THREE-DIMENSIONAL ANALYSIS OF SURFACE SETTLEMENT IN SOFT GROUND TUNNELING

by

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Bachelor of Geological Engineering
University of Minnesota, 2001

Submitted to the Department of Civil and Environmental Engineering in Partial Fulfillment of the Requirements for the Degree of Master of Engineering in Civil and Environmental Engineering at the

Massachusetts Institute of Technology

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ABSTRACT

A parametric study was performed to evaluate the effects of different construction variables on surface settlements above tunnels in soft ground. The finite element program “PLAXIS 3D Tunnel” was used to carry out the numerical analysis for the earth pressure balance (EPB) shield tunneling method in a saturated, normally consolidated clay. The two construction parameters that were varied were the face pressure and the grout pressure.

Longitudinal settlement profiles were obtained for many different face pressures and several different grout pressures. Results show that the ground surrounding the tunnel is very sensitive to changes in grout pressure in terms of surface settlement and failure of the soil body, while a wide range of face pressures can be accommodated without failure. Furthermore, minimum surface settlement is achieved for a certain face pressure and becomes larger as the pressure is increased or decreased from that particular value, which for this analysis corresponded to an overload factor of one. Also, results show that surface settlement decreases with increasing grout pressure, as expected.

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CHAPTER 1. INTRODUCTION

As urban regions continue to grow both in population and in geographical area, an ever-increasing need arises for additional physical infrastructure. The countless public works projects carried out to address the needs of growth in urban settings must take into consideration the disruptions that may be produced by the work. The urban environment is characteristically sensitive to any type of infringement, interference, or pollution (Eisenstein, 1999).

Tunnels, often located adjacent to existing structures and relatively close to the ground surface, are frequently included in these public works projects. Of paramount importance is protecting the adjacent structures from damage caused by excessive settlement, either total or differential, and significant rotation. In addition, utilities must also be safeguarded because damage to lines and mains can bring about economic loss and, in some cases, jeopardize the health and safety of the public. For this reason, ground deformation during the tunneling process, generally manifested in the form of surface settlements, become a major design element.
CHAPTER 2. EPB METHOD

2.1 Introduction

Shield tunneling techniques have been developed in part to minimize surface settlements. In particular, the earth pressure balance (EPB) method has become one of the most commonly used soft ground tunneling schemes in recent years. The reasons for its high rate of utilization are undoubtedly its versatility, economic viability, and effectiveness at minimizing surface settlements in poor ground conditions.

2.2 History

A Japanese construction company, Sato Kogyo Company Ltd., was the first to develop the concept of the EPB shield. The company began experimental work on the EPB shield in 1963 in order to develop a machine that could excavate efficiently while producing minimal environmental impacts. After much laboratory and field research, the first machine was built in 1966 by the Ishikawajima Harima Heavy Industries Company Ltd. (Suwansawat, 2002).

The first actual use of an EPB shield began in the early 1970’s on a 1,900-meter collector drive in Tokyo (Suwansawat, 2002). In the ensuing years, the method was used extensively in Japan. In fact, by 1980, 64 tunnels had employed the EPB method, amounting to 16% of all the shield-driven tunnels in Japan (Naito, 1984).

EPB shields have gained in popularity and have been utilized in many countries around the world. Table 2.1 shows a number of prominent tunneling projects in recent years on which the EPB shield was used. The first use of the EPB shield in the U.S. was on the N-2 project in 1981. The project, located in the San Francisco Bay area, comprised a 900-meter shallow tunnel excavated through soft, saturated cohesive soils (Finno, 1985).
### Table 2.1. Recent tunnels constructed using the earth pressure balance method (Santa Clara Valley Transportation Authority, 2002)

<table>
<thead>
<tr>
<th>Country</th>
<th>Tunnel Name</th>
<th>Date Open</th>
<th>Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>U.S./Canada</td>
<td>St. Clair River Tunnel</td>
<td>1995</td>
<td>1.8 km</td>
</tr>
<tr>
<td>Denmark</td>
<td>Storebælt Rail Tunnel</td>
<td>1997</td>
<td>8.0 km</td>
</tr>
<tr>
<td>Japan</td>
<td>Trans Tokyo Bay Highway Tunnel</td>
<td>1997</td>
<td>9.5 km</td>
</tr>
<tr>
<td>United States</td>
<td>Congress Heights Metro Tunnel Washington D.C.</td>
<td>2001</td>
<td>4.7 km</td>
</tr>
<tr>
<td>Canada</td>
<td>Toronto Sheppard Transit Tunnel</td>
<td>2002</td>
<td>6.4 km</td>
</tr>
<tr>
<td>Thailand</td>
<td>Bangkok MTR Subway</td>
<td>2003 (projected)</td>
<td>22 km</td>
</tr>
<tr>
<td>Germany</td>
<td>Fourth Elbe Road Tunnel</td>
<td>2003 (projected)</td>
<td>2.6 km</td>
</tr>
</tbody>
</table>

### 2.3 General Description

The fundamental concept of EPB tunneling is that a rotating cutterhead is propelled forward by a series of jacks pushing from the concrete lining that has been previously installed by mechanisms in the aft portion of the assembly. The body of the shield, commonly known as the tail skin, is a steel cylinder which not only houses the mechanical components that enable the tunnel excavation to move forward, but also prevents radial groundwater inflow and inhibits inward movement of the soil until the primary or final lining are installed. Soil is excavated from the face through a set of slots in the rotating cutterhead, as shown in Figure 2.4, and deposited into an area known as the spoils chamber, although it is occasionally referred to as the working chamber, soil chamber, muck chamber, or plenum; it will be called the spoils chamber hereafter. The soil is then transported from the spoils chamber by means of a conveyance system to an access area where the material can be hauled away. A schematic of an EPB shield tunneling system is presented in Figure 2.1 (see also Figure 2.2).
The primary characteristics that distinguish EPB shields from other types of shields are the mode of face support and the method of spoil removal. As mentioned, soil is excavated through the slots in the rotating cutterhead and deposited in the spoils chamber as the shield assembly advances. Thus, it is the excavated material in the spoils chamber itself that acts to support the face. With the material in the chamber, an apparatus known as a screw conveyor (see Figure 2.2) rotates at a specific rate as determined by a skilled operator or by an advanced software program to take away the soil. In general, it removes material from the spoils chamber at the same rate as material enters the chamber through the openings in the cutterhead. However, the pressure in the spoils chamber, measured by a series of pressure transducers mounted on the rear bulkhead of the spoils chamber, can be increased or decreased as the rate of screw conveyor rotation is adjusted relative to the rate of excavation at the face. If the pressure on the face must be increased, the rate of soil removal by the screw conveyor is slowed, and vice versa. The measurement of face pressure is a critical responsibility in EPB tunneling because if the pressure is not constant, but oscillates, the varying pressure can lead to collapse of the face (Siegmund, 1991). Figure 2.2 depicts a side view of the EPB system, which highlights the spoils (working) chamber and screw conveyor.
2.4 Ground Conditions

When first developed, the EPB shield was only used in soft, cohesive soils capable of filling the spoils chamber completely in order to support the face. The reason for this narrow range of application was that the soil was needed to seal the screw conveyor against groundwater pressure, which meant that there were high demands on its plasticity (Becker, 1995). Over the years, however, the EPB system has become more adaptable. It has proven that it can readily be applied in somewhat stiffer and coarser material, although it achieves its best performance in soft, cohesive soils with relatively low groundwater pressures and few cobble and boulder obstructions. (Suwansawat, 2002).

2.5 Shield

Shields (see Figure 2.1) provide a means of protection from the collapse of the tunnel walls and crown in the stage before the lining has been installed. In sufficiently stable soils, there is strictly speaking no need for a shield. However, shields are often used even in good ground conditions in order to form a shelter for segment erectors, muck disposal units, and grouting
equipment. In addition, shields can provide a link between the lining and the face so that work in both areas can be carried out simultaneously (Suwansawat, 2002). Figure 2.3 shows an EPB shield.

