Ahmed Marwan

Computational Analysis of Segmental Linings in Mechanized Tunneling
Computational Analysis of Segmental Linings in Mechanized Tunneling

by

M.Sc. Ahmed Marwan

Dissertation

for the degree

Doctor of Engineering (Dr.-Ing.)

Institute for Structural Mechanics
Faculty of Civil and Environmental Engineering
Ruhr University Bochum

Bochum, April 2019
Mechanized shield tunneling by means of tunnel boring machines (TBMs) is a widely used construction method, since it has proven itself as an effective, fast and safe process with a wide scope of applications (e.g. shallow depths, loose soil with low bearing capacity or high ground water level). A primary component in mechanized tunneling is the segmental lining, which is designed to fulfill basic structural, serviceability and durability requirements throughout the lifetime of the tunnel. A prerequisite for a reliable numerical analysis of segmental tunnel lining is the accurate assessment of the lining response with respect to the external as well as the process loads to which the lining structure is subjected. However, only limited insight is available up to date on the actual interactions between the ground response due to the tunnel advancement, the tail void grouting and the response of the segmented tunnel linings.

In the thesis, the 3D process oriented finite element model ekate is applied to simulate the advancement and the excavation process along arbitrary alignments, i.e. straight or curved paths, and to realistically capture the mechanisms involved in the soil-lining interaction by considering the pressurization and hydration induced stiffening of the grouting material in the annular gap. In addition, the computational model is utilized to assess the segmental lining response due to tunneling induced construction loads and the respective structural forces in the lining. To consider the segmentation of the lining system, which implies a non-trivial kinematics along joints, a technique is developed for modeling the segment-wise installation of tunnel linings in the 3D simulation model. The segments of the lining ring are explicitly modeled as separate bodies, and the interactions between segments at the longitudinal and ring joints are modeled by means of a surface-to-surface frictional contact formulation. Using this technique, the temporal sequence of the loads, transferred via the tail gap grouting, from the soil to the lining as well as the relative actions between the joint of segments can be taken into account.

The results obtained from the 3D computational model, which takes the construction process and the interactions between the individual lining segments as well as the interactions between the lining shell and the grouting and the surrounding soil, respectively, into account, provide a better insight into the effect of these mutual interactions. The analyses also provide insights into the extent to which, the full scale modeling, with a higher level of detail, plays a role in regards to tunnel lining design. The influence of the joint arrangement and segmentation are investigated by comparing the results from the 3D computational model with a standard continuous lining modeling technique.
The thesis also investigates the suitability and validity of commonly used segmental lining models and their corresponding loading assumptions by contrasting with results from the computational process simulation for straight and curved tunnel paths. From this comparison, useful conclusions for modeling segmental linings are drawn.
Acknowledgements

The research work presented in this dissertation has been carried out during my research stay at the Institute for Structural Mechanics, Faculty of Civil and Environmental Engineering, Ruhr University Bochum. This period has been a truly life-changing experience in my life.

First, I would like to express my special appreciation and gratitude to my supervisor and principal referee Prof. Dr. techn. Günther Meschke for providing me an opportunity to join his research group. His immense knowledge has enlightened me on the world of computational mechanics. With his invaluable support, guidance and patience, I was able to complete this thesis. I also wish to express my sincere thanks to Prof. Dr. Peter Mark for accepting to be my second referee. My deepest gratitude goes to my third referee Prof. Dr. Mostafa Zaki not only for being a referee for my thesis, but also for his continuous guidance and support throughout my career. I am also grateful for the successful role model that he has provided me.

I would like to thank all my colleagues at the Institute for Structural Mechanics, Ruhr University Bochum and the colleagues at the Collaborative Research Center (SFB 837), as well my colleges in Egypt at the Department of Civil Engineering, Minia University. Big thanks go to Abdullah Alsahly for his wise guidance and help, as well to Sahir Butt for helping me to finalize my thesis.

I would also like to acknowledge the Missions Department, Egyptian Ministry of Higher Education, for the financial support in the first two years of my research stay in Germany. I also acknowledge the financial support from the German Research Foundation (DFG) through the Collaborative Research Center (SFB 837) that enabled me to continue my doctoral study in Germany. Herewith, I acknowledge the support from the Egyptian Cultural Bureau in Berlin for their kindness and help during my stay in Germany.

Finally, special thanks are reserved to my family. To my parents, thanks for your patience, your prayer and all the sacrifices that you have done for me. To my brother, thanks for your support and for being my strength. To my first little love Karim, thanks for being the joy of my life and of course for waking me up in the night. Last but not the least, the most special thanks go to my beloved wife. Thanks for your support, encouragement and understanding, as well for making my journey in life worthwhile.

Bochum, April 2019

Ahmed Marwan
# Contents

Abstract ii

Acknowledgements iii

Contents v

## 1 General Introduction

1.1 Motivation and Background .................................................. 1
  1.1.1 The Role of Numerical Modeling in Tunnel Design .......... 5
  1.1.2 Analysis of Segmental Tunnel Lining ......................... 7
1.2 Aims and Objectives ...................................................... 8
1.3 Overview of the Thesis ................................................... 10

## 2 Computational Modeling in Mechanized Tunneling

2.1 Construction Procedure ................................................. 12
2.2 Ground Support in Shield Tunneling ................................. 13
  2.2.1 Heading Face Support .............................................. 15
  2.2.2 Annular Gap Grouting ............................................. 16
  2.2.3 Infiltration Process in the Soil ................................. 17
2.3 Computational Modeling for Shield Tunneling Process ............ 21
2.4 ekate: Enhanced Kratos for Advanced Tunneling Engineering ..... 31
  2.4.1 Model Components .................................................. 32
  2.4.2 Representation of Support Pressures ......................... 41
  2.4.3 Shield Steering .................................................... 42
  2.4.4 Pressurized Fluid Film within the Steering Gap .......... 44
  2.4.5 Simulation of the Construction Process ..................... 46

## 3 Analysis of Segmental Tunnel Lining

3.1 Introduction ............................................................. 49
3.2 Basic Characteristics ..................................................... 52
  3.2.1 Segmental Lining Joints ......................................... 55
<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.3 Structural Models for Segmental Tunnel Linings</td>
<td>59</td>
</tr>
<tr>
<td>3.3.1 Analytical Solutions</td>
<td>59</td>
</tr>
<tr>
<td>3.3.2 Numerical Models</td>
<td>65</td>
</tr>
<tr>
<td>3.3.3 Concluding Remarks</td>
<td>73</td>
</tr>
<tr>
<td>4 Evaluation of Lining Response using ekate Model</td>
<td>75</td>
</tr>
<tr>
<td>4.1 Model Description</td>
<td>75</td>
</tr>
<tr>
<td>4.2 Geological Conditions</td>
<td>78</td>
</tr>
<tr>
<td>4.2.1 Soil Material Behavior</td>
<td>78</td>
</tr>
<tr>
<td>4.2.2 Coefficient of Lateral Earth Pressure</td>
<td>80</td>
</tr>
<tr>
<td>4.2.3 Level of Ground Water Table</td>
<td>81</td>
</tr>
<tr>
<td>4.3 Shield Design Parameters</td>
<td>83</td>
</tr>
<tr>
<td>4.3.1 Shield Overcut and Conicity</td>
<td>84</td>
</tr>
<tr>
<td>4.3.2 Shield Friction with the Excavated Soil</td>
<td>86</td>
</tr>
<tr>
<td>4.4 Annular Gap Grouting</td>
<td>87</td>
</tr>
<tr>
<td>4.4.1 Grouting Pressure</td>
<td>87</td>
</tr>
<tr>
<td>4.4.2 Time Dependent Properties of Grouting Material</td>
<td>88</td>
</tr>
<tr>
<td>4.5 Advancement along Curved Alignments</td>
<td>90</td>
</tr>
<tr>
<td>4.6 Evaluation of Acting Loads and their Comparison with In-situ Loading Assumption</td>
<td>92</td>
</tr>
<tr>
<td>5 Representation of Joint Behavior Using Contact</td>
<td>95</td>
</tr>
<tr>
<td>5.1 Segment-wise Lining Installation in ekate</td>
<td>95</td>
</tr>
<tr>
<td>5.1.1 Lining-Soil Interaction</td>
<td>98</td>
</tr>
<tr>
<td>5.2 Computational Contact Mechanics</td>
<td>99</td>
</tr>
<tr>
<td>5.2.1 Mathematical Description of Contact Problem</td>
<td>99</td>
</tr>
<tr>
<td>5.2.2 Constraint Enforcement by the Penalty Method</td>
<td>101</td>
</tr>
<tr>
<td>5.3 Implementation of Contact Algorithm in KRATOS</td>
<td>103</td>
</tr>
<tr>
<td>5.3.1 Weak Formulation and Linearization</td>
<td>104</td>
</tr>
<tr>
<td>5.3.2 Verification of Frictional Contact Behavior</td>
<td>106</td>
</tr>
<tr>
<td>5.4 Model Validation</td>
<td>108</td>
</tr>
<tr>
<td>5.4.1 Concrete Joint Test</td>
<td>108</td>
</tr>
<tr>
<td>5.4.2 Full-Scale Test of Botlek Railway Tunnel (BRT) Segments</td>
<td>110</td>
</tr>
<tr>
<td>6 Numerical Assessment of Different Lining Models</td>
<td>117</td>
</tr>
<tr>
<td>6.1 Geometrical Configuration and Properties</td>
<td>117</td>
</tr>
<tr>
<td>6.2 Segmental Lining Model Embedded within the Process Oriented Simulation</td>
<td>119</td>
</tr>
<tr>
<td>6.2.1 Continuous and Segmental Lining Models</td>
<td>121</td>
</tr>
<tr>
<td>6.2.2 Influence of Tunnel Overburden</td>
<td>123</td>
</tr>
<tr>
<td>6.2.3 Influence of Joint Arrangement</td>
<td>125</td>
</tr>
<tr>
<td>6.3 Bedded Beam Model</td>
<td>127</td>
</tr>
</tbody>
</table>
Chapter 1

General Introduction

1.1 Motivation and Background

The world’s population is increasing at a staggering pace and cities are urbanizing very quickly. As shown in figure 1.1, the percentage of world’s population living in urban areas is continuously growing. Rapid population growth and spatial expansion of densely populated urban areas increase the demands for more efficient transportation infrastructures and services. Consequently, the use of the underground space constitutes a key factor for a sustainable and environmentally friendly economic development. For countries with a rapidly increasing population, it provides means for efficient transport of people and goods while reducing the traffic congestion and the associated pollution and noise. Transportation had a significant impact on human life and is considered an integral part of cities’ economic growth, in particular, in densely populated urban areas. An adequate planning for the transportation infrastructure is of crucial importance and inability to keep pace with its increasing demands threatens the economic productivity and overall quality of living.

Figure 1.1: Changes in urbanization across several regions or subregions of the world from 1950 to 2015; black line is the urbanization in developing countries between 1800 to 2015 as a base line for comparison [taken from (DESA 2018) ].
Underground space grants an extra dimension to work with, which provides a significant contribution in urban planning. The use of this extra dimension by tunneling presents a sustainable solution for the increasing density of the subsurface infrastructure. However, tunneling projects involve major financial costs as well as the collaboration of many experts from different fields. The final decisions result from the integration of all these fields. For instance, the choice of the tunneling technique to be used, e.g. cut-and-cover, conventional methods or mechanized tunneling, is one of the major early decisions. Among these different construction methods, there is no absolute optimal technique to construct a tunnel that can generally replace all other methods. Each project has its distinct characteristics and the selection of an adequate tunneling method is dependent on the factors which are unique for every project (e.g. geology, project size, site condition, costs, construction time, etc.). Cut-and-cover construction has a limited use; it is only feasible for the construction of shallow tunnels in areas where the surface activities can be disrupted. Conventional tunneling overcomes such difficulty and provides a more flexible and cost effective tunneling process, especially for relatively short tunnels, since the cost of the required equipment is relatively low.

Mechanized shield tunneling is a widely used construction method, and it has proven itself as an effective, fast and safe construction process. It has a wide scope of application (e.g. shallow depths, loose soil with low bearing capacity or under ground water level). It enables a safe construction of shallow tunnels in urban environment with minor disturbance of ground surface settlements. Compared to other tunneling methods, mechanized tunneling requires sophisticated equipment, i.e. a Tunnel Boring Machine (TBM) as shown in figure 1.2. A long manufacturing time along with a high cost is involved in the production of a TBM. However, the high excavation rate offered by a TBM provides an economic balance in the construction of long tunnels ($\geq 2.0km$). Recently, TBMs are not only used for long drives, but they became, with their efficient ability of controlling ground deformations, preferred solution for shallow urban tunnels in soft ground (EISENSTEIN 1999).

Figure 1.2: Representative figure of Mixshield components: (1) cutting wheel, (2) submerged wall, (3) air cushion, (4) jaw crusher, (5) bulkhead, (6) air lock, (7) slurry circuit, (8) thrust cylinders, (9) shield skin, (10) erecter, (11) wire brushes and (12) Backfilling (TBM ©Herrenknecht AG)
TBM refers to all types of shield machines in most English literature. But, the German Tunneling Committee (DAUB) (2010) differentiates between different types of machines, for e.g. tunnel boring machines (TBM), shield machines (SM), double shield machines (DSM) and combination machines (KSM). These different machines mostly perform similar functions, i.e. cutting process, muck removal, face support and machine advancement (GERMAN TUNNELLING COMMITTEE (DAUB) 1997, ITA WORKING GROUP 14 2000), and the distinction between them is based on the way these functions are performed. Modern tunnel construction has benefited from the continuous development of TBM technology, it has improved the tunneling efficiency and expanded the application range of TBMs. Since the early 1990s, mega TBMs, with diameters $\geq 14$ m have been used in different projects. In return, this has led to an expansion in the use of TBMs in tunneling, in particular for challenging projects such as Tuen Mun-Chek Lap Kok subsea highway link in Hong Kong with a diameter of 17.6 m (the world largest shield), Alaskan Way Tunnel in Seattle, USA with a diameter of 17.4 m and the Brenner Base Tunnel in Europe (the world longest underground railway connection).

In Germany, there has been a noticeable increase in mechanized tunneling activities over the last 10 years. A total amount of 182 km of transportation tunnels were under construction at the turn of the year 2016/2017, out of which 44.9% of the driven length was constructed using shield tunneling methods, see figure 1.3. According to SCHÄFER (2017), there has been an increase in the number of the urban and rapid transit tunnels planned; 30 km tunnels is being planned for the city of Munich, 10 km for Hamburg Metro and pre-planing of 7 km in Leipzig. Further tunneling activities for shorter tunnels ($< 3$ km) are also foreseen in different German cities.

**Figure 1.3:** Statistics of the individual construction methods of tunneling activities in Germany during the period 2016/2017 (SCHÄFER 2017)

In Egypt, tunneling activities have undergone a remarkable development over the last years. Several major tunneling projects were executed, these include e.g. Greater Cairo metro lines I-III, Al-Azhar twin tunnels and Suez Canal tunnels, and many underground parking garages were constructed to overcome the alarming traffic problems. In Greater Cairo, six metro lines with a total length of 207 km were planned, see figure 1.4. Some metro lines are in operation and the rest are under construction or feasibility study in order to meet the transportation demands in Greater Cairo until the year 2032 according to the Egyptian National Authority of Tunneling (NAT).

Under Suez canal, further projects are currently under consideration which can be seen as a major contribution to serve the investment projects established east of Suez Canal and to develop
Sinai Peninsula. Four road tunnels, two at the city of Port-Said and two at the city of Ismailia passing under Suez canal are recently constructed. Plans exist for two additional Railway tunnels at these locations. With an exception of Cairo metro line I, shield tunneling is adopted as the main construction method for these tunnels in Egypt.

![Greater Cairo metro lines](image)

**Figure 1.4:** Metro lines I-VI in Greater Cairo. Some metro lines are currently in operation and the rest are under construction and planning phases. Feasibility studies are performed by SYSTRA 1998/2000 and JICA 2000/2002 (©Egyptian National Authority of Tunneling)

Although shield tunneling is considered a safe and reliable tunneling approach, serious problems are sometimes recorded during construction or over its lifetime. These problems can be related to different aspects, e.g. improper design, inappropriate construction technique, unexpected geotechnical conditions or lack of sufficient maintenance. Cave-in collapses, tunnel flooding, lining damage and excessive deformations are potential hazards during tunnel construction (LANDRIN ET AL. 2006), these problems can lead to delays in the project and an increase of costs (BRAACH 1992, EFRON AND READ 2012). In recent years, some drastic failures have occurred during tunneling such as Rastatt tunnel in Germany (2017), Sasago tunnel in Japan (2012), Hengqin tunnel in Macau (2012), Mizushima subsea tunnel in Japan (2012), Blanka tunnel in Czech Republic (2010), Cairo metro line in Egypt (2009) and Cologne metro line in Germany (2009).

One important aspect in any tunneling project is to ensure that the tunnel lining, which is the main structural component will not get damaged. For 51 case histories of tunnel repairs according to ITA WORKING GROUP 6 (2001), deterioration of tunnel lining is observed as a result of constructional effects (e.g. inadequate bedding, very large earth pressure, eccentric jacking forces and erection damage), operational conditions (e.g. squeezing ground pressure, thermal effects), leakage,
corrosion of reinforcement and aging of construction material. For the case of segmental lining in mechanized tunneling, Sugimoto (2006) discussed the possible causes of damage in segments during construction; it was pointed out that inadequate design may lead to damage and therefore it is necessary to consider the actual construction loads during the design of segments. Even with proper design and implementation, the ongoing construction of tunnels shifts the problem to maintenance as it becomes necessary to keep up and rehabilitate the existing tunnels. Therefore, regular maintenance is a must to avoid the loss of structural integrity which may occur due to different reasons. On the other hand, the suitable rehabilitation method can be identified with respect to the situated damage level (Abd-Elrehim 2018). Consequently, modern design concepts should ensure that the underground structure (i.e. lining and waterproofing systems) is robust enough, that a minimal maintenance is required.

For these reasons, strong technical and economical motivations exist behind the improvement of tunnel design and construction process through better understanding of the holistic process, in particular the response of tunnel lining. This can only be achieved by precise monitoring systems and by the use of detailed structural models in design.

1.1.1 The Role of Numerical Modeling in Tunnel Design

Designing a tunnel is a quite complex and challenging task, hence, careful planning and suitable decision making are a necessity. In the design phase, proper analyses form an essential part for safe and economic tunnel construction. They should depict the "real" behavior of the tunneling processes during the construction and operation phases, e.g. accurate prediction of ground deformations, tunneling effects on adjacent structures and response of tunnel lining. In addition to that, the analyses should also foresee possible hazards to ensure the overall stability of the tunnel.

Over the last few decades, several analytical and empirical approaches have been developed for the prediction of tunneling induced settlements (Atkinson and Potts 1977, Bobet 2001, Peck 1969, Sagaseta 1987). Methods have also been developed for the prediction of the structural forces in the tunnel linings (Ahrens et al. 1982, Blom 2002, Duddeck 1980). Although, these methods are still being adopted in several international standards and design codes, they have several limitations since they incorporate various simplifications. As a result, it is difficult, or even impossible, to replicate the actual problem, in particular complex scenarios that frequently arise in tunneling projects. For instance, the soil-lining interaction is usually oversimplified with respect to the nonlinear spatio-temporal response of the geo-materials (Potts 2003). Moreover, the stress state in the ground medium is highly influenced by the excavation geometry and the applied supporting measures. Therefore, the primary assumption in many analytical solutions, which includes the use of spring stiffness to represent the soil-lining interaction, is questionable Grose et al. (2005), Potts (2003). In addition, the individual analysis of each component of the tunneling problem without observing the mutual interaction may results in over- or under-estimation in design.

On the other hand, the numerical models, being constantly boosted by the advancement in computer powers, provide an alternative solution for complex tunneling problems that overcome such limitations (Grose et al. 2005, Schweiger 2008). Numerical models are able to take into account the complex nature of tunneling problems at various temporal and spatial scales. They can
include the different components of a tunnel while accounting for complex geometries, mutual interactions between different components, the non-linear material response, etc. Furthermore, the development of commercial softwares, such as PLAXIS, MIDAS GTS, FLAC 3D and ABAQUS, as well as in-house codes have expanded the use of numerical models in the research and in the practice allowing for more in-depth analysis of the problem. It should be noted that, it is the engineer’s responsibility to ensure that correct model is being implemented with a suitable analysis method and an adequate constitutive law. A proper numerical model, in comparison with the classical models, gives more reliable results and supports the engineer to better understand the problem in hand.

With regard to the simulations of the tunneling process, progressive development in numerical models has increased their role during the planning as well as the construction phase. For conventional tunneling, different 2D and 3D models are being used, their characterization is based on different assumptions, methods and modeling techniques, see e.g. (BEER 2003, KARAKUS 2007, SWOBODA 1990). For shield tunneling process, models have been developed with an aim to simulate the holistic construction process of the shield advance in soft soils (e.g. DO ET AL. 2014a, MESCHKE ET AL. 2011, MÖLLER 2006) or in hard rocks (e.g. HASANPOUR 2014, ZHAO ET AL. 2012). Detailed discussion on the existing 3D models for mechanized tunneling is presented in section 2.3. Such models are indispensable tools employed for more reliable analyses and for a detailed exploration of specific aspects during the tunneling process: e.g. interaction with surface structures (BOLDINI ET AL. 2016, LOSACCO ET AL. 2014, NINIČ 2015, OBER ET AL. 2018a), interaction between adjacent tunnels (DO ET AL. 2014b), tunneling intersections (ABD-ELREHIM 2003), tunnel face stability (ALSALHLY 2017, KIRCH 2009), ground improvements (EID 2011, MARWAN ET AL. 2016, ZHOU 2015), infiltration process in soil (LAVASAN ET AL. 2017, ZIZKA ET AL. 2016) as well as the damage induced in the tunnel lining (GALL 2018, ZHAN 2016).

Another significant challenge in tunneling is the reliable assessment of safety against tunneling induced damage to the subsurface infrastructure. Accurate prediction of the soil-building interaction, in particular for the buildings with historical and cultural significance, is a prerequisite for safe and economic construction (BOLDINI ET AL. 2016, GIARDINA 2013, LOSACCO ET AL. 2014). In many cases, the application of empirical methods is insufficient and the failure is over predicted (OBER ET AL. 2018a). Even for the standard process oriented models, disregarding the interaction between the building and the ground often leads to unrealistic predictions. Therefore, models with higher level of detail, i.e. including a detailed discretization of the subsurface infrastructure (NINIČ 2015), allow for an accurate prediction of the ground deformation and accordingly more accurate assessment of possible building failure. If the predicted building damage or the predicted ground deformations exceeded the acceptable limits, ground movement can be controlled by means of supporting measures such as grouting, forepoling or ground freezing. The realization of supporting measures in numerical simulations increases the model complexity, in particular for the freezing process, in which coupled thermo-hydro-mechanical formulation is required for reliable representation of the problem ZHOU AND MESCHKE (2013). Considering the additional costs connected with the additional supporting measures, there is a strong economic interest to minimize the costs by minimizing the energy costs for the soil freezing (MARWAN ET AL. 2016) or the grouting volume of cementitious grout (EID ET AL. 2010).
With the ongoing progress in the complexity of the physics being taken into account by these numerical models, the move toward digitalization as well as the sophisticated data management tools have influenced the way to store, visualize, access, process and manage project data. Digital data is generated for a project by different project partners and is stored in different formats. As a result, noticeable errors occur during data exchange between different partners. Building Information Modeling (BIM) overcomes such deficiencies by providing an organized workflow during the entire lifecycle of the project, i.e. planning, design, construction, operation and maintenance (EASTMAN ET AL. 2011). BIM ensures universal data structure by using standardized exchange formats e.g. Industry Foundation Classes (IFC) (ISO 2013). Although BIM methods have been originally applied to Buildings, they have also been applied to tunneling projects (BORRMANN ET AL. 2015, HEGEMANN ET AL. 2012, KÖNIG ET AL. 2016, SCHINDLER ET AL. 2014), which has been referred to as Tunnel Information Model (TIM). In this context, coupling numerical simulation with BIM facilitate the pre- and post-processing in structural analysis. By doing so, the structural mesh can be generated from geometrical data, materials from object properties, simulation steps form the planned time schedule, etc. The output results of the simulation can be connected to a graphical representation for better visualization, coordination and decision making for the projects. To some extent the numerical modeling process, with the assistance of BIM techniques, can be efficiently automatized.

