{ep}$ values was necessary, thereby emphasising the difficulty in the selection of the most appropriate value. Evans’ solution (1983) was found to result in an exact prediction of the ultimate trapdoor stress using Krynine’s coefficient of earth pressure ($K’$) value, despite the fact that a significantly higher value was measured in the corresponding experimental tests. The values of $K_{ep}$ used to match the measured values were significantly higher for the minimum than for the ultimate trapdoor stresses.

The arching effect with regard to vertical stresses was quantified in the soil mass directly above and adjacent to the trapdoor using miniature EPCs. Negative arching (i.e. a decrease in stress due to its transfer to adjacent soil masses) was measured directly above the centreline and directly adjacent to it. At a further distance from the centreline, positive arching (i.e. an increase in stress due to its transfer from adjacent soil masses) was initially recorded, before negative arching was experienced at larger trapdoor displacements. The
difference in behaviour between cells placed directly above and in close proximity to the trapdoor was explained by consideration of a zone of dilatant soil directly above the trapdoor.

The relationship between the degree of trapdoor displacement and the vertical stress allowed the development of the failure mechanisms around an active trapdoor to be investigated. It was concluded that contrary to the traditional view of the development of the failure mechanisms presented in the literature, both internal and external failure surfaces develop as soon as trapdoor displacement begins.

The results obtained from the miniature EPCs compared well with those obtained from the load cell. As the results from the load cell were in good agreement with those published in the literature, the performance of the EPCs was verified. They generally performed well throughout the test series, although the variation in their performance due to placement effects was highlighted. However, it was concluded that the results provided by them were very useful.

### 7.3 Arching due to MTBM

Chapter 5 presents and discusses the results from the series of MTBM tests which were performed in the laboratory. All tests were performed using the Glenview sand at approximately the same density. The tests varied principally in the positions at which the EPCs were placed. Details of this are presented in Tables 3.7 and 3.8.

The MTBM was designed and fabricated specifically for the purposes of this work. It consisted of a rotating cutting face with three cutters, through which the excavated soil moved into the tunnel lining, which directly followed behind the cutting face. The soil was then removed from the lining using a vacuum. The MTBM accounted for the three-dimensional nature of the tunnelling problem by simulating the forward progression of the tunnel face. In addition to this, forces were imposed on the soil mass (particularly at the face) and ground movements encountered which are similar to those in the prototype situation. The use of the MTBM allowed the understanding of the three-dimensional aspect of the arching effect and the associated settlements to be developed. Its design, fabrication and use may therefore be considered a success.
Chapter 7  Conclusions

The calculated surface trough width parameter (K\textsubscript{s}) was in good agreement with the data presented in the literature, and its decrease with increasing volume loss due to development of the typical chimney-type failure mechanism was identified. The Gaussian distribution provided a good prediction of the vertical settlements. The solution of Attewell and Woodman (1982) was found to provide a good prediction of the longitudinal settlement trough when the data which may have been influenced by the boundary conditions was removed from the comparison.

It was concluded that Particle Image Velocimetry (PIV) is a very useful tool to obtain sub-surface transverse and longitudinal settlement data. The transverse sub-surface settlement trough was well predicted by the Gaussian distribution, despite some minor under-prediction at the trough shoulders. The increase of the trough width parameter with depth from the soil surface and reduction with volume loss was identified. The solution of Mair \textit{et al.} (1993) was found to give a better prediction of the increase of the trough width parameter with depth than that of Jacobsz (2002).

The behaviour of the arching zone in a vertical plane perpendicular to the axis of the MTBM as it progressed towards and past it has been quantified. The variation in the behaviour from positive to negative arching as the face advanced towards the plane, followed by the subsequent return to positive arching as the face moved away from it was observed, thereby highlighting the three-dimensional aspect of the problem. The most significant reduction in the vertical stress was recorded directly above the tunnel centreline in close proximity to the crown. The dependency of the arching effect on both the degree of settlement and length of shear planes was discussed, whilst the extent of the arching zone in the transverse direction was identified.

The response of the EPCs in both the trapdoor and MTBM tests was in agreement with the trends observed in the numerical analysis presented in Chapter 6, and there was reasonable repeatability between tests. Therefore, despite some functionality issues which were overcome by redundancy, the use of the miniature EPCs in the laboratory tests may be considered a success.
7.4 Numerical investigation

A FE study was carried out to simulate the laboratory active trapdoor and MTBM tests. The commercially available software PLAXIS was used for both the two dimensional (2D) and three dimensional (3D) analyses (versions 9.2 and 3D Tunnel version 2.4, respectively). The conclusions which were made are summarised below.

7.4.1 Constitutive soil models

A series of laboratory tests were performed on the Glenview sand used in the physical tests in order to obtain some of the parameters used in the FE material models, namely the Mohr-Coulomb, Hardening Soil and Hypoplastic models. The ability of these to simulate the behaviour of the soil was assessed by a comparison of the results of the laboratory tests with those modelled numerically. It was concluded that the Hardening Soil model was more suitable to simulate the behaviour of the sand used for the purposes of this study than the Hypoplastic soil model. This was due to problems with the latter model related to artificial cohesion in the model. The Mohr-Coulomb model was also used due to its simplicity.