![EPB Shield](image)

**Figure 2.3. EPB shield (Mitsubishi Ltd.)**

### 2.6 Face

The spoils chamber is arguably the most important component to the proper functioning of the entire EPB system. The dimensions of the cutterhead must be designed with care so that excavated soil can be brought into the spoils chamber smoothly and efficiently through the cutter slits and so that no clogging occurs. Important factors controlling this process are the geological conditions and the mechanical configuration of the shield. Typical slit widths range from 20 to 50 cm, and the ratio of the area of slit openings to the full face area is generally between 15 and 40% (Naito, 1984). Figure 2.4 shows an EPB shield and cutterhead with slits.
As mentioned above, the EPB method is usually not considered as efficient in non-cohesive soils as it is in cohesive soils. The high internal friction of most non-cohesive soils prevents the material from filling the spoils chamber completely, resulting in incomplete face support and greater surface settlements (Becker, 1995). Even so, it has been observed that the amount of groundwater inflow into the spoils chamber is not adequate or that the soil does not contain enough very fine particles. In these cases, water or a clay-water suspension is introduced into the spoils chamber in order to develop a fluid that can transfer pressures efficiently. The injected material is mixed with the excavated soil into a relatively homogeneous mass by a set of mixing bars mounted on the revolving cutterhead and the stationary pressure bulkhead (Siegmund, 1991).

Even when tunneling in low-permeability cohesive soils, a conditioning agent is often used. A foam substance is injected into the spoils chamber not only to reduce interparticle friction, but
also to reduce friction between soil particles and the metallic parts of the shield in order to decrease the required cutterhead torque and the amount of wear on the machine (Lamberti Spa).

2.7 Screw Conveyor

Screw conveyors are the means by which excavated soil is removed from the spoils chamber. The operator can maintain a positive pressure on the face if the screw conveyor removes soil at a rate less than the rate at which soil is excavated at the face. Screw conveyors similar to flight augers, and typically measure 10-12 m long and approximately 250-300 mm in diameter. Many are designed so that large cobbles and boulders can pass through the conveyor without causing blockage. In many cases, however, the screw conveyor can be removed from the spoils chamber, allowing the face to be sealed off, in the event of a blockage or necessary repair (Reilly, 1982). Figure 2.5 shows a screw conveyor within the EPB shield.

As with the spoils chamber, attempts have also been made to inject fine soil particles into the screw conveyor to make flow easier when tunneling in non-cohesive soils, but with the same accompanying setbacks as mentioned above (Becker, 1995). In addition, a sweeper plate in the spoils chamber, which rotates in a similar manner as the cutterhead, allows for easier discharge of muck into the screw conveyor to prevent blockages (Naito, 1984).
2.8 Ring Erection

The tunnel lining is installed at a given spot after the shield has advanced most of its length past that location. In other words, each lining section is erected within the tail skin and is then exposed to the converging soil mass as the shield moves forward. Figure 2.7 in Section 2.9 illustrates this set-up.

The lining is usually made of concrete, with the most oft-used variety coming in pre-cast segments. Investigations have been made into installing cast-in-place concrete lining into tunnels using the EPB method, but curing time, formwork, and the difficulty of administering quality control on-site present obvious complications (Naito, 1984).

The lining sections are transported into the tunnel through the access portal, and space is usually reserved inside the TBM for segment storage. These segments are then lifted into place by a vacuum erector or a threaded pin/lifting insert system. Then, the segments are usually connected to one another by a curved tie rod. Generally, six to seven pre-cast segments are erected to form the closed circular ring (Reilly, 1982). A segment erector system is shown in Figure 2.6.

![Figure 2.6. Erector arm installing a lining segment (after Suwansawat, 2002)](image-url)
2.9 Tail Void/Grouting

In EPB shield tunneling, the segments are erected inside the tail skin of the TBM, as mentioned above. Also, the shield diameter is larger than the outside diameter of the installed lining sections, resulting in a gap between the segments and the excavated soil mass. This volume, known as the tail void or annulus, tends to become occupied by the inward deforming soil as the shield and tail skin assembly advance. As is always the case in soft ground tunneling, any convergence of material into the tunnel has the effect of producing settlement at the surface (Reilly, 1982). Figure 2.7 is an illustration of the tail void.

![Illustration of the tail void](image)

Figure 2.7. Tail void (after Suwansawat, 2002)

The dimensions of the tail void are larger than the absolute minimum clearance because the tail skin must leave sufficient room to permit assembly of the lining rings, and clearance must be provided between the outside of the segments and the inside of the tail skin to allow for the shield to be maneuvered around curves and to correct misalignment (Suwansawat, 2002).
It has been shown through extensive field experience that the best way to contend with the deformation and thus inhibit surface settlement is to pump a pressurized grout into the tail void. The void is continually filled with grout as the machine advances in one of two ways (Reilly, 1982):

1) Grout is pumped through pre-drilled holes in the pre-cast concrete lining segments; this is the more traditionally used method.

2) Grout is injected through grout ports at the back of the tail skin, i.e. between the inner surface of the tail skin and the outer surface of the lining segments.

The former method is illustrated in Figure 2.8, while the latter method is shown in Figure 2.9.
If the method in Figure 2.9 is used, larger ground settlement will occur because a larger space is necessary to accommodate the grout lines, leading to a larger tail void. In addition, it is difficult to clear out the injection lines and washout valves if they become clogged. On the other hand, the larger tail void will lead to improved steering around horizontal and vertical curves (Reilly, 1982).

Various kinds of tail seals have been developed to prevent fines and grout from entering the shield during the grouting process. One system, shown below in Figure 2.10, consists of rows of wire brushes that have been greased to provide a tight seal, although it is not completely airtight. These devices, arguably one of the most critical design elements, must be able to withstand significant grout pressures (Siegmund, 1991, and Reilly, 1982).
2.10 Spoils Removal

The excavated soil is transported from the face to a conveyor belt system via the screw conveyor. From here, trains or trucks haul the spoil to the portal or an access shaft for disposal. The train can also bring required materials including lining segments and personnel to the EPB machine (Reilly, 1982). Figure 2.11 depicts the spoils removal system commonly employed in EPB operations.

Figure 2.11. Spoils removal from an EPB shield (Los Angeles County MTA)
CHAPTER 3. SURFACE SETTLEMENTS

3.1 Introduction

The adverse effects on surface structures and near-surface utilities induced by tunneling have provided the impetus for many researchers to develop methods of estimating surface settlements. Not only were the magnitudes of final settlements investigated, but also the settlement profiles, or troughs. Defining the spatial characteristics of the surface settlement leads to a greater understanding of which structures/utilities will be affected and to what degree.

Estimating ground deformation around tunnel excavations in soft ground has been approached in the following ways (Suwansawat, 2002):

1) **Stochastic and empirical methods**: Investigators such as Litwiniszyn (1956) and Peck (1969) proposed a stochastic, or probabilistic, approach to understanding soil behavior around tunnels. The main result was an inverted normal probability function, which was empirically determined, to describe the transverse settlement trough. Subsequent researchers such as Attewell and Woodman (1982) proposed a method for estimating the longitudinal settlement trough. These methods will be described in greater detail later.

2) **Analytical methods**: Closed form solutions were proposed by a number of researchers, including Sagaseta (1987), Verruijt and Booker (1996), Loganathan and Poulos (1998), and Pinto (1997). All of these methods were used to obtain estimates of deformation occurring in the transverse cross-section of the tunnel.

3) **Finite element and numerical methods**: These numerical techniques have been extended in recent years primarily due to increased computing power. However, the methods still have limitations such as the lack of accuracy in some of the input parameters due to difficulty in acquiring realistic in situ material properties. Finite element modeling, in particular, will be described later on.

4) **Laboratory experiments**: A number of researchers have run scale model tests in the laboratory to gain a better understanding of ground movement and collapse mechanisms.
The results have shown that the deformation parameters obtained in the tests can be used for estimating settlement trough geometry and maximum surface settlements.

### 3.2 Attewell and Woodman (1982)

While many methods have been proposed to estimate the transverse settlement trough, the empirical approach suggested by Attewell and Woodman (1982) is the only methodology that attempts to predict longitudinal settlements. Knowledge of the approximate dimensions of the transverse settlement trough allows one to estimate which buildings will be affected by the tunneling process and how much they might settle. The longitudinal settlement profile, on the other hand, is generally used to predict the angle of tilt of surface structures located above the tunnel centerline, as shown in Figure 3.1.

![Longitudinal surface settlement caused by shield tunneling (after Suwansawat, 2002)](image)

While surface structures also tilt inward toward the tunnel centerline when viewed in the transverse cross-section, the tilting in the longitudinal direction happens twice (once as the face approaches the structure and once as it moves back to near its original orientation) compared to...
the one rotation in the transverse direction. In addition, the evolution of settlement as the shield tunneling advances underneath a given point can be predicted by the method of Attewell and Woodman. This material gives insight into the amount of total settlement, which may be different from the net settlement if the ground ahead of the face is allowed to heave.