1.1.2 Analysis of Segmental Tunnel Lining

The tunnels are designed usually with a life span ranging between 100-150 years. With regard to the structural analysis of tunnel lining, reliable analysis is an important aspect to ensure the structural integrity during construction as well as over the expected lifetime. Therefore, models used in design must be able to replicate the dominant physical features that result in the observed lining response in the field. In addition to that, the loading assumptions must accurately represent the actual in-situ time-dependent processes and ground loading to which the lining is subjected. Yet, many models used in lining design are significantly simplified for the sake of computational efficiency in the design process; this include the analytical models in (AHRENS ET AL. 1982, DUDDECK 1980, EL-NAGGAR AND HINCHBERGER 2008, ERMANN 1983, LEE ET AL. 2001, WOOD 1975) or the bedded lining models in (ARNAU AND MOLINS 2011, 2012, BLOM ET AL. 1999, GERMAN TUNNELLING COMMITTEE (DAUB) 2013, JSCE-TUNNEL ENGINEERING COMMITTEE 2007, KLAPPERS ET AL. 2006). These models are intended to study solely the lining behavior, which enables more detailed analysis of the lining (e.g. GALL 2018, GALVAN ET AL. 2017, PUTKE 2016). Still, such models tend to neglect the complexity involved in the tunneling problems such as the 3D arching effect in the ground, the non-linear response of the soil, the temporal response of the grout, the soil-lining interaction, the stress relaxation during excavation, the actual loads acting on the lining and the jack thrust forces. As a result, this leads to improper estimation of the stress state in the lining and consequently, the final design lacks the sufficient reliability.

As discussed earlier, the potential predictive capabilities of the 3D models of the holistic tunneling process overcome the aforementioned limitations. With correct implementation, they can offer a reliable estimate of the actual loads and the reaction of the lining. However, large compu-
tional effort is required to perform such 3D simulations. Hence, some aspects in lining are often simplified such as the lining kinematics, the detailed lining geometry including recess, the material response of concrete, the influence of reinforcement. In addition to that, the erection of individual segments with the loading and unloading of the thrust forces and the sequential pressurization as the ring steps out of the shield, are some of the real-life scenarios that are usually overlooked in the 3D process oriented simulations. Continuous monolithic cylinder with linear elastic response is a basic assumption adopted in many models (e.g. ALSAHLY ET AL. 2016, KASPER AND MESCHKE 2004b, LAMBRUGH ET AL. 2012, MÖLLER AND VERMEER 2008, NINIĆ AND MESCHKE 2017, ZHAO ET AL. 2017). Some effort has also been placed into the investigation of ring segmentation in which segments are simplified by shells elements (e.g. DO ET AL. 2014a, KAVVADAS ET AL. 2017) or by volume elements (e.g. CHENGHUA ET AL. 2016). For the investigation of potential segment damage, YE AND LIU (2018) presented a 3D model in which certain region of the lining is realized with a higher level of detail (i.e. lining segmentation, reinforcement and damaged plasticity of concrete).

One of the key challenges in lining design is the basic understanding of the soil-lining interaction. In addition, it is necessary to take into account the influence of the major design parameters on this interaction, in order to have a reliable construction. The current work outlines the different modeling approaches for the analyses of the segmental tunnel linings from which a comparison can be made between the various approaches. The analyses are performed using models with an increasing level of detail, i.e. a basic bedded beam model, a 3D continuum model for continuous lining and finally a 3D continuum model which takes into account, not only the individual rings forming the tunnel lining, but also the individual segments that are used to construct the ring. The 3D lining analyses are performed using the Finite Element Method and the interactions between segments at the longitudinal and ring joints are modeled by means of a penalty-based, surface-to-surface frictional contact algorithm. The reliability of the final result from a model depends on the level of detail being taken into account and its capability to replicate the actual physics of the real conditions. In this regard, prognoses of the time-variant response of the excavation procedure and the actual loads acting on lining demonstrate the role of the adopted 3D computational continuum model. However, if the potential damage in the individual segments is to be investigated, an even higher spatial resolution, in combination with a suitable material constitutive relation, is required. To this end, the main contribution of this work is to compare, highlight and promote the understanding of the different modeling strategies available for the structural analysis of tunnel lining. Specifically, the capability of the numerical methods to provide a reliable design for the lining response under the constructional loads, is investigated taking different level of details into account.

1.2 Aims and Objectives

In this contribution, the development and predictability of 3D computational models for mechanized tunneling problems is discussed. The existing numerical model (NAGEL 2009, STASCHEIT 2010) is extended to provide efficient and reliable solutions to complex applications encountered during tunneling practice. The presented work aims to further promote the existing model for mechanized
tunneling simulations, in order to provide a more in-depth understanding of the response of precast segmental concrete lining. Therefore, a high level of detail for the representation of the segmental lining, including the joints between the segments, is utilized in this thesis. Using such modeling methodology, it is possible to investigate more reliable lining kinematics, while taking into consideration the actual loading state from the 3D process oriented simulation. Additionally, a comparison is made between the available modeling approaches and the effectiveness of the simplified model, i.e. bedded beam model, is investigated. It is shown that the reduction of the model complexity does not always lead to a conservative estimate. The literature review of the existing structural models for the tunnel lining and the results discussed in this thesis report the advancements in the analyses of segmental tunnel linings in order to enhance design reliability.

From the aforementioned aims, the core objectives of this thesis can be summarized as:

- Review and compare the available simulation models of mechanized tunneling in the pertinent literature to assess the capabilities and limitations of these models, and to highlight the areas for improvements. For instance, the incorporation of BIM concepts into the mechanized tunneling simulations, by which it is possible to streamline and simplify the analysis procedure by using geometrical BIM sub-models as a basis for performing structural calculations.

- Present the existing structural models and loading assumptions used in practice for the analysis of tunnel lining to provide the theoretical background with which the developed lining model within this thesis is compared. Specific detail is dedicated to the representation of joints in the simulation of segmental lining.

- Use the 3D process oriented simulation model in order to evaluate the actual time-variant loading on lining during construction. The simulation model includes most of the relevant components related to mechanized tunneling which serves as a basis for a reliable prediction of loading on the lining. A parametric study is performed to investigate the influences of various parameters on the model response.

- Develop a segmental lining model within the 3D process oriented simulation, that includes the longitudinal and ring joints. The use of a process oriented model for tunnel advance accounts for the actual loading condition during construction, while, the consideration of joints improve the representation of lining kinematics. To this end, the inclusion of joints is achieved by frictional contact algorithm that is validated by means of measurements from laboratory tests.

- Perform structural analysis of segmental lining with different level of details. The first class of models are the continuous lining model and the proposed segmental lining model in a 3D simulation. The second class of models includes the bedded beam model, it uses simplified loading assumptions and depicts the lining-soil interaction by means of elastic springs. From the different modeling approaches for the investigation of the segmental lining response, a comparison can be drawn, which aims to investigate, if, and to which extent the more precise modeling of lining plays a role in regards to structural forces in lining and the consequently lining design.
• Extend the applicability of the simulation model to assess the building-tunnel interaction. First, a three-step damage assessment concept, adjustable to the necessary level of detail is presented. In which, the simulation model including the building with a detailed description serves as a basis for the highest level of damage assessment.

• Present the computational framework for the simulation of ground improvement in tunneling applications by means of Artificial Ground Freezing. For an economic design, an optimization algorithm using Ant Colony Optimization is introduced to find the optimal positions of freezing pipes. The optimized arrangement leads to a substantial reduction of freezing time and consequently the energy costs.

1.3 Overview of the Thesis

Chapter 2 provides a review for the development of the 3D numerical modeling in mechanized tunneling and introduces the simulation model ekate and its components briefly. In chapter 3, an overview of the available structural lining models is provided, with focus on the strategies to include the joints between the segments and the assumptions used for the loading conditions. Following this, the efficiency and the limitation of the different models can be outlined.

In this context, a reliable analysis of tunnel lining should generally comprise two aspects; first, realistic simulation of the construction process and the actual loading on lining (focus of chapter 4). Second is the consideration of lining kinematics (focus of chapter 5). In chapter 4, the model response with respect to various input parameters is investigated in order to quantify their influences. The results provide a better insight into the loading on the lining and the induced structural forces in the lining at the steady state after construction. In chapter 5, the segmental lining realization in the simulation model is improved by the inclusion of joints by means of frictional contact algorithm. Additionally, the modeling methodology is validated by numerically reproducing the experimental measurements from literature. Following this, the influence of the joint arrangement and the segmentation are investigated in chapter 6 through comparison with simulations in which a standard, continuous lining modeling technique is employed. As well as, comparison with the response of the classical bedded beam model is provided. The results give a more detailed insight into segmental lining analysis as a step forward to enhance and update the reliability of the current design.

In chapter 7, advanced applications of the ekate model are discussed, this includes the use of BIM concepts in tunneling process simulations, multi-stage assessments of tunneling-induced building damage and simulation of artificial ground freezing. Finally, the thesis closes with conclusions and outlooks in chapter 8. Further, appendices for the calculation of structural forces in lining, the transformation of loading on lining between vertical/horizontal and radial/tangential, and the calculation of simplified loading assumption are included.
Mechanized tunneling is an effective and widely used construction method. It provides continuous support to the ground being excavated, which leads to robustly controllable ground deformations. During design and construction, analysis methods provide valuable tools for settlement prognoses. In this context, a reliable numerical simulation of the mechanized tunneling process requires realistic description of the construction procedure, accompanied with the representation of different components and their mutual interactions. In this chapter, the development of numerical models for the holistic tunneling process is discussed. Then, the simulation model \texttt{ekate}, used within this thesis, is briefly introduced.

A number of computational models have been proposed to address the simulations of various tunneling processes. These models are generally characterized with simplified repetitive sequences of soil excavation and supporting the excavated ground. With respect to mechanized tunneling, the machine advance, support measures at cutting face, tail void grouting, ring installation and hydraulic thrust are relevant physical processes that occur during the construction. In the existing computational models, different schemes are proposed to realize the shield-soil interaction. In (MÖLLER and VERMEER 2008, MROUEH and SHAHROUR 2008), the shield machine is not discretized, instead, explicit contact forces are applied to account for such interaction. In (DO ET AL. 2014a), “fictive” shield simplification is adopted, in which, the deformation of the excavated boundary is checked to prevent the penetration into the predefined position of the shield skin. Some models simulate the shield by rigid volume elements (e.g. KOMIYA ET AL. 1999, SCHMITT ET AL. 2005), yet, the shield is set to be in direct contact with the soil without allowing any separation between the shield and the excavated ground. In the model by LAVASAN ET AL. (2017), interface elements are used to describe the interactions in between. Even in models where the shield is discretized as a deformable body that is explicitly in contact with the ground (KAVVADAS ET AL. 2017), shield
advancement is achieved through prescribed displacements along its path. Thus, the evaluation of a precise steering gap is interrupted with such a controlled movement.

With regards to the annular gap grouting, some models only discretize the hardened grout (e.g. DO ET AL. 2014a, KOMIYA ET AL. 1999, MÖLLER AND VERMEER 2008). In these models, the freshly pressurized grout is modeled via radial pressure. This pressure is then removed at a certain distance from the tail and corresponding elements are activated to model the hydrated material. To account for the hydration process of the grouting mortar, time dependent material properties are adopted during the simulation (e.g. KASPER AND MESCHKE 2004a, KAVVADAS ET AL. 2017, SCHMITT ET AL. 2005). In these models, the segmental lining is represented by shell elements (e.g. DO ET AL. 2014a, KAVVADAS ET AL. 2017, LAVASAN ET AL. 2017) or by volume elements (e.g. KASPER 2005, MÖLLER 2006, NÄGEL 2009) and the jack pressure forces can also be taken into account.

Understanding the difficulties inherent in process related to mechanized tunneling is a key point for establishing a computational model that is able to predict and evaluate the impact of the construction process on the surrounding ground and infrastructure. Therefore, before discussing the computational models for mechanized tunneling, section 2.1 and section 2.2 present an overview of the construction process of shield tunneling in soft soils and the main features related to the shield support. In section 2.3, state-of-the art computational simulation model for mechanized tunneling process is presented with more emphasis on the respective assumptions and modeling techniques. Section 2.4 describes the computational model employed for the presented study, its main features and the software environment.

### 2.1 Construction Procedure

Figure 2.1 shows a slurry shield machine with its main load carrying component in black and mechanical equipment in gray. In what follows, the basic functions related to shield machines with full face excavation in soft ground will be briefly discussed. The cutter head is positioned at the shield face. The rotational movement of the cutter head with the thrust forces of the hydraulic jacks excavates the soil. Right behind the cutter head, the excavation chamber is located. The pressurization in the excavation chamber is achieved according to shield design. The support measure applied by the shield at the excavation face compensates the pressure loss resulting from the excavation. This can be achieved by either mechanical, compressed air, slurry or earth pressure balance support. During excavation and shield advancement, The excavated soil is continuously transported via a special transport system (e.g. pumped pipes, screw conveyor or conveyor belts). The selection of an appropriate system is dependent on the type of ground and the method of face support and excavation. During construction, the shield skin provides an instantaneous support measure to the excavated soil until the installation of temporary support or the tunnel lining. Inside the shield, the erector assembles the lining segments to form a complete ring in its final position. After ring building, the shield moves forward and a gap is created between the lining and the soil. This gap, referred to as the annular gap, is simultaneously filled with pressurized grouting mortar to prevent loosening of the soil and provide bedding for the installed lining. The area between the shield and the lining is sealed.
with wire brushes to prevent the flow of grout into the shield. The newly installed ring supports the hydraulic jacks and counteracts the thrust forces during the advancement of the shield.

**Figure 2.1:** Longitudinal section of a hydro-shield machine showing its main structural components in black and shield equipment, cutting wheel and erector in light gray

### 2.2 Ground Support in Shield Tunneling

The shield machine provides support for the excavated ground during construction until the installation of the lining and the hardening of the grouting mortar. This controls ground deformations and minimize surface settlements. Figure 2.2 shows the different supporting mechanisms; face support at the tunnel face, radial support along the shield and grouting pressure at the shield tail.

There are different means of face support measures that can be counted in the shield design (e.g. mechanical, compressed air, slurry and earth support). The choice of a proper system depends on the geological conditions and the required support level (GERMAN TUNNELLING COMMITTEE (DAUB) 2016). Compressed air support can be adopted for temporary support during inspection and maintenance work. It can be used as the main support system, yet, it is not an effective support measure for coarse-grained soils especially for soil with permeability larger than $10^{-4} m/s$ (MAIDL ET AL. 2013). The realization of a compressed air support in a numerical simulation has been presented by NAGEL AND MESCHKE (2010).
The use of pressurized slurry (a bentonite suspension) to support the earth and water pressure at the tunnel face is applied in slurry shields. The pressure transfer mechanisms are originally inherited from diaphragm wall technology (MÜLLER-KIRCHENBAUER 1972). In hydro-shields, the excavation chamber is filled with the pressurized slurry and the pressure is controlled by an air bubble behind the submerged wall with a regulated in/outflow. The proper realization of pressure transfer mechanism ensures the safety at the tunnel face (BEZUIJEN ET AL., 2001; BROERE AND VAN TOL 2000). In this context, two scenarios can occur with slurry support. One case is that a filter cake, that transfers the support pressure, is formed. This occurs if the bentonite suspension penetrates the excavation face forming an impermeable membrane directly on the face. The type of soil and bentonite content controls the occurrence of this situation. Second case is when the filter cake is not formed, then the bentonite suspension further penetrates into the face and the slurry pressure is transmitted to the soil along the entire penetration depth. The latter can be evaluated analytically (DIN 4126), experimentally (ZIZKA ET AL. 2018) or numerically (SCHAUFLER 2015).

Earth Pressure Balance shield (EPB) uses the concept of earth support. The excavated material mixed with a conditioning agent produces the required support. Pressure control is achieved by the amount of injected slurry and the adjustment of the speed of the screw conveyor with respect to the shield movement. The determination of the level of face support with a sufficient safety factor against failure is crucial. Different cross sections should be checked along the path of the tunnel for different depths and water levels with respect to the chosen supporting type and the required safety factor. The analytical solutions to determine the safety factor require a dominant assumption for the shape of the failure zone. For example, the failure mechanism by HORN (1961) assumes a wedge shape with an overlying prism. This model has been further developed by ANAGNOSTOU AND KOVÁRI (1994). However, the use of numerical techniques provides an alternative solution in particular for cases where analytical solutions are not applicable (e.g. different soil layers). Numerical methods, that are used to evaluate the tunnel face stability, can be found in different literature (ALSALHY ET AL. 2017, KIRSCH 2009, VERMEER ET AL. 2003).

The shield skin supports, the sides of the excavated soil as shown in figure 2.2. From the shield geometry, a gap is created due to the overcut and shield conicity. Moreover, additional gapping occurs when driving along curved alignments (FESTA ET AL. 2015). The contact pressure between the soil and the shield skin controls the ground deformation along the shield. In addition, the existence of an open gap between the soil and the shield skin allows for the flow of the support medium.
around the shield. In case of relatively high overcut, it is possible that a pressure communication occurs between the bentonite at the excavation chamber with the grouting mortar at the annular gap. Bezuijen et al. (2012) presented an analytical formula to evaluate such an interaction that can be integrated into numerical models. The volume loss along the shield contributes to the overall volume loss and it can be controlled by the shield design and the optimal selection of the face pressure and the grouting pressure (Vu et al. 2016). The pressurized grouting mortar is the last support measure that the shield machine generates at its tail. It controls ground settlements and provides bedding for the tunnel lining. There exists different grouting methods and various types of grouting materials. New developments enhance the grouting procedure and expand the application range of mechanized tunneling (Thewes and Budach 2009). Further discussion on the face support, the grouting process and the respective numerical representation in the model is provided.

### 2.2.1 Heading Face Support

In mechanized tunneling with a closed face, a pressureized support medium is applied at the heading face, which provides a permanent support to ensure safety during construction. Such supporting measure maintains the stress state at the excavation face by controlling ground deformation and preventing the inflow of groundwater into the shield. Two main operational modes for shield machines can be distinguished as; a hydro-shield and an earth pressure balance shield. To support the tunnel face, bentonite suspension is used in hydro-shields where the pressure is controlled by a compressed air reservoir in the pressure chamber. On the other hand, slurry extracted from the excavated soil forms the supporting medium in EPB shields. The excavated ground, in particular cohesive soils, under sufficient pressure is used in similar fashion with differences in pressure control mechanism (Maidl et al. 2013). The choice of the shield type and the excavation mode depends on the geological conditions. As well, conditioning and disposal aspects related with environmental risks can influence such a decision (Zumsteg and Langmaack 2017). The typical application range for hydro-shields are for coarse soils with high permeabilities, whereas for finer soils EPB shields are used frequently (German Tunnelling Committee (DAUB) 2000). Figure 2.3 shows the optimum application range for hydro shields according to Krause (1987) and EPB shields according to the European Federation for Specialist Construction Chemicals and Concrete Systems (EFNARC) (2005). However, the continuous development of TBM technology improved the applicability and expanded their application range.

The supporting medium penetrates into the pore spaces under pressurization and two possible scenarios are likely to occur. An impermeable membrane, namely filter cake, can be formed at the tunnel face which occurs when the suspended solids extensively plug the pore space of soil at the face. This is likely expected in relatively low permeability soils and sufficient bentonite content (Maidl et al. 2013). The formation of a filter cake enables an efficient application of the face pressure. In coarse-grained soil, bentonite suspension could penetrate into the excavation face without forming a perfect filter cake. Experiments show a decreasing permeability coefficient even after reaching the final penetration depth (Zizka et al. 2018). With no filter cake, it is expected to cause an increase in the pore water pressure and supporting the face will be questionable (Anagnostou and Kovári 1994).
2.2.2 Annular Gap Grouting

A pressurized grouting mortar is used to fill the gap that exists behind the shield tail, between the excavated ground and the tunnel lining, simultaneously with the advancement of the shield. This steering gap inherently occurs due to the geometrical configuration of the shield (i.e. overcutting and conicity), see figure 2.4. The pressurized grout has an essential role during tunneling: it provides an instantaneous bedding for the segmented tunnel lining and forms protective isolation for concrete lining from the direct contact with soil (MAIDL ET AL. 2013). Moreover, pressurization preserves the stress state around the tunnel thus minimizes ground deformations (AGGELIS ET AL. 2008), and it provides sufficient normal forces in the segmented lining which guarantees sufficiently tight and stiff joints.

There exist different types of grouting materials and mainly two methods to fill the annular gap (THEWES AND BUDACH 2009); in practice, the most typically used materials are cement-based mortar, cement-free mortar and two-component grout. The basic components of grouting mortar are different types of granular material (e.g. gravel, sand, and fly ash) and bentonite-slurry in addition to water. The cement-based grouting mortars are either active or semi-active according
to the water/cement ratio. In addition, Pea gravel, combined with mortar, is used in hard rock tunneling without pressure to fill the annular gap which provides partial embedment. Grouting material is transported either through holes in lining segments or by supply lines through shield skin. Accordingly, grouting mortar should, at the early stage, maintain some characteristics regarding workability (YOUN 2016). With the advancement of the shield machine, the influence of grouting pressure at the shield tail decreases and the stresses in the grouting mortar is dominated by the forces transmitted between the surrounding soil and the lining (TALMON ET AL. 2006).

2.2.3 Infiltration Process in the Soil

In mechanized tunneling practice, the use of pressurized fluids to support the soil triggers an infiltration process, as sketched in figure 2.5. It occurs by the transition of process liquids fines into the pore space of the soil accompanied by the reduction of both pore space and permeability depending on the initial conditions of soil and the support fluid.

![Infiltration Process in the Soil](image)

**Figure 2.5:** Illustration of the infiltration process during mechanized tunneling and the formation of a filter cake (THIENERT 2011)

**Slurry infiltration at the heading face**

The pressurized bentonite suspension is successfully used as the supporting medium in slurry shields, in particular for tunneling in cohesion-less soils under ground water level. Such technology was originally developed from diaphragm walls where the slurry supports the excavated trench. Studies were presented in literature to experimentally and theoretically investigate the slurry-stabilized trenches (MORGENSTERN AND AMIR-TAHMASSEB 1965, MÜLLER-KIRCHENBAUER 1972, WEISS 1967). MÜLLER-KIRCHENBAUER (1972) noted that the assumption of a membrane action is not always valid. A slurry penetration into the pore spaces may occur, the mechanism is governed by the grain size distribution and the pressure gradient of the suspension. HAUGWITZ AND PULSFORT (2009) presented the two limit cases of pressure transfer mechanism as shown in figure 2.6-a and b. In addition, an intermediate state can develop as shown in figure 2.6-c, where the filter cake is formed partially with reduced penetration. The various mechanisms with respect to the filter cake formation are shown in figure 2.7 by the stagnation of bentonite slurry in the test specimens.
In mechanized tunneling, an analogous concept is adopted for slurry support at the tunnel face (Babendererde 1991, Jancszcz and Steiner 1994). The application of an adequate face pressure and transferring this pressure to the soil is an important aspect for a safe and successful construction. Some field measurements show an increase in pore water pressure levels in front of the tunnel face (Bezuigen et al. 2001, 2016, Minec et al. 2004). These were monitored during the excavation phase, as the excess pore water pressure drops during the standstill (ring construction). It should be noted that the developed pore water pressure in front of the shield machine influences the tunnel face stability (Broere 2003).