7.4.2 Simulation of active trapdoor tests

The configuration of the FE model was chosen based on the results of a series of preliminary tests. Amongst the findings of these tests were that an interface should not be used at the transition from the fixity beside the prescribed displacement to the prescribed displacement itself, despite the recommendations in PLAXIS (2010), due to the influence this has on the shear strains in the soil mass in close proximity to it.

The magnitude and extent of the arching effect was found to be dependent on the soil model used. The results obtained from the Hardening Soil model generally showed a greater degree of arching than those from the Mohr-Coulomb model. The former captured the observed behaviour in the laboratory tests just above the trapdoor, where an increase following the minimum value was realised. The development and subsequent failing of the arch in close proximity to the trapdoor was identified from the horizontal stress results using both soil models. It was concluded from the results of the FE analyses that the appropriate value of the coefficient of lateral earth pressure ($K_{cp}$) to use in the analytical
Chapter 7 Conclusions

solutions to predict the forces acting on the trapdoor was not clear, as it varies considerably with the degree of trapdoor displacement and soil model used.

The vertical stress results obtained from the numerical investigation were compared with those from the laboratory tests. The ability of the soil models to predict the behaviour of the sand was limited, particularly with regard to the rate of change of the stress ratio with trapdoor displacement. The Hardening Soil, however, generally provided a better prediction of the measured stresses than the Mohr-Coulomb model.

7.4.3 Simulation of MTBM tests

The 2D simulation of the MTBM test allowed the development of the understanding of the arching effect in a plane transverse to the tunnel face, prior to the 3D modelling of the problem with its associated complexities. It allowed the investigation of the arching effect with respect to horizontal stresses, which could not be measured in the laboratory tests, to be investigated. The relationship between the vertical and horizontal stresses above and adjacent to the tunnel and the corresponding degree of tunnel contraction were examined.

The behaviour of the arching zone in the 2D simulation was dependent on the soil model used to simulate the problem. Using the Mohr-Coulomb model, the most significant positive arching with respect to vertical stresses occurred closer to the centreline than for the Hardening Soil model. The magnitude also varied, with the results obtained from the model using the Hardening Soil showing greater negative and positive arching. The arch above the tunnel crown was identified from the horizontal stresses. Its development on initial tunnel contraction was evident, followed by its failure after continued contraction. Although the results of the 3D numerical investigation could not be compared directly with the laboratory results due to the application of a different volume loss in the FE analysis to that which was measured in the laboratory tests, the general behaviour of the arching zone in the two methods was broadly similar.

A marginal increase in the vertical stress caused by the progression of the MTBM towards the plane of measurement was observed at $1D_f$ from the centreline, where $D_f$ is the diameter of the tunnel face. Minor increases were experienced directly above the tunnel centreline and at distances greater than $1D_f$ from the centreline. The reduction in the vertical stresses caused by the passing of the MTBM was greatest directly above the tunnel.
centreline and decreased with increasing distance from this plane. At all points, an increase in the vertical stress was experienced as the face moved away from the plane of measurement. There was a general trend of points closer to the crown undergoing a greater increase than those further from the crown.

The variation in the horizontal stresses was more complex. As the MTBM progressed towards the plane of measurement, an increase was observed directly above the tunnel centreline, except in close proximity to crown level, where a decrease occurred. A zone of positive arching was identified which extended from springline and crown level directly adjacent to the tunnel to increased heights at greater distances from the centreline. A similar extending zone of negative arching developed through the sand mass as the MTBM passed the plane of measurement. Finally, significant positive arching was experienced at all heights from the crown in close proximity to the tunnel centreline. The increase from the minimum decreased at further distances from the centreline as the distance from the springline increased.

The importance of modelling the 3D nature of the problem was emphasised by the poor comparison of the 2D and 3D analysis. The vertical and horizontal stresses obtained from the 2D analysis varied considerably from the results of the 3D analysis, despite the same total tunnel contraction being imposed in both.

7.5 **Recommendations for future research**

Although the work presented in this thesis has contributed significantly to the knowledge of the arching effect, a great deal more research is required. Further investigations are warranted in the following areas:

- Observation of failure mechanisms and relation to arching effect in reduced scale tests:
  - around an active trapdoor (requires the modelling of half the problem, with the plane of symmetry being the Perspex test chamber face)
  - in the transverse direction around a tunnel (requires simplification of the method of tunnel construction, for example to that of Atkinson and Potts (1977))
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- in the longitudinal direction ahead of a tunnel (requires the modelling of half the tunnel problem, with the plane of symmetry being the Perspex test chamber face)
- Further development of the MTBM
- Direct measurements of the effect of tunnel construction on adjacent sub-surface structures in both reduced scale tests and field tests. With regard to field tests, this should be entirely possible with some existing structures, for example, other tunnels.
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