Some background of methods proposed by previous researchers will be provided hereafter because the Attewell and Woodman technique for estimating the longitudinal surface settlement profile is built upon the prior work. First, Attewell and Woodman based their approach on the stochastic scheme proposed by Litwiniszyn (1956). This stochastic model is rooted in probability theory, and follows from a condition in which the ground is represented by a 3D medium of equidimensional spheres. Movement is initiated according to probability theory and the law of gravity by removing one of the spheres at the bottom of the medium. This process leads to an inverted bell-shaped settlement trough that can be approximated by a normal probability curve.

In later years, the method of Peck (1969) combined observed tunneling settlements with Litwiniszyn’s approach to produce his widely used method for estimating the transverse settlement trough above a single tunnel. The inflection point $i$, which is defined as the distance from the tunnel center line to the location where the concavity of the curve changes from positive to negative, is one of the most useful and important parameters to come from the method. Figure 3.2 shows the Gaussian settlement trough with inflection point.

![Figure 3.2. Normal probability curve used to describe transverse settlement trough (after Suwansawat, 2002)](image-url)
Peck assembled field observations from 17 tunnels that exhibited noticeable trends. The observed inflection point $i$, normalized by the tunnel radius $R$, is plotted versus the tunnel depth $z$, normalized by the tunnel diameter $2R$, as shown in Figure 3.3.

![Figure 3.3. Relation between inflection point and dimensionless tunnel depth for different ground conditions (details including data from actual projects are provided in Peck, 1969)](image)

Ground loss $G.L\%$ is expressed as the percentage fraction of excavated area of the tunnel. The volume of ground loss into the tunnel $V_s$ is defined as the percent fraction $G.L\%$ of the theoretical volume of excavated ground per unit length according to

$$V_s = \frac{G.L\%}{100} \left( \pi \frac{D^2}{4} \right)$$

where $D =$ tunnel diameter.
Thus, a zero ground loss $G.L\%$ would indicate a perfect tunneling process where no inward radial deformation occurs.

Mair (1996) assembled a number of recent case studies that discovered that the EPB method keeps surface settlement at relatively low levels compared to other tunneling methods. From these case studies, the ground loss $G.L\%$ in soft clays was found to be in the range 1-2%. Furthermore, Cording and Hansmire (1975) report that in clays the volume of the settlement trough $V_t$ is nearly equal to the volume of ground loss into the tunnel $V$. The volume of the settlement trough is given by Peck as

$$V_t = 2.5i\delta_{max}$$

where $\delta_{max} = $ maximum surface settlement induced by tunneling.

Thus, $\delta_{max}$ can be predicted in clay from the percent fraction of ground loss $G.L\%$ according to

$$\delta_{max} = \frac{G.L\%}{100} \left( \frac{\pi D^2}{10i} \right)$$

From here, the vertical settlement $\delta$ at any point transverse to the tunnel centerline can be estimated (Peck, 1969) according to

$$\delta = \delta_{max} \exp \left[ - \frac{y^2}{2i^2} \right]$$

where $y = $ horizontal distance on the surface from the tunnel centerline.

Attewell and Woodman assume that half of the maximum surface settlement $(0.5\delta_{max})$ occurs in the vertical plane of the shield face. Furthermore, they assume that point $x_i$, located far behind the advancing face, has undergone maximum surface settlement $\delta_{max}$, while point $x_f$ designates
the amount of surface settlement at the tunnel face. Figure 3.4 shows the coordinate system used by Attewell and Woodman, along with $x_i$, $x_f$, and $\delta$.

![Tunnel coordinate system for longitudinal surface settlement](image)

**Initial ($i$) Position of Tunnel Face**

**Final ($f$) Position of Tunnel Face**

Tunnel Advance

The equation for the settlement $\delta$ is given by Attewell and Woodman as

$$\delta = \frac{V_i}{\sqrt{2\pi \cdot i^2}} \exp \left[ -\frac{y^2}{2 \cdot i^2} \right] \left\{ G \left( \frac{x - x_i}{i} \right) - G \left( \frac{x - x_f}{i} \right) \right\}$$
where \( G(f) \) = integrated normal probability function (described below)

\[ z = \text{depth to tunnel axis}. \]

The \( G \) function is defined as the area under the normal probability function. Table 3.1 from Attewell and Woodman gives results from the integrated normal probability function. In particular, \( G(0) = 0.5 \) and \( G(\infty) = 1 \).

### Table 3.1. Numerical integration of the normal probability curve

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It should be cautioned that the method by Attewell and Woodman is intended for use only in clayey soils because the approach was checked against data from just one tunneling project, namely the Jubilee Line Project in London clay.

### 3.3 Finite Element Modeling

Tunnel engineering is perhaps one of the few areas of applied soil mechanics in which the numerical modeling of stresses and displacements is frequently used in practice. These numerical methods are based on the discretization of continua and employ algorithms that require the use of programmable computers. The frequent implementation of these types of analyses is attributed to a number of factors. First, the variety of and complex interaction between excavation and construction procedures in tunneling exert a strong influence on the distribution of stresses and strains in the ground surrounding the opening. Consequently, these complex interactions cannot be fully captured by analytical solutions. Second, numerical methods can reasonably account for the initial stress distribution, which is often characterized as non-homogeneous, and they can effectively consider non-linear, time-dependent or multi-phase behavior. Finally, the excavation/construction steps can be modeled to a degree of accuracy that is limited only by the required computational effort (Gioda and Swoboda, 1999).

At the present time, tunnels are analyzed mostly using 2D finite element schemes by practicing engineers. Two-dimensional numerical analyses have been popular due to their relative simplicity and their ability to be carried out fairly quickly and economically. Tunneling, however, is a 3D process, and efforts have been made by researchers to accurately simulate 3D construction sequences using 2D models primarily because experience has shown that more advanced 3D models are considered to be very time-consuming and are limited by the required computational effort. Some of the 2D methods that have been developed include a 2D plane-strain approximation of the drive process and the use of Fourier series to “expand” 2D solutions into the 3D realm (Gioda and Swoboda, 1999). For the reasons stated above, 3D finite element analyses have often been left in the domain of researchers, but the more complex analyses are beginning to be within reach of practitioners due to significant strides in computing capabilities.
The study of surface settlements has, quite naturally, been restricted to evaluating ground surface response above shallow tunnels using 3D finite element models. They permit one to consider the non-uniform stress distribution along the length of the tunnel and the deformation of the excavation face, which is particularly important for large-diameter openings. Current research on this topic is proceeding toward increasingly complex engineering problems such as those related to the effects of settlements induced by tunneling on the stability of existing buildings and masonry structures (Gioda and Swoboda, 1999).
CHAPTER 4. PLAXIS

4.1 Introduction

In the present analysis, PLAXIS 3D Tunnel was used to conduct a parametric study of the longitudinal settlement profile for a large-diameter tunnel constructed with the EPB method. PLAXIS 3D Tunnel is a robust finite element program that can handle complex problem geometries and complicated construction sequences. Advanced constitutive models can be also implemented to simulate non-linear and time-dependent soil behavior. Furthermore, the program uses special procedures to treat hydrostatic and non-hydrostatic pore pressures in the multi-phase soil medium and to consider the complex interactions between soil and structural elements.

4.2 Mesh

The 3D mesh is based on the 2D mesh. First, the problem geometry is established in the form of a cross-section model in the x-y plane, as shown in Figure 4.1 for a tunneling problem. This representation includes soil layers, water pressures, structural elements and boundary conditions, which are entered into the program by using a series of points, lines, and other components.

![Figure 4.1. Problem geometry in cross-sectional (x-y) plane](image-url)
After the geometry is entered, the program then automatically discretizes the problem into a 2D finite element mesh according to a prescribed standard node density, which can be refined in critical areas where key soil behavior is to be captured. The 2D model geometry is then reproduced a number of times in the z-direction. The location of each 2D mesh plane along the z-axis is fixed according to the 3D problem geometry, thus creating a series of parallel, stacked cross-sections. While the resulting cross-sections are called z-planes, the volumes between two successive planes are termed slices, as shown in Figure 4.2.