According to Talmon et al. (2013), soil plastering occurs in two steps; a mud spurt phase takes place, followed by the filter cake formation. The penetration depth can be evaluated analytically or through laboratory tests. According to DIN 4126, penetration depth is determined as:

$$L_{\text{penetration}} = \frac{\Delta P \cdot D_{10}}{2 \cdot \tau_p}$$

where $\Delta P$ is the pressure difference between the slurry and groundwater, $D_{10}$ is the effective diameter of the soil and $\tau_p$ is the liquid limit of the suspension. However, laboratory tests give more reliable and comprehensive results. The laboratory tests for bentonite slurry infiltration in saturated sand, by Xu et al. (2017), show that excess pore water pressure is developed during mud spurt.
2.2. GROUND SUPPORT IN SHIELD TUNNELING

phase. Then, filter cake will be formed and flow slows down. This process is dependent on bentonite content and occurs in a time scale of seconds as shown in test results in figure 2.8. Similarly, it is indicated in (Maidl et al. 2013) that this process depends on soil permeability and bentonite content, and it takes about 1 to 2 seconds to occur.

![Diagram of slurry penetration](image_url)

**Figure 2.8:** Laboratory test for slurry penetration (Xu et al. 2017): (a) sketch of test apparatus and (b) test results of water discharge and excess pressure for a bentonite content of 40 g/l

For partially and fully saturated ground medium, the use of multi-phase finite element formulations in numerical modeling enables to model the fluid flow through the pore and the transient description of consolidation process. As well, the distinction between drained and undrained situation is not required; since the permeability coefficient describes the hydraulic properties of the ground. Even so, the drop of hydraulic conductivity due to penetration is generally not accounted for in many numerical models. This controls the distribution of the excess water pressure and consequently the ground deformation. In this context, a finite element transient seepage model (no mechanical deformations are considered) is presented by Zizka et al. (2016) to provide a precise assessment of the local pressure gradient at the local points of the tunnel face, figure 2.9. It uses a time-dependent permeability coefficient according to excavation parameters of the cutting face. This model mainly focuses on the pressure transfer mechanism at the local points on the tunnel face and the cutting process (i.e. distribution of cutting tools, disc penetration and cutting rotation speed) on the excess pore water pressure.

**Grouting mortar infiltration**

In addition to slurry penetration at the face, infiltration of grouting mortar into the soil also takes place. As a result, the grouting mortar consolidates and an initial development of shear strength is obtained, that is not from the hardening process. This ensures the lining bedding and also counteracts floating due to buoyancy. The consolidation of the grouting mortar depends on the mix design of the grout, the pressure difference (between the grouting pressure and pore water pressure) and the hydraulic/mechanical properties of the soil (Talmon and Bezuijen 2009). As interpreted by Talmon and Bezuijen (2006), the decrease in the field measurements of annular gap grouting pressure is resulted from grout bleeding/consolidation. Laboratory tests, with test set up similar
to figure 2.8-a, are performed to investigate the time variant process and its effects on the grout properties and pressure. *Bezuijen and Talmon* 2003. *Han et al.* (2007) studied the effect of relative difference between injection pressure and water pressure on the consolidation process and it is concluded that the pressure gradient accelerates the process and reduces the stabilization time. Further discussion on the formation of filter cake between the soil and the grouting mortar is presented in (Youn 2016).

A meso-scale model is proposed by *Schaufler et al.* (2013) to simulate the physical process of grouting mortar infiltration. This small scale model uses the theory of porous media as a basis for the analysis, in which, a four phase domain describes the grain mixture and flow. It consists of rigid soil skeleton, the pore fluid, the fines attached to the soil and the transported fines, figure 2.10-a and b. The partial mass balance of each constituent depicts the physical problemnumerically in which a mass production term is introduced to account for a local phase change from transported fines to solid-like fines. This characterizes the complex mixture with clogging of fines through the pores and the evolution of hydraulic properties. This model is used to simulate the 1-dim process as shown in figure 2.10-c and its application to mechanized tunneling in figure 2.10-d. In these models, the dotted and solid lines represents the drained and undrained situations for the hydraulic boundary conditions. The numerical model in figure 2.10-d consists of only the grout and the soil, while the lining is represented by respective boundary conditions (i.e. a flux and a traction free surface). Using this model, different phenomena related with grouting have been investigated which can be summarized as:

- Infiltration process of fines into the pores of the soil
- Stiffening effects due to infiltration
- Plastic deformation of soil
- Consolidation of soil due to grout loading
2.3 Computational Modeling for Shield Tunneling Process

Different numerical models have been developed in literature to simulate the mechanized excavation process. They generally adopt the step-by-step excavation scheme and apply the boundary conditions that represent the physical domain. In what follows, different models have been briefly presented with focus on the recently developed models to summarize the state of the art of the existing models. This provides comprehensive overview on the development of computational models for shield tunneling process in literature. Table 2.1 summarizes these models, the respective software environment employed and their main features and modeling assumptions.


This model is one of the early simulations developed to investigate the mechanized tunneling process in a fully saturated porous medium using a coupled finite element formulation. Tunneling in consolidated clay ground was the focus of this model where coupled quadratic solid elements with Mohr-Coulomb material response were used for discretizing the soil. The model components, as shown in figure 2.11, include the shield, the lining and the grout. Thin shell elements are used to model both the shield and the lining while coupled volume elements, similar to soil elements, are used to model the grouting mortar. A linear elastic material is used to represent the steel shield and the concrete lining. The hardening process of the grouting mortar has been introduced with a time variant elastic modulus and Poisson’s ratio. Both face and tail pressures have been applied in the model and their effect on ground deformation and the excess pore water pressure was investigated. It was shown that the ground response is affected by the strength development and permeability change of the grouting material. The shield-soil interaction is not explicitly modeled with contact. Instead, the shield is introduced with equivalent diameter and equivalent elastic modulus. The staged analysis procedure simulates the shield advancement by the deactivation and activation of the corresponding elements that represent different components.

![Figure 2.10: Grout infiltration model (SCHAUFLER 2015): (a) soil and grout at the micro-scale, b) the components of the four phase model ($\phi_s$ solid phase, $\phi_f$ fluid phase, $\phi_{sm}$ fines behave solid-like and $\phi_{fl}$ fines behave fluid-like), (c) illustration of the 1-dim simulation model with boundary conditions and (d) numerical model for a cross section of a tunnel lining](image)
**Simulation Model by Mroueh and Shahrou (2003, 2008)**

This model adopts a simplified modeling approach to simulate the shield tunneling process using the FE software PECPLAS (Mroueh and Shahrou 2003). Neither the shield nor the grout are considered in the simulation. Instead, the *convergence-confinement* method is used to represent the stress relaxation along the shield length, while the lining is assumed to be in direct contact with the ground, see figure 2.12. The effect of pore pressure is not accounted in this model and the ground is modeled as a one phase element with an elastic, perfectly-plastic constitutive law with Mohr-Coulomb yield surface and non-associated flow rule. The excavation procedure is achieved by the step-by-step simulation where the internal forces of the excavated ground elements along the position of the shield are partially decreased by a factor ($\alpha < 1$) that reflects the deconfinement along the shield. At the shield tail, the reduction factor ($\alpha$) is set to zero and the lining is activated. As well, the face support pressure is applied at the tunnel face and assumed to be constant with depth.

**Simulation Model by Schmitt et al. (2005, 2008)**

The simulation model by Schmitt et al. (2005) was developed using the commercial software FLAC3D and it is used to study the probabilistic response of tunneling with an EPB shield. Fig-
Figure 2.13: Components of the simulation model by SCHMITT ET AL. (2005)

Simulation Model by MÖLLER (2006)

MÖLLER (2006) presented a 3D finite element model for the analysis of both open and closed face shield tunneling using the commercial software PLAXIS. A special focus was devoted to the prediction of forces in tunnel lining. The surrounding soil is assumed to be unsaturated, whereas two constitutive models are used to describe the stress-strain behavior, namely the HARDENING SOIL MODEL and the elastic perfectly-plastic MOHR-COULOMB model. The so-called grout pressure method (see also MAIDL 2008) is employed to simulate the interactions between the tunneling process and the surrounding soil; supporting pressure is applied at the heading face to model the face support, and a radial pressure is applied on the excavation boundary to represent the contact pressure between the shield and the soil, see figure 2.14. These pressures are increasing hydrostatically with depth. The lining tube elements are sequentially activated at a distance behind the shield tail, since the tail gap is initially modeled as a free surface, within which a radial pressure is applied to substitute the grouting pressure and the soil in this zone is free to deform. At a certain distance from the shield tail, the grouting pressures are removed and new grouting mortar elements are activated to fill the gap (MÖLLER AND VERMEER 2008), where the mortar is assumed to be hydrated.
Simulation Model by Kasper (2005), Nagel (2009)

Kasper (2005) developed a model for the process oriented simulation of shield tunneling in soft saturated soil using the commercial software MARC. The soil is assumed to be in a fully saturated condition and the modified CAM-CLAY model is used to describe its material behavior. Many relevant components in shield tunneling have been explicitly modeled (i.e. shield machine, hydraulic jacks and grouting mortar), shown in figure 2.15. The shield machine is modelled with volume elements representing the main load bearing components (i.e. the shield skin and the shield wall) while its own weight (weight of the equipment) is applied as distributed gravitational loading. The face pressure is applied directly on both the excavation face and the shield face. While, the interaction between the shield skin and the soil is accounted through the frictional contact mechanics. After the shield movement, grouting elements behind the shield tail and the lining elements inside the shield are activated. The grouting mortar is modeled with a two phase element formulation and a time dependent elastic material law which enables the representation of its hydration process and the pressurization of the pumped grout. The face pressure is applied by the adequate boundary condition where no flow condition is assumed for the membrane situation and a prescribed water pressure is applied for the case of slurry penetration. The grout pressure is applied by prescribing the water pressure and the total stresses at the tail of the shield to ensure a state of zero effective stresses. Systematic parametric studies for the different interactions in mechanized tunneling have been performed using this model.

This model was further developed with a more advanced software architecture Nagel (2009), Stascheit (2010) using the finite element code Kratos. The newly developed model is denoted as ekate (Enhanced Kratos for Advanced Tunneling Engineering). Nagel (2009) provided numerical model for partially saturated soils in tunneling with more advanced simulations that include the compressed air support and the flow of process fluids around the shield. In these simulations, the shield machine is modeled as a deformable body and the hydraulic jacks are modeled as individual components, represented by means of Crisfield truss elements. The latter are supported on both the shield and the lining which transmit thrust forces to the lining in a straightforward manner. Equal elongations are applied to the hydraulic jacks to simulate the advancement process, and therefore, simulations were limited to straight tunnels. Moreover, Alsahly et al. (2016) developed a steering algorithm to steer the shield machine through arbitrary alignments. The algorithm
determines the elongation for each hydraulic jack based on the current position of the shield and the desired movement. In addition, adaptive remeshing strategies are developed to realistically capture the shield motion and the distribution of thrust forces \cite{alsahly2016}. ekate model is being used in this thesis, and therefore, further details are provided in the next subsection.

**Figure 2.15:** Components of the simulation model by Kasper \citeyear{ka}, Nagel \citeyear{na}. (2009)

**Simulation Model by Zhao et al. \citeyear{zhao2012}**

This 3D model simulates the mechanized tunneling process for deep tunnels in rock soils. Both hard rock and weak rock are modeled with special focus on the problems related with squeezing conditions of weak rocks. The commercial software MIDAS GTS is used to develop this model. The model includes the components that are related to the rock TBM tunneling (i.e. single/double shield, grippers, cutter head pressure and two methods for simulating annulus grouting), see figure 2.16. Unlike tunneling models in soft soils, where the surface settlements are the most relevant outputs, the model investigates the Shield/Rock-Mass interaction.

The rock mass is represented via a cylindrical domain without representing the free ground surface. The rock is discretized with eight-nodes hexahedral volume elements with mesh refinements at the expected damaged zone. The rock mass is assumed to be continuous, homogeneous and isotropic. The generalized HOEK–BROWN failure criterion is used to capture the brittle behavior of the rock, while, MOHR-COULOMB constitutive law with a non-associated flow rule describes the squeezing behavior. The applied in-situ stresses are uniform within the domain representing a deep tunnel condition.

The shield machine is modeled by four-nodes quadrilateral plate elements with a linear elastic material law. The shield is either a single or double with front and rear parts. The model includes the jack thrust explicitly as external forces on the newly installed lining for tunneling in weak rock. In addition, the TBM grippers, used with hard rocks, are modeled as radial pressure on steel plates positioned on the excavation boundary. The concrete lining is modeled as a continuous tube without joints using plate elements with a linear elastic material. Backfilling is represented by volume elements with a linear elastic material that symbolizes pea gravel for tunneling in hard rock or cement grout for tunneling in soft rock. The hardening of cement grout is not counted, instead, two values of the elastic modulus are used to represent the phase transition from soft to hardened...
grout. Interface elements are used to describe the Shield/Rock-Mass, Cutter-head/Rock-Mass and Lining/Rock-Mass Interactions. Construction stages are realized with the step-by-step simulation. Specifically, excavation and lining installation are performed within the simulation at the same time step and the shield advancement is achieved by activating the corresponding elements at the current position of the shield.

**Figure 2.16:** Components of the simulation model by ZHAO ET AL. (2012)

**Simulation Model by DO ET AL. (2014a)**

A 3D model is utilized to simulate the shield advancement during mechanized tunneling using the commercial software FLAC3D. In this model, the ground is modeled as a two phase medium in order to model the ground water flow and the consolidation process. In addition, MOHR-COULOMB or CYSOIL models, available in FLAC3D constitutive laws library, are used to describe the non-linear ground response. The shield is not explicitly considered in the simulation model. Instead, a simplification with a "fictive" shield (indicated by dotted line in figure 2.17) is introduced. In this approach, the position of each point, on the excavation boundary, is checked at each computation step with respect to the predefined shield position. The point is free to deform until it penetrates into the shield. Penetration is prevented by fixing the point displacements at the shield surface (i.e. point deformation is bounded at the shield skin). In addition to the pressure applied at the tunnel face, it is assumed that the slurry migrates to the gap between the shield and the soil. Therefore, a radial pressure is assumed to act on the excavation boundary behind the tunnel face as shown in figure 2.17. The effect of hydraulic jacks thrust is considered by applying concentrated forces on the nodes at the edge of the newly installed ring. The distribution of forces is assumed to increase linearly with depth.

The simulation of annular gap grouting is achieved in two steps in order to model the phase change from the liquid state to the solid state. In the liquid state, the grout is realized by a radial uniform pressure acting on both the excavated ground and the lining. The grouting mortar is also assumed to migrate along the shield for a certain distance, which is modeled by a triangular pressure
distribution. The hydration process of the grouting mortar is overlooked and the solid grout is modeled by volume elements with linear elastic properties that correspond to the properties of the fully hardened grout. Tunnel lining is modeled by linear elastic embedded plate elements. The latter are connected to the ground through grouting volume elements. The lining joints are described by the double node connection that includes three translations and three rotations. The commonly used step wise simulation is employed to replicate the construction process. The latter mainly consists of shield advancement and ground excavation followed by lining installation.

**Figure 2.17:** Components of the simulation model by DO ET AL. (2014a)

**Simulation Model by** LAVASAN ET AL. (2017), ZHAO ET AL. (2015)

The commercial FE-code PLAXIS is utilized to develop this model (model components are shown in figure 2.18). The model has been used to simulate the mechanized tunneling process for deep and shallow tunnels. The ground is discretized by 10-noded tetrahedra elements with different constitutive models for granular material representation. A linear elastic model with/without a stress dependent stiffness is utilized for comparisons with analytical methods (ZHAO ET AL. 2017). The isotropic hardening plastic behavior is captured by the HARDENING SOIL MODEL. The 6-noded triangular shell elements at the excavation boundary express both the shield skin and the concrete lining. These elements are assigned a linear elastic constitutive model representing the steel and concrete stiffness.

The lining joints are not idealized in this model, yet, (LAVASAN ET AL. 2017) suggested a reduction in the mechanical properties to represent the reduced bending stiffness. The interaction between the shield skin and the excavated soil is introduced by interface elements. As well as, the lining-soil interaction is expressed by a complete nodal tying or by introducing a contact interface that allows for relative deformations. The interface properties have considerable effects on the lining forces and deformation, however, adequate properties have to be calibrated. For example, (LAVASAN ET AL. 2017) proposed a reduction factor of 60% for the contact interface properties between both shield/soil and lining/soil. This assumption is introduced to resemble the steering gap
for shield/soil interaction and the low grout stiffness compared with concrete stiffness for the lining/soil interaction. The grouting mortar is not included in this model, instead, its effect by means of grouting pressure at the shield tail is considered. This pressure is modeled by two possible approaches. The grouting pressure is accounted for either by a distributed load at the tail of the shield without activating the lining elements at this section or by assigning the water pressure at this location as shown in figure 2.18. The staged simulation steps model the construction process where the progressive excavation is expressed by the deactivation of the corresponding elements and applying the face pressure as a linearly varying distributed load at the excavation face. While, the shield advancement and lining installation are realized by assigning the corresponding material to its corresponding elements.

The realization of the infiltration process of the grouting mortar into the excavated ground has been introduced in (LAVASAN ET AL. 2017) in which the change in hydraulic properties of the infiltrated ground is evaluated through a four phase sub-model and then integrated into the 3D FE-model. Using the time dependent permeability evolution, it was shown that the infiltration of grouting mortar has a significant effect on the ground deformations for soils with high permeability.

**Simulation Model by KAVVADAS ET AL. (2017)**

The developed model is dedicated for EPB shield tunneling simulation using the commercial software ABAQUS. Figure 2.19 provides a schematic description of the model. The 3D finite element model is discretized with kinematic linear elements. The 8-noded prism elements with a linear elastic-perfectly plastic MOHR-COULOMB material response are used to describe the ground medium around the tunnel. The shield is represented by 4-noded quadrilateral shell elements and 8-noded volume elements. The shell elements represent the shield skin, cutter head and bulk head, while volume elements do not contribute to the shield stiffness, instead, they are used for weight distribution of other mechanical components (i.e. excavation chamber and machine equipment). The
The conical shape of the shield has been taken into account as well as the contact interaction between the shield and the soil. No frictional interaction is assumed supposing that the steering gap is lubricated. A controlled displacement, that defines the current position of the shield, is utilized to represent the advancement process.

The annular gap grouting is explicitly modeled via 8-noded volume elements. To model grout pressurization, a prescribed internal pressure is applied to the newly activated elements. The hardening process of the grout is realized with a time variant elastic modulus. The segmental lining is represented with linear elastic 4-noded shell elements. Lining is modeled either by continuous elements or by segments with joints. Node-to-node contact is used to simulate the segments’ interactions in both longitudinal and ring joints for the geometrically compatible lining mesh. The nodal contact is originally defined by six DOFs (i.e. 3 displacements and 3 rotations), that are reduced to a rotation stiffness for the longitudinal joints coupling and a translational shear spring for the ring joints. The successive excavation process is simulated by the repetition of the simulation steps. The removal of the soil at the tunnel face with the prescription of the new position of the shield models the excavation. This is followed by the activation of the newly installed ring inside the shield and the activation of the grouting elements with the fresh grout properties and internal prescribed pressure.

Figure 2.19: Components of the simulation model by KAVVADAS ET AL. (2017)
<table>
<thead>
<tr>
<th>Computational model</th>
<th>Software</th>
<th>TBM-soil interaction</th>
<th>TBM advance</th>
<th>flow around TBM</th>
<th>Jack thrust</th>
<th>Grout hardening</th>
<th>Lining joints</th>
</tr>
</thead>
<tbody>
<tr>
<td>MANSOUR (1996)</td>
<td>FINAL</td>
<td>approximated by equivalent shield</td>
<td>approximated by step-wise activation</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>SWOBODA AND ABU-KRISHA (1999)</td>
<td>in-house code</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MROUEH AND SHAHROUR (2003, 2008)</td>
<td>PECPLAS</td>
<td>approximated by a percentage of nodal forces</td>
<td>approximated by step-wise activation</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>SCHMITT ET AL. (2005, 2008)</td>
<td>FLAC3D</td>
<td>direct connectivity</td>
<td>approximated by step-wise activation</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>MÖLLER (2006)</td>
<td>PLAXIS</td>
<td>approximated by a radial pressure</td>
<td>approximated by step-wise activation</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>MÖLLER AND VERMEER (2008)</td>
<td>commercial</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>KASPER (2005)</td>
<td>MARC</td>
<td>jacks</td>
<td></td>
<td>x</td>
<td>✓</td>
<td>✓</td>
<td>x</td>
</tr>
<tr>
<td>NAGEL (2009)</td>
<td>[commercial]</td>
<td>frictional contact</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ALSAHLY (2017)</td>
<td>KRATOS</td>
<td>+ steering</td>
<td></td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>x*</td>
</tr>
<tr>
<td>ZHAO ET AL. (2012)</td>
<td>[commercial]</td>
<td>prescribed displacements</td>
<td></td>
<td>x</td>
<td>✓</td>
<td>✓</td>
<td>x</td>
</tr>
<tr>
<td>DO ET AL. (2014a)</td>
<td>FLAC3D</td>
<td>approximated by fictive shield</td>
<td></td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>ZHAO ET AL. (2015)</td>
<td>PLAXIS</td>
<td>interface elements</td>
<td>approximated by step-wise activation</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>LAVASAN ET AL. (2017)</td>
<td>commercial</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>KAVVADAS ET AL. (2017)</td>
<td>ABAQUS</td>
<td>frictionless contact</td>
<td>prescribed displacements</td>
<td>x</td>
<td>x</td>
<td>✓</td>
<td>✓</td>
</tr>
</tbody>
</table>

*To be developed in this thesis

Table 2.1: Development of computational models for mechanized tunneling simulation
2.4 ekate: Enhanced KRATOS for Advanced Tunneling Engineering

The simulation model, denoted as ekate (Enhanced KRATOS for Advanced Tunneling Engineering), has been implemented via the object-oriented finite element code KRATOS (DADVAND ET AL. 2010). The latter is an open-source framework dedicated to perform numerical simulations for multi-physics problems. Its modular structure provides efficient and robust implementations of various algorithms and schemes (e.g. solution methods, time integration schemes, element formulations, constitutive laws, etc). KRATOS is written in C++, in which its kernel provides the basic functionalities and data managements. While, applications characterize the implementation aspects of the numerical model for different physical problems. Herein, the simulation model is developed using Kratos Structural Application and Ekate Auxiliary Application. More detailed discussion about the model can be found in (NAGEL 2009), while basic strategies and implementation aspects are presented in (STASCHEIT 2010).

The main goal of the model is to provide an efficient yet realistic simulation environment for all interaction processes occurring during machine driven tunnel construction. Therefore, the model includes all relevant components of the mechanized tunneling process as sub-models, representing the partially or fully saturated ground, the tunnel boring machine, the tunnel lining, hydraulic thrust jacks and the tail void grouting, which are interacting with each other via various algorithms. The interaction between the shield and the excavated ground is depicted via frictional contact algorithm. The shield-lining interaction is described with truss elements (hydraulic jacks) connected between the front surface of the last activated lining segment and the shield, by which, the lining acts as a counter-bearing for the hydraulic jacks thrust to push forward the shield machine. Figure 2.20 shows the basic model components on the left and their respective representation in the finite element mesh on the right. In what follows, the basic model components, the steering algorithm for shield advancement and the simulation script for modeling the construction process are discussed in more detail.

Figure 2.20: Computational model for mechanized tunneling ekate. left: main components involved in the simulation of the mechanized tunneling process and, right: finite element discretization of the model components: (1) Geological and ground Model, (2) Shield Machine, (3) Tunnel Lining, (4) Tail void grouting and (5) Thrust Jacks
2.4.1 Model Components

Ground Model

The ground model is formulated within the framework of the Theory of Porous Media (TPM) (DE BOER 2000) that accounts for the coupling between the deformations of the solid phase and the fluid pressures (i.e. an incompressible water phase and a compressible air phase). In that, deformations and pressures are taken as primary variables. The governing balance equations build a set of partial differential equations as the basis of finite element solution. Herein, the two-phase model for fully saturated soils (figure 2.21) is briefly presented.

![Figure 2.21: Fully saturated soil modeled according to TPM](image)

The following balance equations prescribe the momentum balance of the mixture, and the mass balance of both solid and fluid phase. Under the assumption of incompressible solid and water phase, the mass balance of each constituent $\alpha$ [$\alpha = \text{solid}$ and $\text{water}$] is given by:

$$\frac{D\rho^\alpha}{Dt} + \rho^\alpha \text{div}\dot{x}^\alpha = 0,$$

where $\rho^\alpha$ is the average density of a constituent $\alpha$. The porosity $n$, which defines the volume fraction of water, is used to describe the solid and water phases. Therefore, the average density of the mixture ($\rho$) can be determined by the intrinsic density of each constituent ($\rho^\alpha$) as:

$$\rho = \sum \rho^\alpha = (1 - n)\rho^s + n\rho^w$$

In addition, the velocity of the solid skeleton ($\dot{x}^s = \dot{u}^s = D_s\text{div}\dot{u}^s / D_t$) and the diffusion velocity ($\nu^{ws} = \dot{x}^w - \dot{u}^s$) are used to describe the motion of the constituents. Thus, equation 2.2 yields to:

$$\frac{D_s}{Dt} ((1 - n)\dot{u}^s) + (1 - n)\rho^s \text{div}\dot{u}^s = 0 \quad \text{for solid skeleton}$$

and

$$\frac{D_w}{Dt} (n\dot{u}^w) + n\rho^w \text{div}(\nu^{ws} + \dot{u}^s) = 0 \quad \text{for pore water}$$
For equation 2.4, assuming incompressible solid grains (i.e. \( D_s \varrho_s / D_t = 0 \)), a differential equation for the porosity can be derived as:

\[
-\varrho_s \frac{D_s n}{D_t} + (1 - n) \varrho_s \text{div}\hat{\mathbf{u}}^s = 0
\]

\[
\frac{dn}{dt} = (1 - n)\text{div}\hat{\mathbf{u}}^s
\]  
(2.6)

For the mass balance of the water phase, the time derivative with respect to the current configuration of the water phase is transformed to the current configuration of the solid phase as follow:

\[
\frac{D_w}{D_t}(\rho^w) = \frac{d\rho^w}{dt} + \text{grad}(\rho^w) \cdot \mathbf{v}^{ws}
\]  
(2.7)

The water flow \( \tilde{\mathbf{v}}^{ws} \) through the pore spaces is described by Darcy’s law (Darcy 1856). Accordingly, the flow is governed by the pressure gradient and the volume of pore spaces and expressed as:

\[
\tilde{\mathbf{v}}^{ws} = -\frac{k^w}{\varrho^w g}(\text{grad} \ P^w - \varrho^w \mathbf{g}),
\]  
(2.8)

where \( k^w \) is the hydraulic conductivity that symbolizes the available pore spaces in soil. Using the volume fraction \( n \), the Darcy’s velocity is related to the diffusion velocity \( \mathbf{v}^{ws} \) as \( \tilde{\mathbf{v}}^{ws} = n \mathbf{v}^{ws} \).

Applying equation 2.7 to equation 2.5, the mass balance of the incompressible water phase can be written as:

\[
\frac{d}{dt}(n \varrho^w) + \text{grad}(n \varrho^w) \cdot \mathbf{v}^{ws} + n \varrho^w \text{div}(\mathbf{v}^{ws} + \hat{\mathbf{u}}^s) = 0
\]

\[
\frac{d}{dt}(n) + \text{div}(\tilde{\mathbf{v}}^{ws}) + n \text{div}(\hat{\mathbf{u}}^s) = 0
\]  
(2.9)

The second balance relation is introduced by the overall momentum balance of the mixture using the averaged Cauchy stress \( \mathbf{\sigma} \) as:

\[
\text{div}(\mathbf{\sigma}) + \rho \mathbf{g} = 0,
\]  
(2.10)

where the effective stresses \( \mathbf{\sigma}^{st} \) and water pressure \( P^w \) are the stress variables. According to Terzaghi (1943), the effective stresses define the inner grain interaction (i.e. the stress-strain behavior of the soil skeleton). The effective stresses in a fully saturated soil are determined as:

\[
\mathbf{\sigma}^{st} = \mathbf{\sigma} + P^w \mathbf{I} \quad ; \mathbf{I} \text{ denotes the unity tensor}
\]  
(2.11)

The mass balance and the momentum balance equations form the set of partial differential equations to be solved in which the deformations and water pressures are the primary field variables. Further discussion regarding the multi-phase model for partially saturated soils and its numerical implementation has been presented in (Nagel 2009).
The material behavior of the soil skeleton is represented by means of nonlinear elasto-plastic constitutive laws; namely Drucker-Prager (DP) law (Drucker and Prager 1952) or Clay and Sand Model (CASM) (Yu 1998). DP-law presents a relatively simple model based on the approximation of Mohr-Coulomb criteria using a smooth yield function, see figure 2.22 (left). A generalized behavior for both clay and sand soils can be reproduced by CAS-model. Figure 2.22 (right) shows the yield surface in the principal stress space. The latter is similar to the CAM-CLAY models, yet, it overcomes the limitation of CAM-CLAY models for the characterization of sands and highly over-consolidated clays.

![Diagram](image)

**Figure 2.22**: Yield function in principal stress space and in the $p' - q$ plane: Drucker-Prager-model (left) and Clay and Sand-model (right)

**Shield machine**

The shield is modeled as an independent, stiff body that interacts with the excavated soil along its outer surface by means of frictional contact conditions. The shield geometry is depicted according to its design, see figure 2.23. The respective FE model as shown in figure 2.24 accounts for the main structural and load carrying components (i.e. the shield skin, the shield wall and other stiffening parts). Shield weight including the machinery parts are accounted for. The load is distributed on shield front, along approximately two third of the total length, considering the fact that most of the heavy parts are located at the front. The cutting wheel is not modeled explicitly, instead, the equivalent cutting forces in addition to the face pressure are applied on the shield wall. In addition, overcutting and shield skin tapering are explicitly considered, see figure 2.24. This is beneficial for a reliable prognoses of the ground settlements, as well, the adequate prediction of the shield soil interaction is feasible, in particular for curved alignments.

The hydraulic jacks are represented by Crisfield truss elements (Crisfield 1991), that produce the mutual interaction between the shield and the lining and by which shield advancement is achieved. In this context, a steering algorithm is developed to fully automatize the shield movement (Alshahly et al. 2016). The steering algorithm controls the elongation of each hydraulic jack. Prescribed strains and the counter-bearing produced by tunnel lining provides the momentum to move the shield forward. In addition, steering algorithm includes a correction vector that allows
for counter-steering against weight-induced dropping of the shield and keeps the path of the shield on the prescribed tunnel alignment.

![Illustration of the main aspects related to the numerical representation of the shield machine: main structural components represented by the thick black lines (left) and radial distribution of hydraulic jacks (right).](figure2.23)

**Figure 2.23:** Illustration of the main aspects related to the numerical representation of the shield machine: main structural components represented by the thick black lines (left) and radial distribution of hydraulic jacks (right)

![Finite element mesh of the shield machine, the hydraulic jacks and the lining, and the geometrical parameters involved in the definition of the shield model.](figure2.24)

**Figure 2.24:** Finite element mesh of the shield machine, the hydraulic jacks and the lining, and the geometrical parameters involved in the definition of the shield model

The frictional contact characterizes the interaction between the shield skin and the excavated ground. Following the basic concepts in (Laursen 2002, Simo and Laursen 1992), Kuhn-Tucker condition is applied, which defines the separation or direct contact between surfaces. As a result, the simulation model can predict the contact condition between the shield skin and the ground (i.e. whether a gap exists or not). More detailed discussion about contact and its algorithmic
implementation are provided in section 5.2. Herein, Slave and master contact faces are assigned to shield skin and excavation boundaries respectively, where Augmented Lagrange method enforces the contact constraint.

In the simulation model, the governing equations are the weak form of the mass balance equation for the ground water flow and the weak form of the equilibrium equation. Since large movements are required for the positioning of the shield, total Lagrangian FE formulation is used for shield discretization. It should be highlighted that the inertial forces are neglected since the machine advances through the soil with low speed. The final position and orientation of the shield results from the force balance on the shield, where, the advancement process is achieved by the elongations of the hydraulic jacks, see section 2.4.3.

**Tunnel lining**

The tunnel lining is represented by volume elements with linear elastic constitutive law. During the simulation process, lining rings are activated in a step-wise scheme to model its construction process. In Ekate, lining rings provide support to the excavated ground and counter bearing for the hydraulic jacks. In addition, lining is typically modeled as continuous rings. However, depending on the desired level of detail, the segmentation of the tunnel lining can be explicitly modeled by considering both ring joints and longitudinal joints. For this purpose, several numerical techniques can be employed in order to represent the interaction along the joints’ boundaries, e.g. interface element, springs or contact algorithms. In this contribution, a segmental tunnel lining model is developed and integrated in the large scale simulation Ekate. A surface-to-surface contact algorithm is employed in order to account for segments’ interactions in one ring as well as coupling between the consecutive rings. More comprehensive descriptions and analyses of the segmental lining models are presented in chapters 3 and 6. Continuous lining provides a satisfactory response in particular if ground deformation or global lining response are the main focuses of the analysis. A detailed lining model can realistically describe the kinematics of the tunnel lining especially if large rotations are expected.

**Grouting mortar**

The annular gap between the tunnel lining and the excavated ground is filled simultaneously with a pressurized grouting mortar, figure 2.25-a. The latter is a mixture that consists of a hyper-concentrated two phase material (Bezuijen et al., 2004). It should maintain, at the early stage, a certain degree of workability to be distributed uniformly around the lining. On the other hand, hardening should occur to resist the buoyancy of the lining and to prevent the dislocation of the joints. The setting of grouting mortar is characterized by an increase of mechanical stiffness accompanied with a phase change from semi-liquid to solid state, figure 2.25-b.

To model the pressurized grouting mortar, a two-phase (hydro-mechanical) formulation is used, which is similar to the element formulation of the ground model. The grouting pressure is applied as pore water pressure to the fresh mortar. Stiffening of the grouting mortar is considered by a time dependent hyper-elastic material and time dependent permeability to account for the hydration process. Simultaneous grouting of the annular gap is simulated by the step-wise activation of the
corresponding grouting mortar elements with respect to current shield position, while pressurization is realized by a prescribed pressure boundary condition on the face of the elements at the shield tail.

Herein, an exponential relation is used to define the temporal evolution of permeability. This assumption has been already proposed in (KASPER 2005, KASPER AND MESCHKE 2006a,b). The permeability of the grouting element is updated at the beginning of each time step, where the mathematical expression is given by:

\[
k(t) = (k^{(0)} - k^{(28)})e^{-\beta_{grout} t} + k^{(28)},
\]

where \(k^{(0)}\) and \(k^{(28)}\) are the initial permeability and final permeability after 28 days, \(t\) expresses the age in hours and \(\beta_{grout}\) is a parameter that controls the change with respect to time. Figure 2.26-a shows the time dependent permeability for two different analysis parameters \((\beta_{grout} = 0.05\) and \(0.10)\). With respect to the stiffness evolution of such cementitious material, the proposed material model follows the basic methodology of hyperelasticity for aging materials, as presented in (MESCHKE 1996, MESCHKE ET AL. 1996), see figure 2.26-b.

**Figure 2.25:** Annular gap grouting: (a) sketch of annular gap grouting through a nozzle in shield skin and (b) the process of grouting mortar hydration with stiffness and permeability evolution

**Figure 2.26:** Development of grouting mortar properties with time; (a) permeability evolution for two different analysis parameters and (b) description of the parametric function \(\beta_E(t)\) where the grout is fully hardened after 28 days
For the time dependent increase of elastic modulus, an irrecoverable strain necessarily occurs. Therefore, the strain tensor \( \varepsilon \) is decomposed into a recoverable elastic part \( \varepsilon^e \) and a non-recoverable part \( \varepsilon^t \) associated with the time dependent hydration.

\[
\varepsilon = \varepsilon^e + \varepsilon^t
\]  

(2.13)

According to the theory of hyperelasticity, a time-dependent function of the stored energy defines the stiffening effect and consequently the time dependent stress tensor as:

\[
W(\varepsilon, t) = \frac{1}{2}(\varepsilon - \varepsilon^t) : C^{28} : (\varepsilon - \varepsilon^t)
\]

(2.14)

\[
\sigma = \frac{\delta W}{\delta \varepsilon^e} = C^{28} : (\varepsilon - \varepsilon^t),
\]

(2.15)

where \( C^{28} \) is the standard elasticity tensor of the hardened material in which the superscript \( (28) \) indicates a reference time in days at the end of the aging process. The time dependent material tensor \( C(t) \) is expressed by the development of the Young’s Modulus \( E(t) \) as:

\[
C(t) = C^{28} \frac{E(t)}{E^{(28)}},
\]

(2.16)

and the experimental observations shows that the stress rate is related to the strain rate by the time dependent material tensor:

\[
\dot{\sigma} = C(t) : \dot{\varepsilon}
\]

(2.17)

The stress increment \( \Delta \sigma \) for a certain time interval \([t_n, t_{n+1}]\) is determined from the time integration of equation 2.17

\[
\Delta \sigma = \int_{t_n}^{t_{n+1}} C(t) : \dot{\varepsilon} dt = \frac{1}{E^{(28)}} C^{28} : \frac{\Delta \varepsilon}{\Delta t} \int_{t_n}^{t_{n+1}} E(t) dt = \frac{\chi}{E^{(28)} \Delta t} C^{28} : \Delta \varepsilon,
\]

(2.18)

in which, \( \chi \) expresses the integration of time dependent Young’s Modulus over the time interval \([t_n, t_{n+1}]\). Comparing equation 2.18 with the incremental form of equation 2.15, the incremental time dependent strain yields to:

\[
\Delta \varepsilon^t = \left(1 - \frac{\chi}{E^{(28)} \Delta t}\right) \Delta \varepsilon
\]

(2.19)

The elastic algorithmic tangent \( K^{el} \) can be obtained by the linearization of equation 2.15 after inserting equation 2.19. The algorithmic tangent is given by:

\[
K^{el} = \frac{\partial \sigma_{n+1}}{\partial \varepsilon_{n+1}} = \frac{\chi}{E^{(28)} \Delta t} C^{28}
\]

(2.20)
The time dependent stress-strain behavior of the proposed material is mainly related to the time variant Young’s modulus. The later is expressed as $E(t) = \beta_E(t)E(28)$, see figure 2.26-b. The coefficient $\beta_E(t)$ is defined, according to (MESCHKE ET AL. 1996), with the following expression:

$$
\beta_E(t) = \begin{cases} 
\beta_{I_E}^I &= c_E t + d_E t^2 & \text{for } t \leq t_E \\
\beta_{II_E}^I &= \left( a_E + \frac{b_E}{t - \Delta t_E} \right)^{-0.5} & \text{for } t_E < t \leq 672h \\
\beta_{III_E}^I &= 1.0 & \text{for } 672h < t_E 
\end{cases}
$$

(2.21)

Where $a_E$, $b_E$, $c_E$ and $d_E$ are material dependent parameters determined by the ratio $E_{(1)} / E_{(28)}$, the initial time interval $t_E$ and the time step $\Delta t_E$, see (MESCHKE ET AL. 1996) for more details.

**Surface-structures models**

During tunneling, there exists a mutual interaction between the surface structures and the ground. Such interactions significantly increase if deep foundations are located in the tunneling vicinity (NINIĆ ET AL. 2011, 2014). Generally, a detailed representation of the surface structures requires additional computational efforts, therefore, further simplifications are requested. A straightforward representation of existing buildings can be achieved by building substitute stiffness (SCHINDLER ET AL. 2014). The substitute stiffness accounts for building’s rigidity while numerical discretization is performed with simple geometries (i.e. 2D shells or 3D solids). The building mass and rigidity are determined from its specific structural components (e.g. foundation, roofing system and walls). In addition, OBEL ET AL. (2016) and SCHINDLER ET AL. (2014) included other characteristic features (i.e. external dimensions, year of construction and location) and provided characteristic curves for masonry buildings for rapid evaluation of their mass. Different approaches with various assumptions are presented in literature. In (OBEL ET AL. 2018b), two approaches are adopted for calculating the upper and lower limit of the substitute stiffness of the building. The lower limit is determined from the moment of inertia of the facade. NEUGEBAUER ET AL. (2015) suggested a reduced stiffness as an outcome of the existence of openings. The upper limit of the substitute stiffness includes, in addition to the facade, the stiffness of the roofing system (i.e. slabs and beams). Generally, the respective material of each structural element is accounted by the corresponding elasticity modulus. The lower and upper bound for the substitute stiffness is determined as:

**Lower Bound:**

$$EI_l = E \frac{bh^3}{12} \alpha_w ,$$

(2.22)

**Upper Bound:**

$$EI_u = E \left[ \frac{bh^3}{12} \alpha_w + z_s^2 A \right] + \sum_{i=1}^{n} E_i \left[ \frac{bh_i^3}{12} + z_{si} A_i \right] ,$$

(2.23)

in which $b$, $h$, and $A$ are the dimensions of the outer masonry walls: width, height and cross-sectional area respectively and $\alpha_w$ is a reduction factor that represents windows openings. The second term in equation 2.23 accounts for the floor stiffness where the subscript $i$ is the floor number. $z_s$ and $z_{si}$ are the distances measured from the neutral axis of the structure to the respective element.
In the simulation model ekate, buildings are drawn from a 3D city model and simulated with either shell or volume elements with a substitute stiffness or a detailed discretization of their main structural components, as shown in figure 2.27. However, the use of a detailed discretization of the surface structures, in particular for large models with several buildings, requires high computational costs. For this reason, the accuracy of building discretization is usually reduced and buildings are represented by their substitute stiffness in order to get the global response. Buildings that require particular interest can be further discretized with a higher level of detail, see section 7.2.

![Diagram of building discretization](image)

**Figure 2.27:** Integration of buildings in numerical simulations; (a) 3D city model, (b) simplified volume geometries, (c) lower level discretization by shell or volume elements with substitute stiffness and (d) detailed discretization of the main structural components

Numerically, the mutual interaction between the building foundation and ground domain can be realized by either interface elements, Lagrange-tying or the explicit contact formulation. With these methods, the rigidity of the structure is associated to the stiffness matrix of the soil, and with an adequate formulation, it is possible to use a nonconforming mesh for the soil and the footing. In this simulation model, Lagrange-tying is used to describe the relation between the buildings and the ground. A node to volume tying algorithm is used to impose a deformation constraint in which the building bases and surface settlements are tied regardless of the finite element discretization. The enforcement by a Lagrange multiplier is straightforward and requires less computational effort compared with the contact penalization. The drawback of Lagrange-tying scheme is that a complete bond is always maintained between the soil and the structure, while, contact formulation allows for a separation, that is implicitly included according to the orientation and stiffness of the building. Therefore, tying is set in the vertical direction and only activated when the contact stresses are in compression (NINić 2015).
2.4.2 Representation of Support Pressures

Numerically, the two scenarios, discussed in section 2.2.1, can be characterized by a "membrane model" and a "penetration model". They are described in the model by applying adequate boundary conditions, see figure 2.29. For the so called "membrane model", where a perfect filter cake is formed, the fluid flow is set to zero and a prescribed total pressure is applied at the tunnel face as follows:

\[ t = p_{\text{sup}} n \quad \text{and} \quad q^w = 0 \] (2.24)

For the "penetration model", i.e. without a filter cake, both fluid pressure and total stresses are prescribed at the tunnel face as:

\[ t = p_{\text{sup}} n \quad \text{and} \quad p^w = 0 \] (2.25)

The numerical description of the grouting pressure is achieved in a way similar to the description of face support. At the last face of the newly activated elements, the total stresses and water pressures are prescribed as a linear function using the average grout pressure at the tunnel axis \( \bar{p}_{\text{ax}}^{\text{grouting}} \) and its gradient \( \text{grad} \bar{p}^{\text{grouting}} \):

\[ \bar{p}^{\text{grouting}} = \bar{p}_{\text{ax}}^{\text{grouting}} + z \text{grad} \bar{p}^{\text{grouting}}, \] (2.26)

where \( z \) is the distance in the direction of gravity, measured from the tunnel axis. Consequently, the total pressure and the water pressure at the element face can be defined as following:

\[ p^w = \bar{p}^{\text{grouting}} \]
\[ t^* = -\bar{p}^w n, \] (2.27)

where \( n \) is the normal vector to the grouting face. With the previous description, the effective stresses at the last grouting face, where grout injection is executed, are set to zero.
2.4.3 Shield Steering

In tunneling projects, well-established surveying and positioning systems are a prerequisite to ensure that the tunnel is being constructed according to the designed alignment without exceeding the predefined allowable tolerance. During construction, measuring devices are used through set of reference points to continuously track the current position and direction of the machine with respect to the designed alignment, see figure 2.30. The deviations of the machine’s position are instantaneously displayed in graphical and numerical formats on the control screen; thus it helps the machine driver to steer the machine and to follow the designed path. The spatial movement of the machine is achieved by the hydraulic jacks’ extensions. Such a system, exerting large thrusts, is a key part of the shield. It does not only drive the shield but also controls the posture of the shield to ensure that the shield can advance along the expected path consequently for constructing the planned tunnel line (Huayong et al., 2009). In addition, during the advancement of the machine, the tail skin clearance between the segment and machine tail shall be checked regularly.

The shield provides protection for the construction works during excavation and segments erection. Thus, it should be designed to sustain the acting loads. The latter are considered as external or operational loads, see (Festa et al., 2012, German Tunnelling Committee (DAUB) 2005,
SRAMOON ET AL. 2002, SUGIMOTO AND SRAMOON 2002) for further details. The loads on the shield can be summarized as follows:

- Shield weight, including the weight of its equipment
- Radial loads due to earth pressure, water pressure and reaction forces from shield movement
- Tangential forces due to face pressure, thrust forces, cutting forces, frictional forces caused by shield movement

The shield machine should be driven as close as possible to the designed path with a specific tolerance. In the simulation model, a steering algorithm that controls the shield position has been developed. More illustration of the mathematical formulations used in the steering algorithm can be found in (ALSAHLY ET AL. 2016). The algorithm provides the instantaneous vertical and horizontal position as well as the driving direction which are used for the subsequent movements. In addition, the non-uniform distribution of the thrust forces can be obtained as a simulation result. Within the FE code, steering is implemented as a utility (SteeringUtility). Utilities in Kratos are auxiliary classes that are used for implementing supplementary features in a particular application. The main functions that are used in the simulation are the initialization of the utility, setting the hydraulic jacks and steering the shield. The workflow of the steering during the tunnel simulation is presented in Algorithm 1.