Figure 4.2. Definitions of z-planes and slices (after PLAXIS, 2001)

Following the replication of the z-planes into the 3D model, the 3D mesh is generated in much the same way as the 2D mesh. Figure 4.3 shows the progression from the 2D representation to the full 3D mesh.
4.3 Material Properties

4.3.1 Material Models

Five material models are available to be assigned to geomaterials in PLAXIS 3D Tunnel. Only four of them, however, can be applied to soils; the fifth is a jointed rock model. The four models are: linear elastic, Mohr-Coulomb, soil hardening, and soft-soil creep.

*Elastic model:* This model uses Hooke’s law of isotropic linear elasticity. In particular, the model uses two elastic stiffness parameters, namely Young’s modulus \( E \), and Poisson’s ratio \( v \). The linear elastic model is very limited for the simulation of soil behavior and it is generally considered inadequate for accurately modeling real soils. It is primarily used for bedrock and stiff structures inside the soil body. Therefore, it was not considered in the analysis.

*Mohr-Coulomb model:* This well-known model is customarily used as a first approximation of soil behavior. The model uses five parameters, namely Young’s modulus \( E \), Poisson’s ratio \( v \), the cohesion \( c \), the friction angle \( \phi \), and the dilatancy angle \( \psi \).
Soil hardening model: This is an elasto-plastic, hyperbolic model formulated in the framework of friction hardening plasticity. Moreover, the model involves compression hardening to simulate irreversible compaction of soil under primary compression. This second-order model can be used to simulate the behavior of sands and gravel as well as softer types of soils such as clays and silts.

Soft soil creep model: This is a second order model formulated in the framework of visco-plasticity. The model can be used to simulate the time-dependent behavior of soft soils like normally consolidated clays and peat. The model includes logarithmic compression.

4.3.1.1 Mohr-Coulomb

The Mohr-Coulomb model was chosen for the present analysis for a number of reasons. First, this model is simple enough that computation times are kept reasonable. This is in contrast to the more advanced models such as soil hardening and soft soil creep. Second, Mohr-Coulomb captures realistic material behavior unlike the linear elastic model, which is generally considered too crude to accurately represent true soil behavior. Third, the soft-soil creep model is not much better than the Mohr-Coulomb model for unloading problems such as tunneling, and its use would not be beneficial. Fourth, the five input parameters used in the Mohr-Coulomb model are familiar to practicing geotechnical engineers who can obtain these data with relatively simple and widely used tests, in contrast to the more advanced input parameters required for the more sophisticated models.

The intention of this document is not to treat the intricate details of how different materials are modeled in finite element analyses, although a general description follows. The Mohr-Coulomb model is based on the idea of elastic-perfectly plastic material behavior, which means that irreversible strains are observed after the yield point is reached in a stress-strain diagram, as shown in Figure 4.4.
The onset of the plastic realm is commonly defined by a yield function $f$, which is oftentimes represented as a surface when plotted in principal stress space, as shown in Figure 4.5. In the perfectly-plastic Mohr-Coulomb representation, the yield surfaces are considered fixed according to model parameters and not a function of the developed plastic strains. Soil behavior inside the yield surface is considered linear elastic with strains that are fully recoverable.
Figure 4.5 depicts the case where the friction angle $\phi$ is a non-zero number and the cohesion $c$ is zero, as is the case for granular materials like sand. In the case of undrained cohesive soils like clay, the yield surface is more cylindrical in shape rather than conical.

### 4.3.1.2 Drained versus Undrained

The program allows for a drained or an undrained analysis to be done using any of the material models described above. As is usually the case in geotechnical engineering design, a fully drained analysis will closely approximate long-term settlements while the undrained case will simulate the limiting case. Unfortunately, the PLAXIS 3D Tunnel program does not allow one to explicitly model primary consolidation, so the drained case must be employed to approximate the consolidation settlements. The developers of the software claim that the results of the drained analysis are fairly accurate to what a consolidation analysis might produce (PLAXIS, 2001).

### 4.3.2 Structural Elements

Plates and shells can be implemented into the analysis. Most of them are treated as linear elastic members because they are usually made of strong materials such as steel that yield only at very high stresses. Also, flexural rigidity, normal stiffness, and ultimate bending moment are used to define material behavior. In addition to the structural element itself, the interfaces between the soil and structural elements can be set to rigid ($R_{\text{inter}} = 1$) or a reduced strength ($R_{\text{inter}} < 1$), where $R_{\text{inter}}$ is the interface strength reduction factor. The case of $R_{\text{inter}} = 1$ represents conditions in which the values of adhesion and interface friction are equal to the cohesion and internal friction of the soil. On the other hand, $R_{\text{inter}} < 1$ signifies lower interface values than the strength parameters of the soil.

### 4.3.3 Loads

The program has the capability of applying a variety of load types to soil or structural elements. Point loads, line loads, and distributed loads can be applied to elements in the $x$-, $y$-, and $z$-directions. The distributed loads acting on $z$-planes in the longitudinal direction are of particular
interest in EPB tunneling where the jacking pressure and especially the face pressure are important.
CHAPTER 5. ANALYSIS

5.1 Introduction

The chosen problem geometry consists of a tunnel excavated through a normally consolidated clay using the EPB method. The aim of the analysis is to observe the behavior of the settlement trough on the surface as the shield passes through the material while varying the applied pressure at the face and the grout pressure in the tail void.

The input parameters for the problem were chosen somewhat arbitrarily. However, every effort was made to ensure that the tunnel dimensions, material properties, and boundary conditions were close to what might be encountered in the field. Moreover, if the input parameters are accurate, the modeled behavior will provide insight into mechanisms that will minimize the damage to surface structures.

5.2 Problem Geometry

The problem geometry is based on an actual tunnel excavation that took place south of Rotterdam, the Netherlands, in 1998. The Second Heinenoord Tunnel, as it is called, was excavated under the river Oude Maas using the slurry shield technique (COB, 1999). Figure 5.1 shows the tunnel.
There are two main differences between the actual tunnel and the modeled tunnel. First, the soil profile of the Second Heinenoord Tunnel consists of alternating layers of recently deposited sand and clay, while the model profile is a normally consolidated clay throughout. Second, the water table in the model has been raised from a depth of 1.5 m to the surface.

The depth to the center of the tunnel is 14.75 m, and the inside and outside diameters of the tunnel are 7.8 m and 8.5 m, respectively. Figure 5.2 shows the problem geometry.
Figure 5.2. Problem geometry (scale in meters)

Due to symmetry, only the half-space was modeled, as shown in Figure 5.3. Halving the number of elements significantly reduces computational time.

Figure 5.3. Modeled half-space
5.3 Fixities

Standard fixities were applied to the boundaries of the symmetric half-space as shown in Figure 5.4. These fixities consist of:

1) Vertical geometry lines for which the x-coordinate is equal to the lowest or highest x-coordinate in the model receive a horizontal fixity ($u_x = 0$).

2) Horizontal geometry lines for which the y-coordinate is equal to the lowest or highest y-coordinate in the model receive a full fixity ($u_x = u_y = 0$).

3) Structural elements that extend to the boundary of the cross-section, just as the top and bottom of the shield in Figure 5.5, receive a rotation fixity where no rotation is allowed around the z-axis (longitudinal axis) if at least one of the displacement directions ($u_x$ or $u_y$) of that point is fixed. (PLAXIS, 2001)

![Diagram of applied fixities](image)

Figure 5.4. Applied fixities for (a) vertical geometry lines, (b) horizontal geometry lines, and (c) structural plate (shield) at vertical geometry line
In addition to the fixities in the 2D cross-section, the entire front and rear boundary planes in the full 3D model are fixed so that no movement is allowed in the z-direction \((u_z = 0)\). Furthermore, boundaries in the 2D cross-section that are fixed in the x- and y-directions are also fixed in the z-direction. Conversely, internal nodes (not located at the boundaries) that are free to move in both the x- and y-directions are free to move in the z-direction.

### 5.4 Mesh

The 2D mesh was created with a relatively fine element configuration near the tunnel boundary (and inside the tunnel for stages where the soil elements have not been deactivated) and a relatively coarse element configuration away from the tunnel, as shown in Figure 5.6. The number of elements per plane is 172, with 3440 elements in the entire 3D model.
The total number of nodes in the 3D mesh is 10,948. The location of these nodes in the 2D representation is shown in Figure 5.7.
Figure 5.8 shows the 3D mesh with its dimensions.