Algorithm 1 Implementation of the steering algorithm

1: Initialize SteeringUtility(...)  
2: Set up Jacks SetHydraulicJacks(...)  
3: for (each Step ∈ Excavation_Steps) do  
4:   Define Current_Positon  
5:   Define Target_Positon  
6:   Define Target_Direction  
7:   for (each Step ∈ Moving_Steps) do  
8:     SteerToStation(...)  
9:     Model.Solve(...)  
10:    Model.WriteOutput(...)  
11:  end for  
12:  Update shield position GetActualCenter()  
13:  Reset Jacks ResetHydraulicJacks()  
14: end for

Initialization of the SteeringUtility imports the utility and all its relevant methods. SetHydraulicJacks creates the truss elements that represent the hydraulic jacks; this involves a search algorithm for the jacks on the newly installed rings. The reference length of the hydraulic jack is referred to as $L_{\text{ref}}$. SteerToStation is the function that assigns initial strains into the truss elements to produce stresses that push the machine forward. Within this function, a new reference length ($L_{\text{new}}$) is computed based on the currently requested position. From which, the
prescribed strain is determined. It should be noted that the final elongation after solving the current
time step does not equal the difference \(L_{\text{new}} - L_{\text{ref}}\), but it is resulted from the prescribed strains
and the resultant forces acting on the shield. Therefore, the motion of the shield machine is governed
by the equilibrium of all the acting forces and the resulting trust forces represent the counter forces
acting on the lining to push the shield forward. The computational simulation in conjunction with
the aforementioned shield steering enables a more accurate prediction of the time-variant state of
stresses on the tunnel lining, in particular for any arbitrary curved alignment.

2.4.4 Pressurized Fluid Film within the Steering Gap

The shield skin is not only in contact with the surrounding soil but also with the liquid film within
the steering gap, this induces extra pressure on the shield. Therefore, ground deformations toward
the steering gap are mainly influenced by the shield machine characteristics (SRAMOON ET AL.
2002, SUGIMOTO AND SRAMOON 2002) and the process pressures filling this gap. In this section, a
computational method presented in (BEZUIJEN ET AL. 2012, NAGEL AND MESCHKE 2011) is used
to evaluate the viscous flow and pressurization within the steering gap. The proposed method uses
the contact algorithm to assess the mutual interaction between the shield skin and the surrounding
soil. Meanwhile, a realistic simulation of shield advancement is achieved by the fully automatic
steering algorithm, see section 2.4.3.

This algorithm uses Finite Difference Method (FDM), that computes explicitly the pressures
before each time step using the gap width obtained from the previous solution step. Then, the fluid
pressures are transmitted to the Gauss points and the contact algorithm is modified to account for
these pressures. The resulting fluid pressures are kept constant within the implicit solution for the
global equilibrium. In the aforementioned algorithm, following assumptions are made:

• The process liquids are assumed to behave as BINGHAM fluids with shear strength of \(\tau_{\text{grout}} =
1.6 \text{ kPa}\) and \(\tau_{\text{bentonite}} = 0.8 \text{ kPa}\) according to BEZUIJEN (2007)

• The fluid flow is a one dimensional process along the longitudinal direction of the shield and
the fluids are in a state beyond their shear strength

In (BEZUIJEN 2007, 2009), the evolution equation of the fluid pressure is stated by

\[
\frac{\delta p}{\delta x} = - \frac{\tau_y}{g(x)} \cdot \text{sgn}(v_x),
\]

(2.28)

where \(\tau_y\) represents the shear strength of the respective fluid, \(g(x)\) is the annular gap width, \(v_x\)
is the flow speed and its sign \(\text{sgn}(v_x)\) preserves a pressure reduction along the flow. Herein,
the positive X-direction is set from the tail to the front of the shield. This leads to a positive sign for the
grouting mortar flow. Therefore, the pressure gradient is negative in the calculations and vice-versa
for bentonite. This formula neglects the viscosity since low flow velocities are expected compared
to the yield stresses.

Using equation 2.28, the pressure distributions of the bentonite suspension and the grouting
mortar are evaluated separately as it is assumed that the grouting mortar does not mix with the
bentonite. The prescribed pressure boundary conditions, used in the analysis, are the face support pressure at the front of the shield and the grouting pressure at its back for each calculation step. To do so, the circumferential direction of the shield is divided at different angles with respect to the shield’s FE-discretization as indicated with black dots in figure 2.31-a. At each angle, a 1D FDM is adopted to solve the differential equation 2.28. A central FDM scheme is used to express the spatial derivatives. The pressure gradient is evaluated at the center of the element as:

\[
\frac{\partial p}{\partial x}_{i+1} = \frac{p_{i+1} - p_i}{h_{i+1}} = -\frac{\tau_y}{g(x_c^{e_{i+1}})} \cdot \text{sgn}(v_x)
\]  

(2.29)

In this equation, \(x_c^{e_{i+1}}\) is the central point of element \(e_{i+1}\) and the gap width is computed at this location. Equation 2.29 constitutes a set of linear equations for the pressure gradient of both, the grouting mortar and bentonite. With the assumption that the grouting mortar does not mix with the bentonite, the pressurized fluid film pressure is considered as the larger pressure at each node in the FD mesh, figure 2.31-b.

\[
p_{\text{fluid}}(x, \varphi) = \max[p_{\text{grout}}, p_{\text{bentonite}}]
\]  

(2.30)

Figure 2.31: Process liquid pressure around shield machine: (a) shield skin subdivisions at different location for the computation of fluid pressure and (b) FDM at a certain location along the shield axis with the final pressure combination at the steering gap

A modification in the contact algorithm has to be introduced to account for the aforementioned fluid pressures. The fluid pressures \(p_{\text{fluid}}\) at the nodes of the elements are transmitted to the Gauss points, then, their effect is considered as an equivalent compressive stress acting along the outward unit normal vector on both, the master and slave surfaces. Additionally, a contact separation occurs if the fluid pressure exceeds the contact pressure. Subsequently, the normal contact forces yield to:

\[
t_N = \begin{cases} 
p_{\text{fluid}} & \text{if } p_{\text{fluid}} > \epsilon_N \langle g \rangle \\
\epsilon_N \langle g \rangle & \text{else}
\end{cases}
\]  

(2.31)
2.4.5 Simulation of the Construction Process

In tunneling simulations, the size of the domain should be chosen in a way that the model boundaries do not affect the results in the tunneling vicinity. Generally, the primary state of stresses at the boundaries should not change (Potts and Zdražkovíc 1999). Figure 2.32 shows the ground domain with the prescribed boundary conditions for the simulation of a fully saturated soil using two phase formulation. These boundary conditions remain unchanged during the simulation.

\[ \begin{align*}
    u_x &= 0 \\
    P_w &= P_w^0 \\
    u_y &= 0 \\
    u_z &= 0 \\
    P_n &= 0 \\
\end{align*} \]

**Figure 2.32:** FE mesh of the ground with boundary conditions for the displacements components \( u_x, u_y, u_z \) and pore pressure \( P_w \).

To account for the primary stress state in the soil, the respective values can be either explicitly given to the model or implicitly determined. In the simulation model, the second approach is adopted, in which, a two-steps procedure is followed in the beginning of the analysis. In the first step, the ground model is analyzed under its own weight with the aforementioned boundary conditions. At this point, the ground is assumed to behave elastically and all the other model components are deactivated. The output stresses of this step correspond to:

\[ \begin{align*}
    \sigma'_z &= \sigma_z + u_w; \text{ where } \sigma_z &= -\gamma_{sat} * h \quad \text{and} \quad u_w = \gamma_w * h_w \\
    \sigma'_x &= \sigma'_y = K_0 * \sigma'_z; \text{ where } K_0 = \nu/(1 - \nu) \tag{2.32}
\end{align*} \]

Using the `InsituStressUtility`, the stresses at the GAUSS points are transmitted as pre-stresses. In this utility, a predefined value for \( K_0 \) can be imposed. Then, the second step solves the equilibrium equation with gravitational loading and pre-stressing. The output ground deformation is checked to ensure that it yields to zero, while, the in-situ state of stress is preserved inside the ground.

The aforementioned scheme serves as a basis to determine the primary stress state, that is followed by preliminary steps as shown in figure 2.33. These steps start with the initialization of the contact analysis. The shield is activated and positioned at its starting location and the excavated ground is deactivated (figure 2.33-a). In addition, the face pressure and grouting pressures are applied. Then, the hydraulic jacks are initialized as well, the face pressure and cutting forces are applied on the shield. That leads to evaluation of shield deformation taking into account the contact forces from the ground, the applied loads and its own weight (figure 2.33-b).
Eventually, the step-by-step simulation is carried out as shown in figure 2.34. This is achieved by the repetition of two simulation steps: an excavation step and a ring construction step following the predefined time step for each. The excavation step includes the use of the SteeringUtility to position the shield. This movement is accompanied with the deactivation of the soil and the activation of the grout. Afterwards, the ring construction step is performed by the activation of the lining ring inside the shield accompanied with the resetting of the hydraulic jacks elements on the face of the newly installed ring. Once the shield reaches the final excavation step, the simulation stops.
Figure 2.34: Repetitive scheme for the step-wise simulation of mechanized tunneling process: (a) stand still position, (b) shield advancement and soil excavation achieved by means of the steering algorithm and the de/re-activation of the respective elements, (c) ring construction and resetting of the hydraulic jacks
Chapter 3

Analysis of Segmental Tunnel Lining

Tunnel linings are designed to permanently fulfill basic structural, serviceability and durability requirements throughout the lifetime of a tunnel. In order to ensure structural stability, it is important to correctly assess the response of tunnel linings with respect to the external and process loads to which lining structures are subjected. Within this chapter, a short overview of the basic characteristic of precast concrete segmental tunnel lining is presented followed by literature review of the various models used for the structural analyses. Emphasis is placed on three aspects; the load spreading assumption, the consideration of joints and the representation of soil-lining interaction.

3.1 Introduction

Segmental tunnel lining design is primarily based upon two assumptions, that of the structural model and that of the loading scenarios to which the structure is subjected. The structural model used for design must be able to replicate the dominant physical features that result in the observed segmental response and the loading assumptions must accurately represent the actual in-situ time-dependent processes and ground loading to which the lining is subjected. Several analytical and numerical structural models with different loading combinations have been proposed for this purpose, yet most make significant simplifications to the relevant physical processes that control lining behavior. Numerical bedded beam models, such as those proposed by the German Tunnelling Committee (DAUB) (2013), only represent the ground/grout as radial springs, and many finite element based continuum models used in practice are restricted to 2D, as it is assumed that the longitudinal response of the lining and ground can be largely neglected or simplified using approximation methods (Karakus 2007, Möller and Vermeer 2008). When observed in the context of practical engineering applications, these simplifications are understandable, and in fact necessary, for the sake of computational efficiency and expediency in the model generation process. However, in order to
accurately identify the underlying factors that control segmental lining response, and to be able to determine which simplifications can be made without sacrificing the model accuracy. It is necessary to develop a model that explicitly takes into account all physical features inherent to a segmental lining system.

With respect to the structural analysis of tunnel linings, various analytical solutions and numerical models have been proposed and improved over time. Analytical solutions for the stress state and deformations of segmental linings can be classified as either bedded beam solutions (Schulze and Duddeck 1964), in which the ground reaction is simplified as a continual bedding, or as continuum models, see e.g. (Ahrens et al. 1982), in which the ground is considered as either a perfectly elastic or an elasto-plastic half-plate with a reinforced or unreinforced circular opening. These models invariably result in a complex set of differential stress and displacement balance equations and because of this, solutions are only provided under the assumption of in-situ geological loadings. Important additional loadings or load states needed for comprehensive lining design, such as those resulting from grouting pressure or surcharge loads, are therefore overlooked. The applied loadings are also often simplified by neglecting the increase of ground loadings across the tunnel’s height to simplify the derived solutions (Ahrens et al. 1982, El-Naggar and Hinchberger 2008, Schulze and Duddeck 1964). As a result, analytical models can provide important reference values for use in lining analyses, but are generally not accurate or flexible enough to provide design relevant structural forces.

Numerical models enable a more realistic analysis of the physical problem. Numerical models used for lining analysis can also be generally classified as bedded structural models, where surrounding ground is modeled as springs (Arnaud and Molins 2011, 2012, Blom et al. 1999, Klappers et al. 2006), or 2D/3D continuum models (Kasper and Meschke 2004b, Lambrughi et al. 2012, Möller and Vermeer 2008, Zhao et al. 2017), in which a higher degree of detail can be included. Most design recommendations (German Tunneling Committee (DAUB) 2013, Japanese Society of Civil Engineers (JSCE) 1996, U.S. Federal Highway Administration (FHWA) 2004), however, only provide explicit guidelines for the development of numerical bedded beam models for use in segmental lining design.

Numerical beam models (German Tunneling Committee (DAUB) 2013, Klappers et al. 2006) represent the lining system as a bedded beam structure. In addition, models exist in which bedded shell/volumes elements are used for lining discretization (Abd-Elrehim and Asaad 2017, Arnaud and Molins 2011, 2012, Blom et al. 1999). In such models, the soil is represented by discreet springs, in contrast to a constant bedding, whose stiffness is determined as a function of the bulk modulus of the surrounding soil, the radius of the lining and the influence area of the spring, i.e. by the discretization. The lining can either be considered as a continuous structure, or longitudinal joints can be accounted for as hinges or rotational springs at the segment ends. As per German Tunneling Committee (DAUB) (2013), the influence of consecutive rings can be taken into account by including multiple rings of slightly smaller diameters and by coupling these using non-linear differential displacement springs. The displacement springs represent the foreseen shear coupling mechanism, i.e. Cam-and-Pot connection or steel/rubber dowels, and are included at its location. These models are fairly straightforward to implement, can take into account ring
and segment coupling, and are computationally efficient. However, ground reaction is reduced to a constant radial spring and any non-linear behavior of the ground, e.g. due to shear failure, etc. must be implicitly accounted for in the applied load distributions. The shear coupling between ground, grout, and the soil, and any corresponding tangential loading of the segment, is often neglected or simplified.

Numerical continuum models (Kasper and Meschke 2004b, Lambrughi et al. 2012, Möller and Vermeer 2008, Zhao et al. 2017) offer an opportunity to address many of the problems posed by bedded models. Most importantly, numerical continuum models offer the possibility to model the surrounding soil medium explicitly. 2D continuum models are, however, relatively inefficient, as sequential loading stages can only be poorly analyzed and because the plain-strain assumption often leads to an over-estimation of the ground stiffness following excavation. Approximation methods such as strength reduction method, stress relief method and volume loss control method (Abd-Elrehim and Abu-Krisna 2006, Karakus 2007, Möller and Vermeer 2008) must therefore be employed to arrive at more realistic lining design parameters. In contrast, 3D numerical continuum models do not have any of these drawbacks, as 3D numerical continuum models do not incorporate such assumptions or simplifications.

A properly implemented 3D continuum model can account for all relevant aspects of the mechanized tunneling process, i.e. complex construction processes, various interactions between different components, complex geological stratification, detailed geometrical representation of individual tunnel components, and the non-linear material response of individual components. 3D numerical models can explicitly account for load evolution by accounting for history variables, such as grout hardening or soil plastification, that determine the spatio-temporal evolution of the load acting on the lining. However, performing such 3D simulations of a tunneling problem require a large computational effort and extensive experience of the users in order to properly generate a model and to perform successful analyses. Detailed numerical models of an entire tunnel drive can take multiple hours up to days to obtain solutions (Stascheit 2010). This, in practice, relegates the use of 3D models in design to special scenarios and often results in 3D models being simplified to decrease computation time.

One simplification that is generally made in implementing a 3D continuum model is to model the lining as a continuous ring, either using shell elements or volume elements. It is obvious, however, that the kinematics of a segmented lining cannot be fully represented by a continuous monolithic cylinder. Still, segment interactions are ignored in the pertinent literature (Kasper and Meschke 2004b, Lambrughi et al. 2012, Möller and Vermeer 2008, Zhao et al. 2017). This is most likely because, the accurate ground deformations are the parameter of interest in most 3D tunnel models instead of the lining stresses and their evolution. Additionally, for practicing engineers, arguments have been made to neglect or to otherwise simplify lining segmentation. For example, Dudgeon (1980) suggests that joints should only be explicitly included in the model if rotations are observable. However, recent publications have highlighted the effect of joints on segment response using bedded rings as in (Arnaud and Molins 2012, Blom et al. 1999, Klappers et al. 2006) or continuum models as in (Do et al. 2014a, Guan et al. 2015, Kavvadas et al. 2017). If used for the investigation of lining loads, rigid-lining models can only provide an upper
bound for structural bending forces (Arnaud and Molins 2012, Koyama 2003). It is shown in (Do et al. 2014a, Kavvadas et al. 2017) by comparison with rigid-lining that the introduction of joints and its orientation affects the lining flexural rigidity. Moreover, ground condition and tunnel overburden have influences on the bending moments reduction due to segmentation as discussed in (Guan et al. 2015).

To the best of the author’s knowledge, the segmentation of mechanized tunnel linings has not been incorporated explicitly via contact in a 3D process oriented simulation model except in (Ye and Liu 2018). Many models in literature tend to characterize the ring coupling via interfacial springs which do not realistically account for ring-to-ring friction as the dominant shear-coupling phenomena between consecutive rings. In this thesis, the structural response of segmental concrete lining is investigated by means of a process oriented finite element analysis, in which the actual physical interactions between segments along joints in both longitudinal and ring directions are considered using an explicit contact algorithm. The proposed process oriented model enables the realistic evaluation of the loading on lining during multiple loading stages. The proposed model is a modification of the shield tunnel advancement model ekate (Nagel 2009, Staschitz 2010), in which the geometrical representation of segmented lining has been improved by including longitudinal and circumferential joints. The main goal of this thesis is to investigate, if, and to which extent the precise consideration of the lining kinematics plays a role in regards to the tunnel-soil interaction. The effect of segments’ interactions on the induced structural forces in tunnel linings is demonstrated by a numerical example in chapter 6.

3.2 Basic Characteristics

A segmental tunnel lining system consists of individual segments which are assembled into rings that form the entire lining system. The contact surfaces and resulting interactions between neighboring segments within an individual ring are referred to as longitudinal joints, while, the contact surfaces or coupling method between sequential lining rings are referred to as the ring joints, see figure 3.1. Certainly, the type of shield and geotechnical conditions have a great influence on lining design and the possible types of joints. A lining ring usually consists of five to eight segments plus a keystone. Different design variants exist for the geometrical shapes of lining segments, they can be rectangular, rhomboidal, trapezoidal or hexagonal. Rectangular shaped segments are the most commonly used ones. The keystone is either wedge-shaped and smaller than other segments, or large and has the same size of other segments. Segments/rings width varies between 1.0 m to 2.0 m, while the thickness is in the range of 20 to 50 cm. The use of larger segments leads to faster advancement and less joint length. However, this requires larger lifting capacity for the erector. In addition, driving along curves becomes more difficult with wider rings.

Figure 3.2 shows a detailed drawing of a rectangular segment. The later is designed with flat contact sides and straight inclined bolts in both longitudinal and ring joints. Its basic reinforcement consists of traditional steel rebars cage at the sides of the segments; this promotes the resistance against bending resulted from non-uniform radial loading. Different standards and recommendations pro-
3.2. BASIC CHARACTERISTICS

**Figure 3.1:** Illustration of lining segments layout with longitudinal joints and ring joints


During excavation, the shield machine follows the designed tunnel alignment as close as possible, as well, the erected rings follow the shield path. Therefore, ring tapering is required to enable the construction of a curved tube. For the geometrical design of segmental concrete lining rings, there exists different types of rings as shown in figure 3.3 which differ in assembly in the construction stage, still, this do not affect its function. Precast segmental rings with parallel sides as shown in figure 3.3-a may be used to construct a straight tunnel, whereas tapered rings can accommodate curved alignments (figure 3.3-b and c). This can be achieved by two systems; the universal ring or left-right rings. Rings constructed using tapered segments are slightly conical. By which, curved tunnel alignments can therefore be built through rotation of sequential rings (SWARTZ ET AL. 2005). The use of universal ring geometry promotes a rapid ring construction and facilitates the project logistics since only one ring type is being used. Curvature is achieved by the proper ring rotation. As a consequence, the position of the key stone varies and might be located in the invert which complicates ring construction. With left-right rings, the key segment or keystone is typically placed at the tunnel crown which enables ring construction from the bottom up. Curved lining is built by installing either left or right rings with the appropriate rotation, while, straight alignments are built by alternatively installing between left and then right segments. Tapered segments have for these reasons become the most common segment design used in practice. Ring tapering is specified according to the minimum radius of curvature of the tunnel alignment with respect to ring diameter and width following the relation (Tapering = Width / Diameter / Radius). More detailed illustrations of various segmented lining types with its different joints detailing, waterproofing and erection can be found in (GUGLIELMETTI ET AL. 2007, MAIDL ET AL. 2013).
Figure 3.2: Detailed drawing of a typical rectangular precast concrete segment in a 7+1 ring layout; concrete dimensions in mm (top) and traditional reinforcement details using rebars (bottom)
3.2. BASIC CHARACTERISTICS

3.2.1 Segmental Lining Joints

The high degree of joints is a key characteristic in segmental lining. Joints are mainly ring joints for ring to ring coupling or longitudinal joints for segment to segment coupling. Joints should preserve the structural integrity by transmitting the structural forces between the segments. Choice of an appropriate joint detail is based on its load-bearing capacity, waterproofing, risk of spalling, etc.

Longitudinal joints

Longitudinal joints between individual segments within a single ring primarily determine the radial bending stiffness of an individual ring. These joints can be classified with respect to the geometry of its contacting surfaces. Various longitudinal joint designs have been proposed in practice as:

- Flat longitudinal joint
- Convex longitudinal joint
- Convex/concave longitudinal joint

Flat longitudinal joints are characterized by $\approx 50\%$ reduction of the lining thickness at the flat segment to segment contact surface. Joint thickness identifies its rotational behavior. The latter generally parallels that of a concrete joint (JANSSSEN 1983). The thickness of the contact surface is determined based on the desired rotational stiffness of the joint and on the expected concrete splitting forces that develop at the longitudinal joint as a result of compressive force transfer (GERMAN TUNNELLING COMMITTEE (DAUB) 2013). With an adequate bending stiffness, a reduction in the joint thickness promotes equilibrium by enabling the resultant forces of the transferred compressive stresses through the joint to act inside the core of the joint. As a consequence, splitting tension due to stress concentration should be checked as well sealing has to be ensured (MAIDL ET AL. 2013).

Flat joints are most commonly used in modern engineering practice. In special cases, however, non-flat joint designs may also be seen. BAUMANN (1992) recommended convex joints for

![Diagram of segmental concrete lining rings with respect to ring tapering for the construction of straight and curved alignments.](image)

**Figure 3.3:** Types of segmental concrete lining rings with respect to ring tapering for the construction of straight and curved alignments: (a) straight ring, (b) tapered ring (left and right rings) and (c) universal ring (GUGLIELMETTI ET AL. 2007)
joints with high compressive forces and large rotations, where contact area is ensured even with an increased rotation. As well, convex-concave joints may be used if ring compressive forces are expected to be exceptionally high (Osgouí et al. 2017). Similarly, in tunnels with slipping-prone keystones, tongue and groove longitudinal joints are used (see figure 3.5). In addition, the use of guiding rods provide more guidance for assembly (German Tunnelling Committee (DAUB) 2013). Such designs are, however, associated with higher risk of concrete spalling.