![3D mesh with dimensions](image)

**Figure 5.8. 3D mesh with dimensions**

### 5.5 Material Properties

#### 5.5.1 Material Models

##### 5.5.1.1 Mohr-Coulomb

To bring the behavior of the modeled clay into conformity with what might be observed in real soils, the material properties were based on a normally consolidated Boston blue clay, which is a marine illitic clay found in the area around Boston, Massachusetts. An extensive soil testing program was undertaken at MIT in the early 1960's to evaluate the foundations of the many stately buildings located throughout the campus. The project, called FERMIT (Foundation Evaluation and Research—MIT), obtained data from nearby ongoing construction projects. The
most thorough database from the project was created for the Boston blue clay. FERMIT reports that the soft to medium Boston blue clay has an average plasticity index $I_p$ of 25%.

Skempton’s (1957) empirical relationship for the normalized undrained strength is given by

$$\frac{s_u}{\sigma'_{vo}} = 0.11 + 0.0037 \cdot I_p \%$$

where $s_u =$ undrained shear strength

$\sigma'_{vo} =$ initial effective vertical stress.

A plasticity index of 25% gives

$$\frac{s_u}{\sigma'_{vo}} = 0.20$$

$$s_u = 0.20 \sigma'_{vo}$$

Typically, the ratio of the shear modulus over the undrained shear strength $G/s_u$ is in the range 50-100. Arbitrarily setting this value to 75, we are able to obtain the undrained Young’s modulus $E_u$ according to

$$E_u = 2(1 + \nu_u)G$$

where $\nu_u$ is the undrained Poisson’s ratio. This value is set to 0.495 because a completely incompressible material can cause a singularity in the stiffness matrix, leading to instability in the numerical calculations. Therefore, $E_u$ is given by

$$E_u = 2.99G$$
The effective Young's modulus $E'$ is related to the undrained Young's modulus by

$$E' = \frac{2(1 + \nu')}{3} E_u$$

where $\nu'$ has been chosen as 0.33. The reason for obtaining the effective Young's modulus is because when an undrained analysis is run in PLAXIS, the input program only accepts effective elastic parameters. In contrast, undrained Mohr-Coulomb strength parameters may be entered into the program.

In order to obtain the undrained shear strength profile for the finite element model, the vertical stress profile must first be determined. Assuming a saturated unit weight $\gamma_{sat}$ of 18 kN/m$^3$ and hydrostatic pore pressure conditions with the phreatic surface at grade, the variation in vertical stresses and water pressures with depth were computed, as shown in Figure 5.9.

![Figure 5.9. Vertical stress profile with hydrostatic water pressures](image-url)
From here, the undrained shear strength profile was determined according to the equation above. Instead of having the undrained shear strength go to zero at grade, it was kept constant with depth ($s_u = 2.5 \text{kN/m}^2$) for the uppermost 1.5 m, as shown in Figure 5.10.

![Undrained strength profile](image)

**Figure 5.10.** Undrained strength profile

Similarly, the stiffness profiles (for both effective and undrained) were computed according the equations above. The constant undrained shear strength for the upper 1.5 m causes Young’s moduli to also remain constant with depth over this range. The effective Young’s modulus was kept at 490 kN/m$^2$, as shown in Figure 5.11.
In addition to the stresses and material properties with depth above, the at-rest coefficient of earth pressure $K_0$ was set as 0.5 throughout the entire depth profile.

Table 5.1 shows the soil properties that were used in the Mohr-Coulomb analysis.
Table 5.1. Material properties for soil

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Name</th>
<th>Upper clay¹</th>
<th>Lower clay²</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material model</td>
<td>Model</td>
<td>Mohr-Coulomb</td>
<td>Mohr-Coulomb</td>
<td>-</td>
</tr>
<tr>
<td>Type of material behavior</td>
<td>Type</td>
<td>Undrained</td>
<td>Undrained</td>
<td>-</td>
</tr>
<tr>
<td>Dry unit weight</td>
<td>$\gamma_{unsat}$</td>
<td>18</td>
<td>18</td>
<td>kN/m³</td>
</tr>
<tr>
<td>Saturated unit weight</td>
<td>$\gamma_{sat}$</td>
<td>18</td>
<td>18</td>
<td>kN/m³</td>
</tr>
<tr>
<td>Young’s modulus</td>
<td>$E$</td>
<td>490 (constant)</td>
<td>490 at top (increasing at 326.51 kN/m²/m)</td>
<td>kN/m²</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>$\nu$</td>
<td>0.33</td>
<td>0.33</td>
<td>-</td>
</tr>
<tr>
<td>Undrained shear strength</td>
<td>$s_u$</td>
<td>2.5 (constant)</td>
<td>2.5 at top (increasing at 1.64 kN/m²/m)</td>
<td>kN/m²</td>
</tr>
<tr>
<td>Friction angle</td>
<td>$\phi$</td>
<td>0</td>
<td>0</td>
<td>°</td>
</tr>
<tr>
<td>Dilatancy angle</td>
<td>$\psi$</td>
<td>0</td>
<td>0</td>
<td>°</td>
</tr>
</tbody>
</table>

Notes:
1- Uppermost 1.5 m of deposit
2- Below 1.5-m depth

The input cohesion was derived from the undrained shear strength profile (Figure 5.10), which shows a constant strength of 2.5 kPa throughout the upper clay layer, after which the strength increases with depth at 1.64 kPa/m until a value of 47.5 kPa is reached at a depth of 29 m. Similarly, the input Young’s modulus comes from the stiffness profile (Figure 5.11), in which the stiffness is constant in the upper clay layer and increases from 490 kPa at a depth of 1.5 m to 9469 kPa at a depth of 29 m. The rate of increase with depth is 326.51 kPa/m. The other input parameters were chosen rather arbitrarily, in keeping with a normally consolidated clay, as mentioned above.
5.5.2 Structural Elements

5.5.2.1 Shield

Table 5.2 shows the structural element properties that were used in the analysis.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Name</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of behavior</td>
<td>Material type</td>
<td>Elastic</td>
<td>-</td>
</tr>
<tr>
<td>Normal stiffness</td>
<td>$EA$</td>
<td>$1.00 \times 10^7$</td>
<td>kN/m</td>
</tr>
<tr>
<td>Flexural rigidity</td>
<td>$EI$</td>
<td>$5.00 \times 10^4$</td>
<td>kN.m$^2$/m</td>
</tr>
<tr>
<td>Weight</td>
<td>$w$</td>
<td>38.15</td>
<td>kN/m/m</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>$\nu$</td>
<td>0.00</td>
<td>-</td>
</tr>
<tr>
<td>Interface strength reduction</td>
<td>$R_{inter}$</td>
<td>1.0 (rigid)</td>
<td>-</td>
</tr>
</tbody>
</table>

The normal stiffness $EA$, flexural rigidity $EI$, and weight $w$ were selected based on the material properties of the shield used in the Second Heinenoord Tunnel project (PLAXIS, 2001). It should be noted that the $EA$ and $EI$ relate to the stiffness per unit width and $w$ is the specific weight in units of force per unit area. Furthermore, Poisson’s ratio $\nu$ is set to zero in PLAXIS for long, slender structural elements such as sheet-pile walls and, in this case, cylindrical steel plates. The strength properties of the interface between the shield and the soil were assumed to be equal to the soil strength properties, thus $R_{inter} = 1$.

5.5.2.2 Lining

Table 5.3 shows the concrete lining properties that were used in the analysis.
The elastic parameters $E$ and $\nu$ were also based on the material properties from the Second Heinenoord Tunnel project (PLAXIS, 2001). The interface strength reduction factor was also set to one, as in the shield case. The input dry and saturated unit weights of the lining are higher than standard concrete. In the analysis, the lining with unit weights of 24 kN/m$^3$ tended to drift upward as it approached the rear plane boundary for all loading cases. In order to stop this phenomenon from occurring, the unit weight was roughly doubled, which corresponds to an approximate doubling of the lining thickness. Changing the lining thickness in the problem geometry could also have solved the problem, but this alternative would have been very time-consuming.

### 5.6 Loads/Boundary Conditions

#### 5.6.1 Introduction

As mentioned above, the two loads that will be varied in the analysis are the face pressure and the grout pressure. Figure 5.12 is a schematic showing where these loads will be applied in the model.
In addition to the face pressure and the grout pressure, the jacking pressure will be applied to the lining to simulate the high compressive stresses experienced by the concrete and its effect on the overall system.