**Ring joints**

Specifications for ring joints determine the shear coupling method between consecutive rings within the lining. The newly installed ring are loaded with axial thrust forces via thrust pads and ring joints mainly transfer this axial loading through successive rings, as well as, they govern the rings’ coupling mechanism. Figure 3.5 shows the most commonly designed ring joints, which can be summarized as:

- Flat ring joint
- Tongue-and-groove system
- Cam-and-pocket system

Ring joints may be designed as flat surfaces, in which case the resulting ring coupling mechanism is achieved mainly through ring-to-ring friction. Plastic dowels may be used in connection with flat ring joints in order to aid the assembly procedure. These simplify the ring assembly and provide minimal additional shear resistance (German Tunnelling Committee (DAUB) 2013, Maidl et al. 2013). In practice, such designs are being increasingly implemented, but in theory, however, it is unclear how the magnitude of available shear resistance is to be calculated. For this reason, additional coupling elements such as cam-and-pocket systems, rubber bi-cones or steel dowels, are often included in the segment and ring designs. These methods provide a sort of cantilever support to resist relative radial displacements between consecutive rings.
If high shear forces in the ring are to be expected, such as at cross passages, steel dowels are foreseen in the ring design, and if only minimal shear forces are expected, rubber bi-cones may be used. A stronger mechanical coupling between successive rings is achieved by using tongue-and-groove or cam-and-pocket systems (figure 3.5). Cam-and-pocket connections are laid out for more general loading cases. In a tongue-and-groove system, the coupling forces are along the joint width, while, coupling in cam-and-pocket are achieved at certain points (i.e. quarter points of the segment). The groove/pocket are generally larger than the tongue/cam within few millimeters to avoid problems associated with constructions tolerances (MAIDL ET AL. 2013). For transferring higher coupling force, cam-and-pocket shall be reinforced. However, pocket edge has to be stiffer than the cam. This ensures water tightness, if damage occurs, as the cam will be sheared off first (GERMAN TUNNELLING COMMITTEE (DAUB) 2013). The joints are to be designed with a packing material between them in order to facilitate a smooth force transfer.

A final factor that affects both the bending behavior and shear stiffness of segmental tunnel linings is the joint arrangement in successive rings. Staggered joint placement in consecutive rings provides additional bending support, whereas aligned joints result in a response similar to that of a single lining ring. If joints are installed in a staggered arrangement, segment bending in successive joints tends to result in contradicting displacements. Neighboring rings therefore resist each other’s bending behavior through the shear coupling mechanism, resulting in a generally stiffer ring response. For this reason, some design codes recommend installation of sequential rings in a staggered pattern GERMAN TUNNELLING COMMITTEE (DAUB) (2013), which also improves the water tightness of the lining structure. Furthermore, depending on the relative displacement between successive rings, the ring coupling method may be more or less effective depending on its placement. If no or little relative displacement occurs at the location of the ring coupling element between successive rings, the ring coupling method will not provide the intended support.
Connections

In order to provide temporary support within the shield skin and to ensure primary ring compression immediately after construction, segments within a ring are typically secured using segment bolts. These bolts may be removed after a certain time, if they are primarily to be used as a construction aid, or be left in the lining permanently (Harding et al. 2014). In addition, connectors are also used between consecutive rings in order to facilitate assembly or to provide additional shear resistance as discussed in the previous subsection.

Figure 3.6: Illustration of the different connecting systems in concrete segments: shear dowels in ring joints and curved bolts in longitudinal joints (top) and inclined bolts in both ring and longitudinal joints (bottom)

Possible variants for connecting segments are plastic/steel dowels, inclined bolts and curved bolts, see figure 3.6. These bolts are, however, not typically intended to provide additional rotational capacity. As can be seen in figure 3.6, the bolt axis typically passes through the center of the longitudinal joint. This placement ensures symmetrical rotation behavior and minimally affects the rotational stiffness (Majdi et al. 2016). Again, parallels may be drawn to concrete joints, as traditional reinforcement layouts for concrete joints pass through the joint hinge point, i.e. the center of the joint (Leonhardt and Reimann 1966, Tvede-Jensen et al. 2017).
3.3 Structural Models for Segmental Tunnel Linings

Tunnel lining has to permanently fulfill some basic requirements concerning structural safety, serviceability, and durability for the entire live time of the tunnel. In practice, tunnel linings provide continuous support for the soil medium and prevent water flow inside the tunnel. With respect to the structural analysis of tunnel linings, various analytical solutions and numerical models have been proposed and improved over time. Analytical solutions and numerical models can be generally classified as bedded structural models and continuum models. In bedded models, the ground reaction is simplified as a continual bedding or numerically discretized as springs. While, the ground and its nonlinear behavior are explicitly considered in continuum models which are generally able to include a higher degree of detail (i.e. the ground and its constitutive behavior are explicitly considered as part of the model). In addition, the actual loading on lining is not only influenced by the geological condition but also the construction conditions and it experiences significant changes with time. During construction, lining rings are being erected inside the shield while they provide thrust resistance for the advancement of the shield machine. With shield movement, the erected ring leaves the shield concurrently with confinement by the pressurized grouting mortar at the annular gap. Accordingly, loading on tunnel lining, during construction, can be broadly divided into longitudinal loadings induced by the thrust jacks and circumferential loadings induced by the surrounding grouting mortar and soil. Safe and cost efficient tunnel lining designs require a reliable determination of the expected time-variant stresses and deformations in the lining and prognoses of possible critical loading conditions. Methods with which to account for lining segmentation in structural models for tunnel linings range from simple modifications of analytical solutions to direct inclusion of joint behavior by means of contact, rotational springs or interface elements in large scale 3D numerical analyses.

3.3.1 Analytical Solutions

There are a number of analytical models for the structural design of tunnel lining. The first solutions for an elastic continuum were introduced in the early part of the 20th century (SCHMID 1926, VOELLMY 1937) in which the lining is assumed to be bedded within an elastic domain. Later on, further developed models were proposed, e.g. (AHRENS ET AL. 1982, EINSTEIN AND SCHWARTZ 1979, HAIN AND HORST 1970, MORGAN 1961, SCHULZE AND DUDDECK 1964, WINDELS 1967, WOOD 1975). In these models, different scenarios are assumed for the representation of the lining-soil interaction, e.g., shallow tunnels are characterized by lack of support at the crown as shown in figure 3.7.

Such models require introducing further simplification to the system; as well, they adopt significant simplified loading assumptions. SCHULZE AND DUDDECK (1964) proposed a closed form solution in which a one dimensional bedded structure is used. The solution is based on circular beam theory (neglecting shearing forces and torsional moments) accounting for the second order series expansion of the solution. A commonly adopted loading assumption, i.e. the active soil pressure equals to the in-situ state of stresses of the undisturbed ground, is shown in figure 3.8. The surrounding ground is assumed to be an elastic medium and represented numerically via an addi-
CHAPTER 3. ANALYSIS OF SEGMENTAL TUNNEL LINING

60

shallow tunnel

deep tunnel

\[ D_h < 3D \]

\[ D_h \geq 3D \]

continuum model

partial bedded model

continuum model

bedded model

partial

loads

only

\[ \beta \]

\[ 90^\circ \leq \beta \leq 100^\circ \]

Figure 3.7: Lining bedding assumption used in analytical continuum and bedded models (PUTKE 2016); partial bedding (left) and full bedding (right)

Functional bedding term in the force balance equation. The bedding stiffness is determined from the following expression:

\[ K_r = f \cdot \frac{E_s}{r} \]

(3.1)

where, \( E_s \) and \( r \) are the soil elastic modulus and the lining radius. A scalar parameter \( f \) is introduced ranging between \( \frac{2}{3} \) to 3 according to the loading condition and the properties of the lining (i.e. continuous or segmented). In this model, bedding is neglected at the crown of the tunnel. As mentioned earlier, this assumption is typically made for shallow tunnels where full arching effect of the ground above the tunnel may not fully develop.

without bedding

\[ \sigma_v = \gamma (h - r \cos \phi) \]

\[ \sigma_h = k_v \sigma_v \]

Bedding stiffness

\[ K_r \]

Figure 3.8: The in-situ stress loading assumption with partial bedding for the analytical solution by SCHULZE AND DUDDECK (1964)
3.3. STRUCTURAL MODELS FOR SEGMENTAL TUNNEL LININGS

A continuum model by Ahrens et al. (1982) assumes a perfectly circular thin ring of a given thickness bedded in an infinite elastic half plate. A simplified in-situ stress state, as shown in figure 3.9, is the basis for loading assumption; the linear variation of the in-situ loading across the height is neglected. Uplift of the tunnel is therefore also neglected (i.e. the vertical forces remain in equilibrium). Loads are transformed to the polar coordinates and the tangential load is neglected considering that the existence of grout reduces the frictional bond between the lining and the soil. The analytical solution is based on a kinematic assumption in which both lining and the soil have similar radial deformation and the in-situ stresses are balanced with the resistance forces of the lining and the soil.

**Figure 3.9:** Simplified loading assumptions used in the analytical continuum model by Ahrens et al. (1982) in which the vertical stresses are in equilibrium and the horizontal pressures are uniform

\[
P_v = \gamma h \\
P_h = k_h \gamma (h+r)
\]

Wood (1975) pointed out the improper assumptions that lead to the overestimation of stresses and consequently a more conservative and uneconomic design. Contrary to the models presented in (Ahrens et al. 1982, Schulze and Duddeck 1964), Wood (1975) enclosed percentage of the total in-situ stresses to account for the ground relaxation. Thus, such simplified approach gives an estimate for the upper and lower limits for the lining responses, yet, it does not consider the complex nature of the 3D time-dependent stress state.

The Japanese Society of Civil Engineers (JSCE) (1996) proposed another closed form solution for calculating member forces of circular tunnels. Figure 3.10 represents design loads that are being incorporated in shield tunnel lining design. A closed form formula exists for the calculation of normal forces and bending moments for each load component. The proposed loading distributions are in equilibrium, in both vertical and horizontal directions, and the lateral pressure varies linearly with depth. These loading conditions and its analytical solutions are adopted by the U.S. Federal Highway Administration (FHWA) (2004) as well. In addition, the overburden depth defines the loosening pressure for the determination of the vertical earth pressure as a result of soil arching; Terzaghi’s formula (Terzaghi 1943) can be used to determine the arching effect (i.e. reduction of vertical pressures).
**Consideration of lining joints in analytical solutions**

The effect of joints in the analytical lining models have been implicitly or explicitly accounted for. The simplest method with which to account for ring segmentation is by reducing the bending stiffness of continuous ring models (Einstein and Schwartz 1979, Lee and Ge 2001, Wood 1975). The most commonly used approach is that proposed by Wood (1975), in which the reduced effective lining stiffness, $I_e$, is obtained as a function of the continuous lining stiffness, $I$, the number of longitudinal joints in the ring $n$ and the longitudinal joint stiffness, $I_j$, as follows:

$$I_e = I_j + \left(\frac{4}{n}\right)^2 I \quad (I_e \neq I, n > 4)$$

(3.2)

Lee and Ge (2001) proposed an iterative approach to estimate the effective rigidity ratio of segmental lining $\eta$, expressed as:

$$\eta = \frac{EI_{eff}}{EI_{cont}},$$

(3.3)

where $EI_{eff}$ and $EI_{cont}$ are the effective flexural stiffness of the equivalent continuous lining and the continuous flexural stiffness, respectively. The proposed iterative approach adopts an analytical solution, that accounts for the effect of joint, as a reference solution. Then, the effective rigidity ratio is updated within an iterative scheme in order that the vertical or horizontal displacement of the equivalent continuous ring matches the reference solution. The estimated ratio can then be elaborated in further numerical analyses assuming continuous lining incorporating the effective stiffness.
Another estimate of the flexural stiffness reduction has been introduced by Blom (2002) as:

\[ \eta = \frac{1}{1 + \frac{3}{4} \frac{t^3}{l^2 R} (C_x + C_y)}, \]  

(3.4)

where \( t \) is the segment thickness, \( l \) is the longitudinal joint length and \( R \) is the lining radius. The parameters \( C_x \) and \( C_y \) are state parameters that should be determined based on the orientation of the joints.

Similarly, a stiffness reduction method has also been proposed by Koyama (2003); the effective rigidity ratio is estimated by comparing the radial deformation at the sides for the stiff ring and the jointed ring and can be determined by:

\[ \eta = \frac{D + \Delta D_{\text{side, ring}}}{D + \Delta D_{\text{side, jointed ring}}} \]  

(3.5)

In addition to the stiffness reduction, ITA Working Group No. 2 (2000) adopted a transfer ratio (\( \xi \)) following the fact that higher moments arise at the middle of the segments and vice versa at the joints location. The moment distribution is modified to be \( M(1 + \xi) \) and \( M(1 - \xi) \) at the segment and at the joint respectively as indicated in figure 3.11. Such parameter depends mainly on the joint characteristic, however, there is no specific scheme to estimate such a parameter according to the knowledge of the author.

Figure 3.11: Moment distribution at the joint and the middle of the segment within a uniform rigidity model according to ITA Working Group No. 2 (2000)

Stiffness reduction methods, however, can only serve to provide first-order estimates of stresses and strains within tunnel linings. The use of uniform rigidity does not account for joints’ orientation and characteristics. Such an approach ignores the complex characteristics of the joint. Therefore, further analytical models tend to include the jointed rings by directly incorporating joint stiffness to characterize joint rotation.

Lee et al. (2001) presented an analytical solution for predicting the deformations and member forces in segmental lining considering lining joints. The force method has been used as the basis for the solution in which joints are depicted by means of joint flexural stiffness. Laboratory test were performed to provide a proper determination of the joint stiffness. The loading assumptions adopted in these calculations are similar to those shown in figure 3.10. The outputs showed that the joint stiffness has a significant influence on the bending moments distribution and not on axial forces. Although Lee et al. (2001) noted that the joint stiffness is influenced by the level of bending moments, axial forces and pre-stressing in the bolting, the flexural stiffness is assumed to
be constant. This model is restricted to a symmetric joint distribution and only models one ring, therefore, ring to ring coupling is ignored. As well, it does not ideally model lining-soil interaction.

**Blom** (2002) provided an analytical solution for two coupled rings as shown in figure 3.12. The two lining rings are radially loaded and the applied load is divided into a uniform compression loading $\sigma_{\text{comp}}$ (that dominates the ring compression) and an ovalisation loading $\sigma_{\text{oval}}$ (that dominates the bending stresses within the ring). The load distribution is characterized according to:

$$\sigma_{\text{total}} = \sigma_{\text{comp}} + \sigma_{\text{oval}} \cdot \cos(2\theta).$$

(3.6)

A non-linear rotational stiffness is used to describe the behavior of the longitudinal joint. The non-linearity was extended by incorporating the non-linear material behavior of concrete. **Blom** (2002) describes the joint behavior in three stages; in the first two stages, the ultimate concrete compressive strains are in the elastic stage, and a third stage in which the maximum compressive concrete strain is reached, and the concrete experiences plastic strains. On the other hand, ring coupling is assumed to occur only by friction through contact of packing materials, see figure 3.12-right. The frictional coupling force is determined as the multiplication of the relative radial deformation at the coupling point by a linear shear stiffness. In this sense, if both rings deformed equally, no interaction will occur. The lining-soil interaction is considered through continuous radial bedding with a linear stiffness and a reduction factor is introduced to account for the loads carried by the soil.

**Figure 3.12:** Analytical model developed by **Blom** (2002); the geometry of the two lining rings including joint location (left) and schematic overview and structural model of the ring-to-ring coupling by means of shear springs and the longitudinal joints by means of rotational springs (right)

**El-Naggar and Hinchberger** (2008) derived an analytical solution for jointed tunnel lining. The lining is introduced as a composite inner thin shell and outer thick cylinder. Within the
solution, rotation of the joint is determined by its rotational stiffness. Homogeneous infinite elastic medium around the lining is assumed to represent the ground medium. Loading is idealized by a uniformly distributed vertical and horizontal loads. The solution is extended to different number of joints and different joint orientations. However, such a plain strain assumption restricts the solution to the situation of aligned joints. Neither ring to ring coupling nor staggered joints are applicable in the derivation.

Generally speaking, analytical solutions are simplified situations of the problem that provide a quick estimate of the member forces in the lining. The main disadvantages of such solutions are that they require many assumptions a priori and have general tendency for overlooking many physical details which can be summarized as:

- Lining is reduced to a structural beam element
- Soil structure interactions are reduced to a linear bedding
- The common use of undisturbed in-situ state of stress as the main design load
- The confinement due to grouting as well as hydraulic jacks’ thrust are usually not considered.

Analytical solutions, that include the effect of joints, adopt either an indirect method by the stiffness reduction or direct method by the inclusion of joints as springs. However, these solutions are restricted to special situations. Therefore, numerical methods, such as the finite element method, provide an opportunity to model complex scenarios. Starting from the in-situ state of stresses, the realization of step-wise excavation with respect to lining erection and shield advancement provides an opportunity to model an accurate stress distribution in tunneling vicinity. The explicit simulation of constructional loadings during tunneling process enables a precise prediction of loading on lining.

### 3.3.2 Numerical Models

A numerical model refers to any model for which the solution is arrived at by using a solution scheme which first discretizes the solution space and then uses algorithms (most often some form of matrix inversion) to converge to a solution. In this sense, any model solved using the Finite Element Method (FEM), the Finite Difference Method (FDM), or any generalized displacement method is here referred to as a "numerical model". Numerically, lining-soil interaction can be modeled by either direct bedding or by 2D/3D continuum models.

#### Bedded structural models

More commonly used structural models for the analysis of segmental tunnel linings are the 2D numerical bedded beam models that explicitly account for joint segmentation, such as those proposed by the Japanese Society of Civil Engineers (JSCE) (1996) or the German Tunnelling Committee (DAUB) (2013).

In engineering practice in Japan, the full-circumferential spring model is used alongside with the analytical solution by the JSCE (the analytical solution is discussed in the previous subsection). The structural model is a beam spring model as shown in figure 3.13. As presented in (Koyama 2003), a uniform rigidity ring or two jointed rings can be used in the analysis. The bedding stiffness does not only depend on the soil stiffness and the tunnel radius but also on the state of the annular gap...
grouting, where the stiffness increases after the grout hardens. Longitudinal joints are modeled by rotational springs, while, ring joints are modeled at the packer location either by rigid members or shear springs. The assumption of rigid connection is carried out by assuming equal displacements between neighboring ring at the ring joint locations, otherwise, the use of shear springs allows for relative displacements. In this model, the ground condition characterizes the applied loads (i.e. the

$$P_r = \gamma_w (h + D)$$

$$P_w = h_w \gamma_w$$

$$P_v$$

$$\lambda P_v$$

Figure 3.13: Numerical lining model presented by Koyama (2003); structural beam model with full bedding in radial and tangential direction (left) and main loading assumption considered including vertical earth pressure, horizontal earth pressure and water pressure in radial direction (right)

total earth pressure is applied for tunneling in cohesive soil with low permeability. On the other hand, effective earth pressure and water pressure are applied on the tunnel lining in a cohesionless soil with high permeability). A uniform pressure, denoted as $P_v$, in figure 3.13-right, is applied in vertical direction. It represents the soil weight above the tunnel calculated by the classical Terzaghi’s formula (Terzaghi 1943) with a reduction in overburden in case of deep tunnels. The horizontal earth pressure is applied with linear variation with depth using the coefficient of horizontal earth pressure $\lambda$ ($\lambda = 0.45 - 0.60$ for sandy soils and $\lambda = 0.40 - 0.80$ for clayey soils) (Koyama 2003). It should be noted that $\lambda$ for a clayey soil, represents the coefficient of lateral earth pressure for the soil and water together. While for sandy soils, the water pressure is applied separately perpendicular to the lining surface.

In the DAUB model, two rings of slightly different radii are considered in the analysis, see figure 3.14. The stiffness of the radial springs is approximately determined as $K_r = E_s / r$ where $E_s$ is the elastic modulus of the ground and $r$ is the radius of the lining, see also (Grübl 2012) for further details. For shallow tunnels in soft ground, partial bedding is assumed (i.e. no bedding at the crown). In contrary to JSCE model, tangential bedding is not accounted (only 1% of the radial spring stiffness is applied in tangential direction to maintain numerical stability). This assumption follows the fact that fresh grouting mortar has low shear stiffness. It is worth mentioning that, there is no specific recommendation given by the DAUB for the loading assumption used in the bedded model. However, for the static analysis of shield machines, the classical Terzaghi’s formula (Terzaghi 1943) is used for the analytical determination of the earth pressure loading (German Tunnelling Committee (DAUB) 2005). The difference between shallow and deep overburden is suggested
3.3. STRUCTURAL MODELS FOR SEGMENTAL TUNNEL LININGS

Nonlinear moment-rotation relationships used to describe longitudinal joints in segmental tunnel linings are typically based on the assumption that longitudinal joints function in a similar manner as concrete hinges, as investigated by LEONHARDT AND REIMANN (1966). This assumption was first introduced by JANSSEN (1983) but has since been adopted and modified by various authors, e.g. (BLOM 2002, TVEDE-JENSEN ET AL. 2017), and experimentally verified by HORDIJK AND GIJSBERS (1996). Although the construction of a concrete hinge varies significantly from that of a plane longitudinal joint between tunnel lining segments (a concrete hinge is a slender thinning of a cross section in a continuous concrete structure, whereas a longitudinal joint between two segments is characterized by two separate surfaces in contact) the phenomenological rotational response is similar. In LEONHARDT AND REIMANN (1966), a moment-rotation relationship (see figure 3.15) is provided to describe concrete joints as a function of the normal force $N$, bending moment $M$, concrete stiffness $E_c$, joint length $l$ and the joint rotation $\varphi$ (per meter of length). The relationship describes a linear behavior for the state in which a joint is closed, i.e. when $M \leq Nl/6$, and a nonlinear rotational behavior for an open joint state, $M > Nl/6$. The relationship is given as follows:

$$\varphi = \begin{cases} 
\frac{12M}{E_cl^2} & \text{if } M \leq Nl/6 \\
\frac{8N}{9(1 - 2\frac{M}{Nl})^2 E_c l} & \text{if } M > Nl/6
\end{cases}$$  (3.7)

As seen in equation 3.7, the moment capacity is provided as a function of the effective ring normal force. However, the mechanical properties are idealized as a bi-linear envelope as per JANSSEN (1983) in figure 3.15. Similarly, figure 3.16 shows the moment-rotation relationship at different normal force levels. Higher normal forces provides higher moment capacity. A typical idealization of the analytical solution of concrete joint is plotted as the dotted line in figure 3.16 in which 80% of the ultimate moment capacity defines the tangential stiffness.
The ring coupling between consecutive rings is accomplished by coupling the two rings using a non-linear differential displacement spring, located at ring coupling mechanism. In the case of cam-and-pot connections, figure 3.17 provides a typical geometry of the system with different states of deformations, from which, the mechanical behavior of the springs is defined by a piecewise force-displacement relationship. This relationship accounts for an initial slip, followed by elastomer compression and ends with concrete-to-concrete contact. Numerical beam models are fairly straightforward and are able to capture most of the dominant tunnel lining kinematics.
In addition to bedded beam models, lining could be discretized by shell or volume elements. Arnau and Molins (2012) analyzed the structural response of tunnel lining using 3D embedded shell elements. To investigate the ring coupling effects, eleven consecutive rings are modeled and the results are compared with a single ring model. Each ring consists of 7 segments plus a key stone. As shown in figure 3.18, concrete segments are modeled by quadrilateral shell elements, while nonlinear interface elements replicate the interaction between the segments at both longitudinal and ring joints. Noting that integration at the interface is employed along the length and the thickness of the joint in order to replicate the joint gaping. The interface element provides a rigid connection in compression and allows for joint opening in tension (Arnau and Molins 2011). The interface elements used in longitudinal and ring joints have the same properties. They are defined within the joint thickness along the complete width for the longitudinal joints and along the contact area of the packer material for the ring joints. The frictional response of the packer-concrete contact is described with Mohr-Coulomb constitutive relation.

Figure 3.18: Typical rectangular concrete lining segment of L9 subway tunnel used in the analysis of tunnel lining in (Arnau and Molins 2011); segment dimension, configuration of longitudinal joint and packer locations (left) and FE discretization of the segment using shell elements and interface elements for the representation of longitudinal and ring joints (right)

Figure 3.19: Loading assumption used in the analysis of Barcelona-L9 tunnel lining for the bedded shell model by Arnau and Molins (2012) including soil and water pressure
In the aforementioned model, the soil-lining interaction is realized by the spring stiffness $K_r$, $K_t$, and $K_l$ for the radial, tangential and longitudinal directions, respectively. The radial spring stiffness is determined by the soil elastic modulus $E_s$, Poisson’s ration $\nu$ and the lining radius $R$ where $K_r = E_s /[R(1 + \nu)]$ while the tangential and longitudinal stiffness are assumed as one third of the radial stiffness. The loading assumptions, presented in figure 3.19, is applied on the shell surface area. The analysis of this model is achieved in different loading steps; first, the lining is activated without bedding and longitudinal loading is applied on the installed ring. Then, the bedding is activated and ground loading is applied. Arnau and Molins (2012) pointed that the staggered configuration of the coupled rings provides higher stiffness compared with a single ring analysis. As well, thrust force produced by the shield machine is large enough and sufficient to preserve ring coupling.

In (Bloom et al., 1999), a 3D finite element analysis for the “Green Heart” tunnel lining is used to predict the stress distribution during tunneling. The model simulates 12 consecutive rings that include grout hardening, jacking forces, and the interactions between segments. The interaction between segments is realized via point to point interface elements. The location of the packing material defines the area where interface contact is defined. Figure 3.20 shows the developed lining model and the interface contact points at the packer. The interface contact transmits only compression in the normal direction and shear in the tangential direction. The axial forces are applied on the axial direction from one side to simulate the thrust forces. The grout hardening for the 12 rings model is realized as follows: one ring (the first ring) is not loaded or supported as a replication of the ring inside the shield. Then, five rings are loaded with grouting pressure that linearly varies with axial direction. The remaining six rings are loaded and supported by the ground loads adopting Duddeck’s loading assumptions (Duddeck 1980). The grout hardening is accounted by changing the bedding stiffness of the five grouting rings from zero to the ground bedding stiffness with assumption that the completely hardened grout has similar properties as the surrounding ground.

![Figure 3.20: Three dimensional finite element model for the "Green Heart" tunnel developed by Bloom et al. (1999) using interface elements for the representation of joints](image)
The bedded structural models have been used in different literature, see (GALL ET AL. 2018, KROETZ ET AL. 2018, MASHIMO AND ISHIMURA 2003, ORESTE 2007, VU AND BROERE 2018, WANG ET AL. 2015, YUAN ET AL. 2012, ZHANG ET AL. 2014). Within such models, however, the ground is overly simplified as a radial spring bedding, active soil pressures on the lining are commonly assumed to be equal to the primary stresses in the undisturbed ground and separate construction stages can only be taken into account with the assumption of separate loading conditions.

**Lining models in continuum domain**

The development of computer powers leads to increasingly advanced numerical continuum models. They include a higher degree of detail which overcomes the limitations of other methods. 2D continuum models, adopting plain-strain assumption, can serve as a basic design tool for the sake of computational efficiency. In order to simulate the third dimension effect in 2D models, approximation methods, such as the stiffness reduction method, load reduction method or volume loss method (KARAKUS 2007, MÖLLER 2006) must be employed to arrive at realistic lining design parameters. On the other hand, a proper implementation of a 3D continuum model provides a reliable tool for the evaluation of the tunnel lining response and more specially the spatio-temporal loading on lining.

With regard to segmental lining kinematics, some models embrace conservative simplification by modeling the segmental lining as a continuous monolithic cylinder as per (KASPER AND MESCHKE 2004b, LAMBRUGHI ET AL. 2012, MÖLLER AND VERMEER 2008, ZHAO ET AL. 2017, 2012). Rigid lining models, if used for the investigation of lining member forces, provide an upper bound of the stresses. Recently, the effect of joints within the 3D continuum models is being addressed in literature (DO ET AL. 2014a, KAVVADAS ET AL. 2017). A detailed discussion for the different 3D continuum models of mechanized tunneling simulation is already provided in section 2.3, as well, a brief description of the computational model "ekate", used within this thesis, is presented in section 2.4. Therefore, 3D continuum models that explicitly include segmental joints are presented herein.

DO ET AL. (2014a) proposed a 3-D continuum model to be used in the structural analysis of segmental linings. The lining is modeled by shell elements and joints are simulated by axial, radial and rotational springs, see figure 3.21. In addition, ring joints and longitudinal joints are assumed to have the same interaction mechanism.

**Figure 3.21:** Ring joint stiffness in the axial, radial and rotational directions as developed by DO ET AL. (2014a)

Similarly, the segmental lining, in the mechanized tunnelling simulation model by KAVVADAS ET AL. (2017), is also discretized with shell elements and the joints are included by springs. The joint behavior is characterized by 3 rotational and 3 translational stiffnesses. The longitudinal and ring joints are depicted by assigning the rotational stiffness around the tunnel axis and the shear
spring stiffness, as illustrated in figure 3.22. The remaining stiffness components are assigned with high values (i.e. compatible deformations). According to KAVVADAS ET AL. (2017), these remaining stiffness do not influence the lining response.

Figure 3.22: The numerical representation of longitudinal and ring joints via rotational springs and shear springs in the simulation model by KAVVADAS ET AL. (2017)

A detailed representation of the lining is performed in (CHENHUA ET AL. 2016), by which, the influence of a shaft excavation on an already existing tunnel is investigated. In this model, the lining tube is depicted as a rigid cylinder at the far field, while joints are only realized at the area of interest (i.e. nearby the shaft excavation). The contact interaction is used to describe the joint behavior between the segments as shown in figure 3.23. In addition, bolts are represented as beam elements and embedded in the segments at its ends. It should be noted that this model focused on investigating the induced lining deformations by an adjacent excavation, and it does not predict stresses and strains during the mechanized tunneling process.

Figure 3.23: 3D volume representation of the segmental lining model including segments joints via contact and embedded bolts (CHENHUA ET AL. 2016)

The structural analysis of tunnel lining, specifically in the process oriented simulation models, usually adopts a straightforward assumption of a linear elastic concrete response. If the failure mechanism of concrete is not the focus of the analysis, the linear elastic assumption is still valid and considerably reduces the computational effort. On the other hand, a more advanced model is required to predict the concrete damage in lining. In the context of process oriented simulation, the lining model, developed by YE AND LIU (2018) using the commercial software ABAQUS, are represented by volume elements, while, the material response is described with a concrete damage plasticity constitutive law. The conventional reinforcement, with steel rebars, is included either by embedded beam elements or by equivalent surface elements on both outer and inner surfaces. The
interaction between segments (i.e. segments joints) is depicted by the normal and tangential contacts at the mutual interfaces in a similar fashion to (Chenghua et al., 2016). The analysis is motivated by a case study of a tunneling project in China, where the EPB shield had to advance with a very slow rate (i.e. 2 rings per day) when excavating through a highly permeable soil. The main aim of this study is to evaluate the possible lining damage induced by the water inflow at the heading face in this situation.

### 3.3.3 Concluding Remarks

The tunnel lining design using bedded models are often analyzed adopting simplified loading assumptions, which improperly describe the soil structure interaction. Yet, the actual loading on the lining are resulting not only from the in-situ stresses and the hydro-static groundwater but also from constructional loads and stress relaxation in the tunneling vicinity. Therefore, these models have limitations for the realistic depiction of the structural behavior in the construction stage and operating stage, due to the involved simplifications. The latter can be briefly summarized as:

- Soil stresses are equivalent to the undisturbed conditions (in-situ stress assumption)
- The soil structure interaction is oversimplified by radial springs
- The lack of description of constructional conditions (e.g. successive excavation and shield support)
- The inability to model complex ground conditions and ground improvements

The numerical continuum models serve as a basis for a reliable prediction of the response of the tunneling process. These models provide advanced simulations of the physical problem since it enables the representation of the following:

- The complex construction process
- The various interactions between different components
- The complex geological stratification
- A detailed geometrical representation
- The non-linear material response
- The modeling of different strengthening techniques

3D numerical tunnel lining models, integrated in a process oriented simulation overcome the shortcomings of the bedded models. Different models have been already proposed by various authors, refer to section 2.3. This is of interest in order to reliably investigate the final state of stress on tunnel lining with respect to the influence of constructional and operational loading. To this end, the simulation model "ekate", as explained in section 2.4, is used to replicate the step-by-step tunnel construction process. In the next chapters, a comparative study is performed to demonstrate the model capabilities and the effect of the different parameters/interactions on the structural forces in lining. In addition, the description of lining kinematics is enriched by including the lining joints by means of contact.
Chapter 4

Evaluation of Lining Response using ekate Model

The simulation model ekate serves as a predictive tool for the assessment of tunneling induced settlements, loading on lining as well as the structural forces in the lining for the process oriented tunnel advance. The superiority of this model is that it includes most of the relevant interactions involved in mechanized tunneling process. For this reason, the simulation model is used to study the effect of the design related parameters and the shield operation parameters. In this chapter, a systematic parametric study is presented focusing on the influence of the geological conditions, the annular gap grouting and the shield design parameters for shallow soft ground conditions under the water table. The results provide a better insight into the effect of these parameters in order to investigate, if, and to which extent a precise and detailed modeling of the mechanized tunneling process plays a role in regards to the tunnel lining analysis.

4.1 Model Description

The analysis depicts a straight tunnel with an overburden depth of 1.7D, following mechanized tunneling excavation in soft soil. The dimensions and the material properties used in the model and their respective variations are summarized in table 4.1, in which, bold font indicates to the reference values. The ground is assumed to be silty soil in full saturation, with an elastic modulus $E = 50$ MPa, Poisson’s ratio $\nu = 0.3$, effective cohesion $c^\prime = 30$ kPa, effective friction angle $\phi^\prime = 30$ $^\circ$, lateral earth pressure coefficient $K_0 = 0.42$ and a permeability $k_w = 10^{-6}$ m/s.

Tunneling is assumed to be performed by a shield machine with front and rear diameters of 9.50 m and 9.48 m respectively. The shield is steered with twenty-eight hydraulic jacks that are equally distributed and their elongations are obtained from the steering algorithm (see section 2.4.3). The
concrete lining is represented by a continuous tube with an external diameter of 9.20 m. At this level, detailed inclusion of lining joints is not considered. In this chapter, the loads acting on the lining and the structural forces (i.e. normal forces and bending moments) in the lining are investigated. The acting loads are extrapolated from the assigned tying conditions between the lining rings and the grouting elements, see section 5.1.1, and the structural forces are determined as discussed in appendix A. The mechanical properties and permeability of the annular gap grouting are defined as time-dependent parameters in order to account for the grout hydration process, see section 2.4.1. The dimensions and the finite element discretization of the investigated tunnel section are shown in figure 4.1.

Figure 4.1: Numerical investigation of the effect of design related parameters; dimensions used in the simulation models (left) and finite element mesh with the detailed model components and the shield geometrical parameters (i.e. overcut, conicity and length) (right)

In what follows, a parametric study is performed while focusing on the design related parameters and the shield’s operating parameters, in which, the investigated parameters can be characterized as:

I. Geological conditions:
   I.1. Type of soil (i.e. cohesive and granular soils depicted by DRUCKER-PRAGER-model, and the linear elastic assumption)
   I.2. Coefficient of lateral earth pressure
   I.3. Level of ground water table

II. Shield design parameters:
   II.1. Shield overcut
   II.2. Shield conicity
   II.3. Friction coefficient between the shield and the excavated soil

III. Annular gap grouting:
   III.1. Grouting pressure
   III.2. Time dependent properties of the grouting material

IV. Advancement along curved alignments
### 4.1. MODEL DESCRIPTION

#### Parameters for the ground

<table>
<thead>
<tr>
<th>Property</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s modulus</td>
<td>MPa</td>
<td>50</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>[</td>
<td>0.3</td>
</tr>
<tr>
<td>Density</td>
<td>kg/m³</td>
<td>2000</td>
</tr>
<tr>
<td>Effective cohesion</td>
<td>kPa</td>
<td>0.01 / 30</td>
</tr>
<tr>
<td>Effective friction angle</td>
<td>degree</td>
<td>30</td>
</tr>
<tr>
<td>Lateral earth pressure coefficient</td>
<td>[</td>
<td>0.35 / 0.42 / 0.5 / 0.67</td>
</tr>
<tr>
<td>Permeability of water</td>
<td>m/s</td>
<td>10⁻⁶</td>
</tr>
<tr>
<td>Level of ground water</td>
<td>m</td>
<td>0.0 / -3.0 / -7.0</td>
</tr>
</tbody>
</table>

#### Parameters for the concrete lining

<table>
<thead>
<tr>
<th>Property</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s modulus</td>
<td>GPa</td>
<td>30</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>[</td>
<td>0.2</td>
</tr>
<tr>
<td>Density</td>
<td>kg/m³</td>
<td>2500</td>
</tr>
<tr>
<td>Width</td>
<td>m</td>
<td>1.50</td>
</tr>
<tr>
<td>Thickness</td>
<td>cm</td>
<td>0.45</td>
</tr>
<tr>
<td>Radius</td>
<td>m</td>
<td>9.20</td>
</tr>
</tbody>
</table>

#### Parameters for the shield machine

<table>
<thead>
<tr>
<th>Property</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s modulus</td>
<td>GPa</td>
<td>210</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>[</td>
<td>0.3</td>
</tr>
<tr>
<td>Density</td>
<td>kg/m³</td>
<td>7600</td>
</tr>
<tr>
<td>Front diameter</td>
<td>m</td>
<td>9.51 / 9.50 / 9.49</td>
</tr>
<tr>
<td>Tail diameter</td>
<td>m</td>
<td>9.49 / 9.48 / 9.47</td>
</tr>
<tr>
<td>Length</td>
<td>m</td>
<td>9.00</td>
</tr>
<tr>
<td>Weight</td>
<td>ton</td>
<td>600</td>
</tr>
<tr>
<td>Friction coefficient</td>
<td>[</td>
<td>0.0 / 0.25 / 0.50</td>
</tr>
</tbody>
</table>

#### Parameters for the grouting mortar

<table>
<thead>
<tr>
<th>Property</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s modulus</td>
<td>MPa</td>
<td>300</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>[</td>
<td>0.3</td>
</tr>
<tr>
<td>Density</td>
<td>kg/m³</td>
<td>2000</td>
</tr>
<tr>
<td>Stiffness ratio</td>
<td>[</td>
<td>0.65</td>
</tr>
<tr>
<td>Hydration parameter $t_E$</td>
<td>h</td>
<td>6</td>
</tr>
<tr>
<td>Hydration parameter $\Delta t_E$</td>
<td>h</td>
<td>4</td>
</tr>
<tr>
<td>Initial permeability</td>
<td>m/s</td>
<td>10⁻⁵</td>
</tr>
<tr>
<td>Final permeability</td>
<td>m/s</td>
<td>10⁻⁸</td>
</tr>
</tbody>
</table>

#### Parameters for the advancement process

<table>
<thead>
<tr>
<th>Property</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shield advance rate</td>
<td>m/h</td>
<td>2.0</td>
</tr>
<tr>
<td>Ring constriction time</td>
<td>h</td>
<td>0.75</td>
</tr>
<tr>
<td>Face support pressure</td>
<td>kPa</td>
<td>220</td>
</tr>
<tr>
<td>Face support pressure gradient</td>
<td>kPa/m</td>
<td>12</td>
</tr>
<tr>
<td>Grouting pressure</td>
<td>kPa</td>
<td>200 / 230 / 260</td>
</tr>
<tr>
<td>Grouting pressure gradient</td>
<td>kPa/m</td>
<td>10</td>
</tr>
</tbody>
</table>

Table 4.1: Design related parameters and their variations for the parametric investigation
CHAPTER 4. EVALUATION OF LINING RESPONSE USING EKATE MODEL

4.2 Geological Conditions

4.2.1 Soil Material Behavior

In order to determine the influence of the ground material properties, two different constitutive laws are used in the analysis. DRUCKER-PRAGER constitutive law is adopted to describe the non-linear soil behavior for a cohesive soil (silty soil with an effective cohesion $c^\prime = 30$ kPa and an effective friction angle $\phi^\prime = 30^\circ$) and a granular soil (sandy soil with an effective cohesion $c^\prime = 0.01$ kPa and an effective friction angle $\phi^\prime = 30^\circ$). In addition, the study includes the assumption of a linear elastic soil response. This assumption is considered since it replicates to some extend the spring stiffness representing the soil in the bedded beam models.

![Graphs showing surface settlement, radial loading, normal forces, and bending moments.](image)

**Figure 4.2:** (a) Computed surface settlements at the monitoring point during shield advance, (b) radial loading on lining, (c) normal forces and (d) bending moments at the monitoring section at the steady state for different soil material response.
It is shown in figure 4.2-a, that the nonlinear material response has a noticeable effect on the predicted ground deformation. The maximum predicted surface settlement for a linear material assumption is 3.65 mm that rises for nonlinear materials up to 4.3 mm for \( c = 30 \) kPa and 5.12 mm for \( c = 0.01 \) kPa. Such increase is attributed to the presence of plastic deformation which is governed, in this case, by the soil cohesion (see figure 4.3). It is clear that larger deformations, caused due to the material nonlinearity, affects the mechanical behavior of the soil. As a consequence, plastic deformations limit the soil self support, that leads to higher loads on the lining, see figure 4.2-b and table 4.2. In addition, a decrease in the ring normal forces is predicted for stiffer material with higher cohesion or with linear elastic assumption (figure 4.2-c). The maximum levels of normal forces are predicted at the springline as -922 kN/m, -985 kN/m and -1012 kN/m for linear elastic, cohesive and granular soil types, respectively. As well, a change in the level of bending moments is captured, in particular between the linear and the nonlinear assumptions (figure 4.2-d). Table 4.3 shows the maximum predicted values of structural forces and its percentage with respect to the case of linear material assumption.

![Figure 4.3: Volumetric plastic deformations in the soil at the crown, springline and invert of the tunnel at the steady state](image)

<table>
<thead>
<tr>
<th>Material behavior</th>
<th>( S_{max} ) [mm]</th>
<th>Radial loading on lining at crown [kN/m²]</th>
<th>Radial loading on lining at springline [kN/m²]</th>
<th>Radial loading on lining at invert [kN/m²]</th>
<th>( \Delta P_{val}/\Delta P_{hal} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Linear elastic</td>
<td>3.65</td>
<td>187</td>
<td>173</td>
<td>225</td>
<td>1.19</td>
</tr>
<tr>
<td>Cohesive soil</td>
<td>4.3</td>
<td>200</td>
<td>187</td>
<td>237</td>
<td>1.17</td>
</tr>
<tr>
<td>Granular soil</td>
<td>5.12</td>
<td>205</td>
<td>193</td>
<td>243</td>
<td>1.16</td>
</tr>
</tbody>
</table>

Table 4.2: Maximum predicted surface settlements and the radial loading on the lining at different locations with respect to different material behavior

<table>
<thead>
<tr>
<th>Material behavior</th>
<th>( N_{max} ) [kN/m]</th>
<th>( M_{max} ) [kNm/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Linear elastic</td>
<td>-922</td>
<td>-166/-170</td>
</tr>
<tr>
<td>Cohesive soil</td>
<td>-985 (107)</td>
<td>150/-158 (90/93)</td>
</tr>
<tr>
<td>Granular soil</td>
<td>-1013 (110)</td>
<td>147/-156 (89/92)</td>
</tr>
</tbody>
</table>

Table 4.3: Maximum predicted structural forces and their deviation for different material behavior
4.2.2 Coefficient of Lateral Earth Pressure

The influence of the lateral confinement of soil is investigated by performing the analysis with four different coefficients of lateral earth pressure ($K_o = 0.35$, 0.42, 0.5 and 0.67). From figure 4.4-a, the reduction in lateral earth pressure coefficient of the soil is accompanied with an increase in the surface settlements. The maximum predicted surface settlements for $K_o$ values of 0.67, 0.50, 0.42 and 0.37 are 1.76 mm, 3.46 mm, 4.30 mm and 5.25 mm, respectively. This is justified as the reduction of lateral pressures limits the arching effect in the soil. On the other hand, the predicted radial loading on lining at the crown and invert is not significantly influenced since the investigated cases have the same overburden. Yet, the loading at springline decreases which agrees well with the reduction of lateral pressure, see figure 4.4-b and table 4.4.

Figure 4.4-c and d shows the normal forces and bending moments distributions in the lining. The latter can be interpreted by the comparison with the corresponding loads; the increase of lateral pressure at the springline causes an analogous increase in the normal force levels at the crown and the invert, yet, the maximum normal force levels at the springline are almost unchanged (an increase of 4% at the limit case with $K_o = 0.67$). With respect to bending moments, the change of $K_o$ levels leads to a change in the ratio between vertical and horizontal loading (see $\Delta P_{val}/\Delta P_{hal}$ in table 4.4) that in return causes a change in the maximum values of bending moments, see table 4.5. The maximum moments are 86/-93 kNm/m, 130/-138 kNm/m, 150/-158 kNm/m and 166/-173 kNm/m for $K_o$ values of 0.67, 0.50, 0.42 and 0.37, respectively which correspond to approximately 50% difference. This shows that the predicted maximum moments are highly dependent on the adopted lateral confinement pressure.