### 5.6.2 Face Pressure

The pressure exerted on the face is controlled by the relative amounts of material that enter and exit the spoils chamber. A higher rate of ingress than egress will gradually increase the face pressure and vice versa. Researchers have formulated a relationship between the stresses acting on the face and the undrained shear strength of the soil for a circular tunnel constructed in a homogeneous, plastic clay (Peck, 1969, and Leca, 2000). The overload factor \( N \) is obtained by

\[
N = \frac{\sigma_s + \gamma H - \sigma_r}{s_u}
\]

where
- \( \sigma_s \) = surcharge load
- \( \gamma \) = soil unit weight
- \( H \) = depth to center of tunnel
- \( \sigma_r \) = support pressure applied at the face
\( s_u = \) undrained shear strength.

Figure 5.13 depicts these parameters.

![Figure 5.13. Model for tunnel face stability (after Leca, 2000)](image)

Peck (1969) reported that \( N \) should not exceed about 6 and that tunneling could be carried out without unusual difficulties in plastic clays if \( N \) remains below about 5. He also noted that in shield tunneling, values of \( N \) much greater than 5 may cause the clay to infiltrate the tail void too rapidly so that the annulus cannot be filled with grout satisfactorily. In addition, for values of \( N \) approaching 7, tunnel advance may become slow and difficult because the shield has a tendency to tilt.

In the analysis performed here, different face pressures, corresponding to different values of \( N \), were applied at the face in order to observe the settlement profile for each one. Figure 5.14 shows how the face pressure was applied in the model.
5.6.3 Jacking Force

In order to obtain the jacking force required to move the EPB shield through the soil medium, the frictional force, or drag, along the tail skin must be obtained. The frictional force, denoted $F_{\text{friction}}$, is related to the force exerted on the face $F_{\text{face}}$ by

$$F_{\text{friction}} = F_{\text{jack}} - F_{\text{face}} = \sigma_{\text{jack}} \cdot A_{\text{lining}} - \sigma_f \cdot A_{\text{face}}$$

where

$F_{\text{jack}}$ = thrust force of the shield onto the lining

$\sigma_{\text{jack}}$ = average jack pressure

$A_{\text{lining}}$ = cross-sectional area of lining

$A_{\text{face}}$ = cross-sectional area of face.
The frictional force is given by

\[ F_{\text{friction}} = \mu \cdot N_{\text{friction}} \]

where \( \mu \) = coefficient of sliding friction between the soil and the shield

\( N_{\text{friction}} \) = normal force acting on the shield.

Empirical values of \( \mu \) are reported by Stein et al. (1989). For sand, \( \mu \) is between 0.3 and 0.6; for clay, \( \mu \) ranges from 0.2 to 0.4. For this exercise, the middle value for clay of 0.3 was chosen.

To obtain the normal force acting on the tail skin, Pellet and Kastner (1998) recommend the following relationship that is obtained by integrating the normal stress acting on the external surface of the shield:

\[
N_{\text{friction}} = L \cdot D_e \cdot \frac{\pi}{2} \left[ \sigma_{EV} + \frac{\gamma \cdot D_e}{2} + K_o \left( \sigma_{EV} + \frac{\gamma \cdot D_e}{2} \right) \right]
\]

where

- \( L \) = length of shield
- \( D_e \) = outer diameter of shield
- \( \gamma \) = total unit weight of soil
- \( K_o \) = coefficient of earth pressure at rest
- \( \sigma_{EV} \) = vertical stress at the tunnel crown.

The vertical stress at the tunnel crown is often computed using Terzaghi’s (1951) arching model:

\[
\sigma_{EV} = \frac{b \left( \gamma - \frac{2 \cdot c}{b} \right)}{2 \cdot K \cdot \tan \delta} \left[ 1 - e^{-2 \cdot K \tan \delta \left( \frac{b}{b} \right)} \right]
\]

where \( c \) = cohesion at tunnel crown
Three-dimensional analysis of surface settlement in soft ground tunneling

$h = \text{height of cover at tunnel crown (equivalent to } C \text{ above).}$

The other parameters in the equation have been proposed by a number of researchers, but Terzaghi's empirical values have proven to be the most accurate when checked against field measurements. They are:

$$b = D_e \left[1 + 2 \cdot \tan \left(45^\circ - \frac{\phi}{2}\right) \right]$$

$$\delta = \phi$$

$$K = 1$$

where $\phi = \text{angle of internal friction}.$

Assuming undrained conditions in the soft clay ($c = c_u$ and $\phi = 0$),

$$b = 8.5m \left[1 + 2 \cdot \tan \left(45^\circ - \frac{0^\circ}{2}\right) \right] = 8.5m[1 + 2]$$

$$b = 25.5m$$

This value of $b$ and the undrained shear strength (from Figure 5.10) at the tunnel crown are used to obtain the vertical stress at the crown. Furthermore, taking tangents of the friction angle close to zero,

$$\sigma_{EV} = \frac{25.5m \left(18 \frac{kN}{m^3} - \frac{2 \cdot 17.61kPa}{25.5m} \right)}{2 \cdot 1 \cdot \tan(0)} \left[1 - e^{-2 \cdot \tan(0)\left(\frac{10.55m}{25.5m}\right)} \right]$$
Three-dimensional analysis of surface settlement in soft ground tunneling

\[ \sigma_{EV} = \frac{441.39 \text{kPa}}{2 \cdot \tan(0)} \left[ 1 - e^{-0.827 \tan(0)} \right] \]

\[ \sigma_{EV} = 175.3 \text{ kPa} \]

The normal force acting on the shield is computed by

\[ N_{friction} = 9m \cdot 8.5m \cdot \frac{\pi}{2} \left[ 175.3 \text{kPa} + \frac{18 \text{kN} \cdot 8.5m}{2} + 0.5 \left( 175.3 \text{kPa} + \frac{18 \text{kN} \cdot 8.5m}{2} \right) \right] \]

\[ N_{friction} = 120.2m^2 \left[ 175.3 \text{kPa} + 76.5 \text{kPa} + 125.9 \text{kPa} \right] \]

\[ N_{friction} = 45,392 \text{ kN} \]

The total frictional force along the length of the shield is calculated by

\[ F_{friction} = 0.3 \cdot 45,392 \text{kN} \]

\[ F_{friction} = 13,618 \text{ kN} \]

Knowing the frictional force acting on the shield, the amount of jacking force required is given by

\[ F_{jack} = F_{friction} + F_{face} \]

Because \( F_{friction} \) will be assumed constant in all PLAXIS simulations, \( F_{jack} \) will change for each pressure (force) that will be applied on the face. Figure 5.15 shows that in the PLAXIS model the jacking force is applied on the lining several segments back from where it is actually applied in
the field. The reason for this representation is that two segments are missing behind the shield so that the grouting pressure can be applied. This will be described in the following section.

![Diagram showing jacking force in 3D model]

**Figure 5.15. Application of jacking force in 3D model**

### 5.6.4 Grout Pressure

The grout injected into the tail void is for this analysis assumed to still be liquid over the two lining segments located just behind the shield i.e. from 9 to 15 m. After 15 m, the grout is assumed to be hardened and the concrete lining segments, with no accompanying applied pressure, are activated in the model. The grout is applied as a water pressure in the model in an outward radial direction, as shown in Figure 5.16.
The grout ratio \( M \) is similar to the overload factor and is defined as

\[
M = \frac{p_G}{\sigma_{vo}}
\]

where \( p_G \) = average grout pressure at center of tunnel

\( \sigma_{vo} \) = average total vertical stress at center of tunnel.

Different grout pressures, corresponding to different values of \( M \), were applied to observe the settlement profile for each one.
5.6.5 Ground Loss

The contraction parameter $\alpha$ is an input value used in PLAXIS 3D Tunnel. It is defined as the cross-sectional area reduction per slice as a percentage of the whole tunnel cross-sectional area, which is equivalent to the ground loss $G.L\%$ defined above. For this exercise, it was set to 0.16\% per 3-m section of the shield, which corresponds to a ground loss of 0.48\% over the 9-m shield length, as shown in Figure 5.17. This ground loss can be attributed to either overcutting or slight conicity in the shield. In the latter case, a ground loss of 0.48\% would be consistent with a 20 mm decrease in the shield radius from face to tail.

![Figure 5.17. Application of contraction parameter](image)

5.7 Construction Phases

Fourteen construction stages were used to simulate the movement of the shield into the clay. The shield enters the model in Phase 1 having advanced one slice, after which the assembly moves one slice in each phase. At Phase 14, the face has moved 42 m into the model (shield three slices [9 m], liquefied grout zone two slices [6 m], concrete lining nine slices [27 m]). The configurations of the construction phases, including loading and model geometry changes, are shown in Table A and Figures A1-A14 in the appendix. The model configuration at Phase 14 is
shown in Figure 5.18. Phase 14 corresponds to the stage from which settlement results were analyzed, as will be seen later.

Figure 5.18. Model configuration after Phase 14

5.8 Base Case

It was important to establish a “base” case, where a particular face pressure could be used for all of the simulations in which the grout pressure would be varied, and vice versa. For the face pressure, the goal was to find the case where very small longitudinal deformations were observed at the face; in essence, a true earth pressure balance. After seeing the results from many face pressure variations, the average pressure for the base case was set as 217 kPa ($N = 2$).

The grout pressure, however, was found in a slightly different manner. First, using the base face pressure, the grout pressure was reduced from an initial trial value until the soil body
collapsed, where collapse is defined in the model as a decrease in the magnitude of the applied load in two successive steps. It should be noted that the reason for inward collapse is that the two lining segments are missing at the location where the grout pressure is applied in the model. If the segments were in place, as they are in reality, the soil body would not completely collapse inward, but it would come to rest on the lining. Second, the grout pressure was increased until the soil body failed by outward expansion. Then, these values were averaged to find the base grout pressure of 226 kPa \( (M = 0.850) \). The lower pressure causing the soil body to collapse was 206 kPa \( (M = 0.775) \), while the upper pressure causing failure was 246 kPa \( (M = 0.925) \).
CHAPTER 6. FACE PRESSURE VARIATION

6.1 Model Results

Table 6.1 shows the overload factors and corresponding average face pressures that were modeled using the base grout pressure as described above. Only results from overload factors between −3 and 4 are reported in the table. Overload factors below −3 were not modeled because values in this range were deemed unrealistic to be achieved in the field. Overload factors above 4 were attempted but the soil body collapsed inward through the face opening at \( N = 4.5 \) due to the extremely low face pressure, which was 157 kPa.

![Table 6.1. Modeled face pressures](image)

Overload Factor, \( N \) & Average Face Pressure, \( \sigma_f \) [kPa] & Force on Face, \( F_{\text{face}} \) [kN] & Jack Force, \( F_{\text{jack}} \) [kN] & Average Jack Pressure, \( \sigma_{\text{jack}} \) [kPa] \\
--- & --- & --- & --- & ---
4 & 169 & 9,358 & 22,976 & 2,596
3 & 193 & 10,697 & 24,315 & 2,747
2 & 217 & 12,036 & 25,654 & 2,899
1 & 241 & 13,375 & 26,993 & 3,050
0 & 266 & 14,714 & 28,332 & 3,201
-1 & 290 & 16,053 & 29,671 & 3,353
-2 & 314 & 17,392 & 31,010 & 3,504
-3 & 338 & 18,731 & 32,349 & 3,655

*Note: \( F_{\text{friction}} = 13,618 \) kN for all cases*

Figure 6.1 shows a typical model result at the end of Phase 14. The deformed 3D mesh shows vertical displacements, which are of the most interest in this exercise. One can see the longitudinal deformation profile because the modeling of the half-space allows one to observe the vertical displacement directly above the tunnel centerline. Furthermore, the right half of the transverse settlement profile can be seen in the plane model boundary in view, as shown in Figure 6.1.
Figure 6.1. Typical vertical displacement results after Phase 14 (mesh deformed 20 times; settlement negative, heave positive)

Figure 6.2 shows longitudinal settlement profiles for varying face pressures, where heave is positive and settlement is negative. In addition, the solution from Attewell and Woodman (1982) is included for comparison.
It was described previously that the plane boundaries (z-planes) are fixed so that no displacement is allowed in the z-direction, which causes the ground surface to be exactly horizontal at the boundaries. In the case of the front plane boundary, located ahead of the tunnel face, this boundary effect introduces some inaccuracies in the amount of heave/settlement, especially in the later construction phases as the face approached the front plane boundary. In essence, the higher the face pressure (i.e. the lower the overload factor), the greater the increase in the amount of ground heave ahead of the tunnel face due to the boundary effect. To lessen the adverse result, the longitudinal settlement profile ahead of the face was taken at Phase 7 and combined with the profile behind the face from Phase 14. In this manner, the boundary effects ahead of the face had less impact on the longitudinal profile, as shown in Figure 6.3.

Figure 6.2. Longitudinal settlement profiles for different overload factors
At the rear plane boundary, located behind the advancing shield, a similar problem was encountered, even in the later construction phases when the boundary is farthest from the advancing face. The boundary effect influences the longitudinal settlement profile above the lining for cases in which the applied face pressure is high or low ($N = 3$ and above and $N = 0$ and below). As shown in Figure 6.3, the result was a rather wavy settlement profile near the rear boundary, even though the longitudinal settlement profile is predicted to be relatively constant above the lining far from the face. The boundary effects were ignored by considering the surface settlements above the lining to be constant, as shown for values of $z$ greater than approximately 22 in Figure 6.2.

Figure 6.3. Boundary effects on longitudinal settlement profile

Figure 6.5 shows the amount of maximum surface heave $\delta_H$ and the maximum surface settlement $\delta_S$ with the change in overload factor, where heave is considered positive displacement and
settlement is negative. In addition, the total settlement $\delta_T$ is plotted on the same graph. The total settlement is defined as

$$\delta_T = \delta_H - \delta_S$$

The above surface displacement parameters are shown schematically in Figure 6.4.

In addition, the overload factor was used in Figure 6.5 because it is a more useful and versatile parameter than face pressure for examining typical settlement behavior in clay.
6.2 Discussion

As expected, the amount of surface heave ahead of the face increases with decreasing overload factor, which is equivalent to increasing face pressure. The settlement profiles for $N \leq 1$ exhibit surface heave ahead of the face, while those for $N > 1$ indicate no surface heave. Another observation is that all of the profiles for $N \leq 2$ cross each other at approximately $z = -6$ m and $\delta = -3$ mm. The profiles for $N \geq 3$, on the other hand, are very similar to each other but the settlement starts occurring approximately 10 m ahead of all the other longitudinal settlement profiles, as shown in Figure 6.2.

The estimate by Attewell and Woodman provides a relatively close match to the settlement profiles in the main grouping of profiles in Figure 6.2 i.e. for $N \leq 2$. The closest match to the
solution by Attewell and Woodman was the profile for \( N = 1 \). The ground loss parameter \( \text{G.L}\% \) in their solution was given by

\[
\text{G.L}\% = \left( 1 - \left( 1 - \frac{\alpha}{100} \right)^3 \right) \cdot 100
\]

where \( \alpha \) = contraction parameter used in PLAXIS 3D Tunnel (percentage of ground loss per slice).

The term \( \left( 1 - \frac{\alpha}{100} \right) \) is equal to the new tunnel cross-sectional area after the shield has moved one slice, or 3 m in this case. The term is cubed because the reduction in cross-sectional area for the next slice is represented by the same equation, where the area reduction is based on the already decreased area from the previous contraction. It was assumed that 0.16\% of the shield area is lost per 3-m section and three of these sections comprise the total length of the tail skin. It then must be converted back into “ground loss” terms, giving the above \( \text{G.L}\% \) equation.

Figure 6.5 shows a decrease in maximum surface settlement with a decrease in the overload factor, again equivalent to an increase in face pressure. However, for overload factors less than 1, the trend switches and the maximum surface settlement increases with decreasing overload factor. Other researchers have reported similar findings to what has been discovered in this exercise. Research by Finno and Clough (1985) and Abu-Farsakh and Tumay (1993) concluded that the magnitude of the maximum surface settlement decreases with decreasing overload factor. In particular, Finno and Clough compiled data from the N-2 project site in the San Francisco Bay area, in which the tunnel was excavated through similar soil conditions to the model in this exercise. As shown in Figure 6.6, the data show a decrease in maximum surface settlement with increasing face pressure, but the average pressure was kept relatively low. In fact, it appears that the rate of decreasing surface settlement with increasing face pressure is decreasing at higher face pressures. During N-2 tunnel construction, it was decided not to produce very high face pressures, but the PLAXIS model in this exercise was able to capture the soil response in this upper range. As indicated, the settlement data in Figure 6.6 are plotted
against face pressure only. Because the total vertical stress and undrained shear strength are not known for the locations at which the settlement values were measured, no attempt was made to plot them explicitly versus overload factor.

![Figure 6.6. Surface settlement with varying face pressure for the N-2 project (after Finno and Clough, 1985)](image.png)

Although Finno and Clough report smaller settlements with increasing face pressure, they also conclude that initial heave caused by high face pressures can have deleterious effects on surface structures because the excess pore pressures induced by the high face pressures can cause significant consolidation settlement as these pore pressures dissipate. Unfortunately, this phenomenon could not be substantiated in this exercise because PLAXIS 3D Tunnel does not allow one to calculate consolidation.

Maximum surface settlement is not always the most important factor in minimizing damage to buildings on the surface. The total settlement, as defined above, is important because a building
may heave and then undergo an amount of settlement greater than the difference between its final and initial positions. This additional movement may cause further damage to a structure than the maximum surface settlement might suggest. Figure 6.5 shows that the minimum total settlement corresponds to overload factors of 1-2, in which the total settlement for each is approximately equal. The maximum surface settlement for \( N = 2 \) is slightly greater than for \( N = 1 \), but the maximum heave is greater for \( N = 1 \). Thus, the optimum overload factor is in this range, which in this case corresponds to an average face pressure of approximately 150 kPa.

In addition to the amount of total vertical settlement induced by different face pressures, the tilt of the surface is also significant. Figure 6.7 shows that high and low overload factors produce a steeper slope as the ground settles during shield passage. Steep slopes will lead to greater angles of tilt in surface structures. Again, \( N = 1 \) appears to be the optimum overload factor because it exhibits the gentlest slope.

![Diagram](image-url)

**Figure 6.7. Close view of longitudinal settlement troughs**
According to Peck’s work, the soil body should not have failed inward through the face at an overload factor of 4.5 as it did in this study. Peck reported that the overload factor should not exceed about 6. He assumes, however, that the medium in which the tunnel is driven is entirely homogeneous. The soil profile in this exercise is homogeneous in its normalized undrained shear strength \( (s_u/\sigma'_vo) \) with depth, but heterogeneous in its absolute strength i.e. undrained shear strength increases with depth. This is the probable reason that failure occurred in the model at a lower overload factor than predicted by Peck. Another likely cause for the collapse is attributed to boundary effects and the slight error that is inherent in any numerical modeling analysis.
CHAPTER 7. GROUT PRESSURE VARIATION

7.1 Model Results

Table 7.1 shows the grout ratios and corresponding average grout pressures that were modeled using the base face pressure as described above. As indicated in Section 5.8, failure occurred at lower and upper grout ratios of 0.775 and 0.925, respectively. Therefore, only grout ratios between 0.800 ($p_G = 202$ kPa) and 0.900 ($p_G = 239$ kPa) were modeled in the analysis.

<table>
<thead>
<tr>
<th>Grout Ratio, $M$</th>
<th>Average Grout Pressure, $p_G$ [kPa]</th>
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</thead>
<tbody>
<tr>
<td>0.800</td>
<td>212</td>
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<tr>
<td>0.825</td>
<td>219</td>
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<tr>
<td>0.850</td>
<td>226</td>
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<tr>
<td>0.875</td>
<td>232</td>
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<tr>
<td>0.900</td>
<td>239</td>
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</table>

Figure 7.1 shows the longitudinal settlement profiles obtained by modeling the tunnel with the varying grout pressures.
The interference from the front plane boundary ahead of the face proved not to be as significant as in the face pressure variation case because the applied face pressure was kept at the base level where very little heave was induced. However, a slightly wavy longitudinal settlement profile was observed near the rear plane boundary at higher grout pressures, for which the same procedure was applied as in the face pressure variation case.

Figure 7.2 shows the amount of maximum surface settlement $\delta_N$ with the change in grout ratio, where settlement is considered negative as in Figure 6.4. The reason that maximum surface heave $\delta_H$ and total settlement $\delta_T$ were not plotted on the same graph is because the total settlement $\delta_T$ was nearly equal, in absolute value terms, to the maximum surface settlement $\delta_N$ due to the negligible heave ahead of the face.
Discussion

It can be concluded that grout pressure is a construction parameter that must be kept within a narrow range. The system is very sensitive to grouting pressure, in contrast to the application of face pressure in which the system could accommodate a much wider range of values before the soil body collapsed. However, as previously stated, the soil body collapse at the lower grout pressure limit of 206 kPa ($M = 0.775$) would not occur in a real EPB tunneling operation because the converging soil mass would be supported by the lining segments that are missing in this analysis. On the other hand, it is possible that the 246-kPa grout pressure ($M = 0.925$), which causes failure by outward expansion in the model, might occur at a lower pressure in an actual tunneling operation because tail seals have a limit to the amount of grout pressure they can
withstand. Thus, the grout pressure cannot exceed this maximum amount due to leakage through the seal.

Figure 7.2 shows that increasing the applied grout pressure decreases the maximum surface settlement. In fact, the maximum settlement is increased two-fold as one goes from the lower grout pressure that causes collapse to the higher one. Figure 7.3 shows a fitted curve, from a regression analysis, of vertical settlement as a function of grout ratio according to

\[ \delta_s = -1970.1 \cdot (M)^2 + 3566.7 \cdot M - 1635.3 \]

It can be seen that from the figure that the regression produced a very close fit to the data. This is also evidenced by the very high \( R^2 \) value of 0.9999. It should be noted, however, that the equation can only be applied over the narrow range of grout ratios as indicated in the figure.

---

![Figure 7.3. Vertical settlement versus grout ratio with fitted curve](image-url)
CHAPTER 8. CONCLUSIONS

Longitudinal settlement profiles were obtained using the finite element program PLAXIS 3D Tunnel for an EPB shield in a normally consolidated clay. The face pressure and grout pressures were varied to see how they might influence the magnitude of surface settlements. The important conclusions resulting from this modeling exercise are as follows.

Face pressure variation:
1) As expected, the amount of heave ahead of the tunnel face increases with increasing face pressure.
2) The amount of vertical settlement above the lining decreases with increasing face pressure up to a point, after which it increases slightly. In this simulation, the face pressure at which the transition occurred was 241 kPa \((N = 1)\).
3) The clay through which the tunnel advances can withstand very wide variations in face pressure before collapse occurs. In fact, the upper limit of face pressure was not found. A lower limit was found, in which the soil body collapsed through the face into the tunnel; it was 157 kPa \((N = 4.5)\).
4) Because only an undrained analysis was run, it may be possible that the long-term settlement profiles are greater than those obtained from this numerical modeling simulation. Consolidation settlements take place in the cases where high face pressures have induced elevated pore water pressures, with the settlements occurring over long periods of time as these excess pressures dissipate.

Grout pressure variation:
1) As expected, the amount of vertical settlement above the lining decreases with increasing grout pressure.
2) Only a narrow range of grout pressures can be applied before the soil body collapses. Both the upper and lower limits were found. They were 246 kPa \((M = 0.925)\) and 206 kPa \((M = 0.775)\), respectively.
REFERENCES


APPENDIX
Table A.1. Geometry configuration and water conditions for staged construction

### Geometry Configuration Mode

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**Legend**
- Unchanged U
- Face Pressure F
- Shield Passage S
- Lining Deactivated D
- Jacking Pressure J
- Lining L

### Water Conditions Mode

| Slice Plane | Slice Plane | Slice Plane | Slice Plane | Slice Plane | Slice Plane | Slice Plane | Slice Plane | Slice Plane | Slice Plane | Slice Plane | Slice Plane | Slice Plane | Slice Plane | Slice Plane | Slice Plane | Slice Plane | Slice Plane | Slice Plane | Slice Plane | Slice Plane | Slice Plane | Slice Plane | Slice Plane |
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| **Initial Phase** | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U |
| Phase 1 | D | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U |
| Phase 2 | D | D | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U |
| Phase 3 | G | G | D | D | D | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U |
| Phase 4 | G | G | G | G | D | D | D | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U | U |
| Phase 5 | D | D | G | G | G | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D |
| Phase 7 | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D |
| Phase 8 | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D |
| Phase 9 | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D |
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| Phase 14 | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D | D |

**Legend**
- Unchanged U
- Dry D
- Grout Pressure G
Figure A1. Phase 1

Figure A2. Phase 2
Figure A3. Phase 3

Figure A4. Phase 4
Figure A5. Phase 5

Figure A6. Phase 6
Figure A7. Phase 7

Figure A8. Phase 8
Figure A9. Phase 9

Figure A10. Phase 10
Figure A11. Phase 11

Figure A12. Phase 12
Figure A13. Phase 13

Figure A14. Phase 14