<table>
<thead>
<tr>
<th>$K_o$</th>
<th>$S_{max}$ [mm]</th>
<th>Radial loading on lining [kN/m$^2$]</th>
<th>$\Delta P_{val}/\Delta P_{hal}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>at crown at springline at invert</td>
<td></td>
</tr>
<tr>
<td>0.67</td>
<td>1.76</td>
<td>204 205 244</td>
<td>1.09</td>
</tr>
<tr>
<td>0.50</td>
<td>3.46</td>
<td>201 192 239</td>
<td>1.15</td>
</tr>
<tr>
<td>0.42</td>
<td>4.3</td>
<td>200 187 237</td>
<td>1.17</td>
</tr>
<tr>
<td>0.35</td>
<td>5.25</td>
<td>198 183 236</td>
<td>1.19</td>
</tr>
</tbody>
</table>

Table 4.4: Maximum predicted surface settlements and the radial loading on the lining at different locations with respect to different coefficients of lateral earth pressure

<table>
<thead>
<tr>
<th>$K_o$</th>
<th>$N_{max}$ [kN/m]</th>
<th>$M_{max}$ [kNm/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(% )</td>
<td>(%)</td>
</tr>
<tr>
<td>0.67</td>
<td>-1022 (104)</td>
<td>86/-93 (52/54)</td>
</tr>
<tr>
<td>0.50</td>
<td>-994 (102)</td>
<td>130/-138 (78/80)</td>
</tr>
<tr>
<td>0.42</td>
<td>-985 (101)</td>
<td>150/-158 (90/91)</td>
</tr>
<tr>
<td>0.35</td>
<td>-979</td>
<td>166/-173</td>
</tr>
</tbody>
</table>

Table 4.5: Maximum predicted structural forces and their deviation for different coefficients of lateral earth pressure
4.2. GEOLOGICAL CONDITIONS

4.2.3 Level of Ground Water Table

Three different levels of ground water table are investigated in this subsection (0.0 m, -3.0 m and -7.0 m under the ground surface as shown in figure 4.1-left). In the comparative analyses, soil density above water table is assumed to be in full saturation ($\gamma_{sat} = 2000 \text{ kg/m}^3$) to maintain the same amount of overburden pressure at the tunnel crown for all the investigated cases. In addition, the support pressures, i.e. the face pressure and the grouting pressure, are adjusted with respect to the situated water level. The face support pressure and the grouting pressures at the center of the tunnel are adopted as 220/230 kPa, 190/200 kPa and 150/160 kPa for water levels 0.0 m, -3.0 m and -7.0 m, respectively with a linear increase with depth. The adopted pressures preserve the same amount of excess pore pressures at the shield tail (i.e. 30 kPa) in all cases.
The drop in water level does not reduce the buoyancy forces during tunneling as long as the lining shell is under the water level. Instead, such drop significantly reduces the lateral confinement (noting that $K_{\text{water}}$ is equal to unity) and as a consequence, the surface settlements significantly increases up to 6.04 mm and 7.46 mm for ground water levels at -3.0 m and -7.0 m, respectively (figure 4.5-a and table 4.6). The level of hydrostatic state of stresses around the tunnel follows the height of water level; a decrease in the water level results in a reduced loading on the lining as shown in figure 4.5-b. This is more noticeable at the springline following the fact that reduction in pore pressure is more than the increase of lateral effective stresses. This is can be seen in table 4.6, in which the vertical to lateral pressure ratio increases from 1.17 for water level of 0.0 m to 1.28 when considering a lower water level of -7.0 m.

Figure 4.5-c and d shows the distribution of the normal forces and the bending moments at the steady state for the three investigated levels of water. As expected, the normal forces proportionally decrease with the decreasing water levels. The computed maximum normal forces are -985 kN/m, -921 kN/m and -844 kN/m for water levels of 0.0 m, -3.0 m and -7.0 m, respectively. Also, as the vertical to lateral pressure ratio increases, an increase in the maximum bending moments is observed. The maximum moments are increasing from 150/-158 kNm/m up to 181/-188 kNm/m and 212/-218 kNm/m respectively, with decreasing height of water. This corresponds to an increase of 40% in bending moments and decrease of 14% in normal forces, see table 4.7.

<table>
<thead>
<tr>
<th>G.W.L</th>
<th>$S_{\text{max}}$ [mm]</th>
<th>Radial loading on lining [kN/m²]</th>
<th>$\Delta P_{\text{val}}/\Delta P_{\text{hal}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>4.3</td>
<td>200 at crown 187 at springline 237 at invert</td>
<td>1.17</td>
</tr>
<tr>
<td>-3.0</td>
<td>6.04</td>
<td>187 at springline 168 at invert 225</td>
<td>1.23</td>
</tr>
<tr>
<td>-7.0</td>
<td>7.46</td>
<td>171 at springline 147 at invert 204</td>
<td>1.28</td>
</tr>
</tbody>
</table>

Table 4.6: Maximum predicted surface settlements and the radial loading on the lining at different locations with respect to different levels of ground water table

<table>
<thead>
<tr>
<th>G.W.L</th>
<th>$N_{\text{max}}$ [kN/m]</th>
<th>$M_{\text{max}}$ [kNm/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>-985</td>
<td>150/-158</td>
</tr>
<tr>
<td>-3.0</td>
<td>-921 (94)</td>
<td>181/-188 (121/119)</td>
</tr>
<tr>
<td>-7.0</td>
<td>-844 (86)</td>
<td>212/-218 (141/138)</td>
</tr>
</tbody>
</table>

Table 4.7: Maximum predicted structural forces and their deviation for different levels of ground water table
4.3 Shield Design Parameters

In the simulation model, the shield is modeled as a deformable body that is in contact with the excavated ground. In the pertinent literature, the shield-soil interaction is usually represented via interface elements or by simple nodal connectivity (i.e. contact with the ground is usually ignored). In addition, shield advancement is achieved via reactivation of elements or via controlled deformations towards its current position, see section 2.3. In ekate, unlike other simulation models, the shield geometry is depicted using the volume elements representing the main structural components, loading is explicitly applied as uniform load and shield advancement is achieved by an automatic steering algorithm, see section 2.4.1 and 2.4.3. In this section, the effect of shield geometry and the influence of friction between the shield and the soil are investigated.
4.3.1 Shield Overcut and Conicity

In tunneling practice, the shield’s front diameter is relatively smaller than the cutting wheel, in addition, tapering along shield length is used. This facilitates the construction process by preventing the shield from getting stuck and assisting the steering of the machine in particular along curves. Herein, the influence of shield geometry (i.e. front and tail diameters) is investigated. In figure 4.6, the solid lines represent different overcut values (1 cm, 2 cm and 3 cm) with a conicity of 2 cm, while, dash lines refer to different conicities (1 cm and 3 cm). The predicted results in figure 4.6 indicate that the shield’s geometry has a significant effect on the predicted settlements as well the structural response of the lining.

Surface settlements increase with higher overcut and/or higher conicity (i.e. larger deformations).
are attributed to the higher ground relaxation as a result of increasing overcut and/or conicity). The predicted surface settlements are 2.43 mm, 4.30 mm and 6.86 mm for an overcut of 1 cm, 2 cm and 3 cm, respectively with a conicity of 2 cm, see table 4.8. Larger overcut and larger conicity causes more stress release in the soil and hence, lower loads on lining and lower normal forces are predicted (figure 4.6-b and c). It should be noted that the distribution of loads along the circumference is strongly dependent on the shield geometry. For lower overcut and conicity indicated by the green curves in figure 4.6, the pressure at the springline is almost equal to the pressure at the crown, see also table 4.8. With higher overcut and conicity, more specifically for the case of overcut equals 2 cm and a conicity equals 2 cm (blue curve), the results generally show a reduction in radial pressures in particular at the springline. With further increase in overcut or conicity (red curves), a further decrease in the radial pressure is noticed all around the tunnel lining.

With respect to the structural forces in tunnel lining, maximum normal forces are obtained with lower overcut and conicity (green curves in figure 4.6-c). The highest moment is obtained for an overcut of 2 cm and a conicity of 2 cm (blue curve in figure 4.6-d), which matches with the predicted load ovalization ratio ($\Delta P_{val}/\Delta P_{hal}$) as shown in table 4.8. Additionally, figure 4.6-c shows a direct relation between the shield geometry and the predicted normal forces (i.e. larger is the overcut/conicity, lower are the normal forces). On the contrary, the maximum predicted bending moments have a nonlinear relationship with the shield geometry as shown in figure 4.6-d. Maximum moments initially increase with an increasing overcut/conicity (green to blue curves), but as the overcut/conicity is increased further (blue to red curves), a decrease in maximum moment values is observed. As shown in table 4.9, approximately 20% difference in the maximum values of normal forces and bending moments are obtained for the investigated scenarios.

### Table 4.8: Maximum predicted surface settlements and the radial loading on the lining at different locations with respect to different shield geometries

<table>
<thead>
<tr>
<th>$D_{front}$ [m]</th>
<th>$D_{tail}$ [m]</th>
<th>$S_{max}$ [mm] at crown</th>
<th>Radial loading on lining [kN/m²] at springline at invert</th>
<th>$\Delta P_{val}/\Delta P_{hal}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.50</td>
<td>9.49</td>
<td>2.39</td>
<td>217 210 248</td>
<td>1.10</td>
</tr>
<tr>
<td>9.51</td>
<td>9.49</td>
<td>2.43</td>
<td>213 209 243</td>
<td>1.09</td>
</tr>
<tr>
<td>9.50</td>
<td>9.48</td>
<td>4.30</td>
<td>200 187 237</td>
<td>1.17</td>
</tr>
<tr>
<td>9.50</td>
<td>9.47</td>
<td>6.54</td>
<td>175 168 209</td>
<td>1.14</td>
</tr>
<tr>
<td>9.49</td>
<td>9.47</td>
<td>6.86</td>
<td>162 156 197</td>
<td>1.15</td>
</tr>
</tbody>
</table>

### Table 4.9: Maximum predicted structural forces and their deviation for different shield geometries

<table>
<thead>
<tr>
<th>$D_{front}$ [m]</th>
<th>$D_{tail}$ [m]</th>
<th>$N_{max}$ [kN/m] (%)</th>
<th>$M_{max}$ [kNm/m] (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.50</td>
<td>9.49</td>
<td>-1075 (109)</td>
<td>130/-135 (87/85)</td>
</tr>
<tr>
<td>9.51</td>
<td>9.49</td>
<td>-1074 (109)</td>
<td>130/-135 (87/85)</td>
</tr>
<tr>
<td>9.50</td>
<td>9.48</td>
<td>-985</td>
<td>150/-158</td>
</tr>
<tr>
<td>9.50</td>
<td>9.47</td>
<td>-887 (90)</td>
<td>134/-141 (89/89)</td>
</tr>
<tr>
<td>9.49</td>
<td>9.47</td>
<td>-799 (81)</td>
<td>116/-122 (77/77)</td>
</tr>
</tbody>
</table>
4.3.2 Shield Friction with the Excavated Soil

Three different values of friction coefficient between the soil and shield skin are used in the simulations (0.0, 0.25 and 0.50). As shown in figure 4.7, the increase of frictional forces causes more heave at the tunnel face and more surface settlements at a distance from the shield tail. This can be related to the development of frictional forces at the excavated boundary along the driving direction, which tend to provides additional pressure ahead from the tunnel face. Moreover, the distribution of frictional forces (more forces are predicted at the bottom) cause a change in soil lateral deformation that results in higher settlements, see figure 4.8. According to figure 4.7-b, c and d, only slight changes in radial loading and normal forces are predicted, and some reduction of bending moments is captured due to the change of radial loads at the springline.

![Figure 4.7](image-url)

**Figure 4.7:** (a) Computed surface settlements at the monitoring point during shield advance, (b) radial loading on lining, (c) normal forces and (d) bending moments at the monitoring section, at a steady state value, for different friction coefficient between the shield skin and the excavated ground.
4.4 Annular Gap Grouting

4.4.1 Grouting Pressure

Three different levels of grouting pressure are considered in the analysis: 200 kPa, 230 kPa and 260 kPa, which are applied at the tunnel axis with a gradient of 10 kPa/m along depth. It is shown in figure 4.9-a that the predicted surface settlements decrease with the increasing level of grouting pressure. It should be noted that the grouting pressure has a relatively large influence ahead of the shield. Such behavior is caused by the model assumptions. According to NAGEL (2009), the model overestimates the pore water pressure while in practice, the grout infiltrates into the soil in such a manner that limits the excess pressure in ground water due to grouting. According to figure 4.9-b, the higher the grouting pressure, the higher is the loading on lining. In addition, table 4.10 shows that slight reduction in the ratio between vertical and horizontal pressures is observed with increasing grouting pressure; this is justified as the increased grouting pressure generally raises the level of hydrostatic pressure at the shield tail. Accordingly, the normal forces slightly increase and the bending moments slightly decrease, see figure 4.9-c, d and table 4.11

<table>
<thead>
<tr>
<th>Grout pressure [kPa]</th>
<th>$S_{max}$ [mm]</th>
<th>Radial loading on lining [kN/m$^2$]</th>
<th>$\Delta P_{val}/\Delta P_{hal}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>at crown</td>
<td>at springline</td>
<td>at invert</td>
</tr>
<tr>
<td>200</td>
<td>5.29</td>
<td>196</td>
<td>183</td>
</tr>
<tr>
<td>230</td>
<td>4.30</td>
<td>200</td>
<td>187</td>
</tr>
<tr>
<td>260</td>
<td>2.97</td>
<td>206</td>
<td>194</td>
</tr>
</tbody>
</table>

Table 4.10: Maximum predicted surface settlements and the radial loading on the lining at different locations with respect to different levels of annular gap grouting pressure
Figure 4.9: (a) Computed surface settlements at the monitoring point during shield advance, (b) radial loading on lining, (c) normal forces and (d) bending moments at the monitoring section at the steady state for different levels of annular gap grouting pressure

<table>
<thead>
<tr>
<th>Grout pressure [kPa]</th>
<th>$N_{max}$ [kN/m]</th>
<th>(%)</th>
<th>$M_{max}$ [kNm/m]</th>
<th>(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>200</td>
<td>-967</td>
<td>–</td>
<td>154/-161</td>
<td>–</td>
</tr>
<tr>
<td>230</td>
<td>-985 (102)</td>
<td></td>
<td>150/-158 (97/98)</td>
<td></td>
</tr>
<tr>
<td>260</td>
<td>-1012 (105)</td>
<td></td>
<td>156/-153 (101/95)</td>
<td></td>
</tr>
</tbody>
</table>

Table 4.11: Maximum predicted structural forces and their deviation for different levels of annular gap grouting pressure

4.4.2 Time Dependent Properties of Grouting Material

In this subsection, the influence of the characteristics of grouting material is investigated considering two scenarios; first, the grouting is assumed to be active-grout with a time dependent properties and
permeability as described in table 4.1 and compared with an inactive-grout with a constant elastic modulus of 50 MPa, Poisson’s ratio of 0.3 and a permeability of $10^{-6}$ m/s.

The predicted surface settlements in figure 4.10-a show that grout characteristics play a role and lead to different responses. The maximum settlements are 4.30 mm and 5.68 mm for the case of time-dependent properties and constant properties, respectively. In addition, considerably different distributions of loading on the lining are predicted, see figure 4.10-b. With constant grout properties, the radial loads from the crown to the springline are relatively equal and increasing at the invert, see table 4.12. The constant elastic modulus of the inactive-grout is relatively higher than the initial stiffness of the active grout directly at the shield tail. As explained in (NINIĆ AND MESCHKE 2017), such condition does not allow for full pressurization of the annular gap grouting. As a consequence, higher normal forces are predicted with the time dependent grout properties (figure 4.10-c), as well
as, higher bending moments are predicted (figure 4.10-d) with approximately 20% difference in the maximum moment values between the two cases, see table 4.13.

<table>
<thead>
<tr>
<th>Grout properties</th>
<th>$S_{max}$ [mm]</th>
<th>Radial loading on lining [kN/m²]</th>
<th>$\Delta P_{val}/\Delta P_{hal}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Time-dependent</td>
<td>4.30</td>
<td>200</td>
<td>187</td>
</tr>
<tr>
<td>Constant properties</td>
<td>5.68</td>
<td>179</td>
<td>237</td>
</tr>
</tbody>
</table>

Table 4.12: Maximum predicted surface settlements and the radial loading on the lining at different locations with respect to two different grout material models

<table>
<thead>
<tr>
<th>Grout properties</th>
<th>$N_{max}$ [kN/m] (%)</th>
<th>$M_{max}$ [kNm/m] (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Time-dependent</td>
<td>-985 (95)</td>
<td>150/-158</td>
</tr>
<tr>
<td>Constant properties</td>
<td>-933 (95)</td>
<td>125/-127 (83/80)</td>
</tr>
</tbody>
</table>

Table 4.13: Maximum predicted structural forces and their deviation for two different grout material models

### 4.5 Advancement along Curved Alignments

The aim of this section is to investigate the influence of the driven tunnel path on the distribution of the resulted loads on the lining. For this purpose, the investigated tunnel model is analyzed along a short straight path, then followed by a horizontal leftward curve with a radius of 500 m as shown in figure 4.11. This model (130 m length and 120 m width) has the same properties and constructional conditions similar to the previously investigated straight alignment.

![Figure 4.11: Numerical investigation of the effect of the driven tunnel path; finite element mesh for a curved alignment with half of the ground domain and a detailed representation of the tapered lining rings and the shield machine.](image-url)
Figure 4.12: (a) Computed radial loading on the lining, (b) normal forces and (c) bending moments at the monitoring section at the steady state during shield advance along straight and curved alignments.
Figure 4.12 compares the loading on lining as well as the resulted structural forces in the lining as predicted from the FE-models for both straight and curved alignments. It has been indicated in section 4.3.1 that the shield-soil interaction governs the distribution of the resulted forces on the lining and in turn the lining responses. In curved alignment, the shield-soil interaction results not only from the shield tilting forward due to its own weight but also from its orientation upon driving along the curve (ALSAHLY 2017), as shown in the schematic illustration in figure 4.13. Employing the steering algorithm, the shield advances along the prescribed path which in turn causes diverse contact pressures at the springline at the shield tail. For the driven leftward curve, more ground support is provided at the right side and more relaxation occurs at the left side. This explains the predicted distribution of the loading on lining as shown in figure 4.12-a. The maximum loading on lining for the case of curved alignment is predicted at the lower right side of the lining. At the springline, the radial pressures are \( \approx 252 \text{ kPa} \) and \( 169 \text{ kPa} \) at the right and left sides, respectively. The respective value predicted from the straight alignment model is 187 kPa, this corresponds to an increase of 35\% at the right side when advancing along curve (\( R = 500 \text{ m} \)). Figure 4.12-b and c show the varying normal forces and bending moments with respect to the driven path. Along the curved path, the predicted normal forces tend to increase at the right springline and decrease at the left springline. Similarly, the bending moments increase at the right side by 18\% and significantly decrease at the left side by 59\%. This can be explained as a result of ground relaxation and shield-soil interaction during the simulation of the excavation and the advancement process along leftward curved alignment. Here, the shield tail at the right side provides more ground support, which in turn sustain more loads and vise-versa at the left side.

Figure 4.13: Schematic illustration of the shield orientation upon driving along curved alignment (top view)

### 4.6 Evaluation of Acting Loads and their Comparison with In-situ Loading Assumption

The range of the predicted radial and tangential loading on the lining at the steady state, as obtained form all the investigated scenarios, is discussed in this section. In order to compare the range of the predicted loading on lining with the in-situ loading assumption, as proposed by ITA WORKING
4.6. EVALUATION OF ACTING LOADS AND THEIR COMPARISON WITH IN-SITU LOADING ASSUMPTION

Group No. 2 (2000), the variations in lateral earth pressure coefficient and ground water level, and the effect of curved alignment are excluded herein. As can be seen in figure 4.14, the final loads acting on the lining do not converge to a unique value, instead, large variations are obtained within the range of the adopted parameters. Generally, the highest radial loads are developed at the invert of the tunnel lining and the lowest radial pressures tend to develop at the springline. The values of tangential pressures and their respective differences are negligible when compared to radial loads, therefore, they do not have pronounced effects. The upper limits of the predicted radial loading are 217 kPa, 210 kPa and 248 kPa at the crown, the springline and the invert, respectively. These values are obtained for the case with the largest shield diameter with lowest shield conicity and lowest overcut. For the same excavation diameter with larger conicity and overcut, i.e. overcut = 3 cm and a conicity = 2 cm, more ground relaxation occurs along the shield which leads to the lower limits. The later are 162 kPa, 155 kPa and 197 kPa at the crown, the springline and the invert, respectively. From this numerical model with the adopted parameters in hand, approximately 54 kPa difference between the upper and lower limits is obtained which is approximately equal to 25% of the predicted values. It should be pointed out that the range of the predicted loadings on lining shown in figure 4.14 is obtained from the variation of the 3D model parameters in particular, the soil non linearity, shield diameter and supporting pressures. In the bedded lining model, these parameters can not included, hence, their influence on tunnel lining can not be evaluated with such simplified modeling approach, see section 3.3.2.

Finally, the predicted range of the loading on lining is compared with an in-situ loading assumption, as proposed by ITA Working Group No. 2 (2000). In this assumption, the vertical earth pressures are applied as uniform loads, the lateral earth pressures linearly vary with depth and water pressure is applied in radial direction, see appendix C.3 and figure C.4 for further details. The transformation of radial and tangential loads to vertical and horizontal directions are presented in appendix B. The vertical and lateral loads are depicted in figure 4.15.

Figure 4.14: Computed range of loading on lining from the simulation model with various model parameters (gray shaded area): loading in radial direction (left) and loading in tangential direction (right), noting that the variations in the coefficient of lateral earth pressure and the ground water level are excluded.
Figure 4.15: Computed range of loading on the lining in vertical and horizontal directions from the simulation model (gray shaded area) in comparison with the loading assumption according to ITA WORKING GROUP No. 2 (2000) (solid line)
Chapter 5

Representation of Joint Behavior Using Contact

For the design of segmental tunnel linings, precise structural models are needed, as the segmentation imbues non-trivial kinematics of the lining system. In this context, a novel technique for modeling segmental tunnel linings is proposed. The segments of the lining ring are explicitly modeled as separate bodies, and the interactions between segments at the longitudinal and ring joints are modeled by means of a penalty-based, surface-to-surface frictional contact algorithm. In order to examine three-dimensional stress distribution in the segmental concrete lining under realistic, time-dependent, process loading, the lining model is integrated into the process oriented finite element simulation "ekate".

5.1 Segment-wise Lining Installation in ekate

Shield tunnel lining is constructed by the assembly of segments into a complete ring. The simulation of segmental lining model including the joints is similar to the continuous model to some extent. The main difference is the assignment of the contact interactions at the joints between the segments. The generation of the segmental lining model starts with the consideration of a single ring as shown in figure 5.1. The ring model, generated by GiD (MELENDO ET AL. 2015), consists of volume elements which represent different segments in which the boundary surfaces of each segment are separately defined. Figure 5.1 shows a ring that consists of 7 segments with equal size including bolts and dowels on joints. Each longitudinal joint contains two bolts at the center line, while each segment has two shear dowels in the ring joint. In total, 14 bolts and 14 dowels are used in each ring. The exact joint geometry is not described in this model since the global structural response is the main interest of this study. As such, the effect of the rubber sealing gasket is not explicitly considered. In addition, only joints with flat contact surfaces will be addressed.
The contact interactions between the segments in one ring and between consecutive rings are shown in figure 5.2. The contact algorithm presented in section 5.2 is used to characterize the response of joints; this requires the definition of master and slave surfaces for the possible contact surfaces. Since, the complete lining model consists of a large number of joints, there will consequently exist a large number of master and slave contact surfaces. Therefore, each pair of contacting surfaces is associated with a distinct contact index, as indicated in figure 5.2, to speed up the search algorithm.
Beam elements with elastic material properties are used to represent the dowels and bolts in the joints. The geometrical properties of the beam is defined according to the corresponding diameter, length and type, as shown in figure 5.3. Pre-stressing in the bolts can be considered by applying a certain pre-stressing force for the corresponding element. In finite element formulation, the internal force vector for the beam element with pre-stressing is defined as:

\[ \mathbf{R}_{int} = \mathbf{K} \cdot \mathbf{u} + \mathbf{F}_{prestress} \] (5.1)

The assigned properties to the beam elements describe the desired structural behavior of the elements. For the simulation of shear dowels, only the shear stiffness is required while the axial stiffness can be omitted by setting \( A_{axial} = 0 \). Bolts are mainly simulated by considering axial stiffness and the pre-stressing force if required, while, the flexural stiffness can be either considered or ignored. Generally, the dowels/bolts are assumed to act at the center line of the joints and therefore it is not expected that they contribute significantly to the overall flexural stiffness of the lining ring.

![Figure 5.3: Representation of bolts and dowels in segmental lining joints](image)

The physical interaction between the segments and the dowels/bolts is accounted for by using the node to volume tying, see figure 5.3. The nodes at the ends of each bolt are embedded in their corresponding volume elements. In the finite element code Kratos, the EmbeddedPointLagrangeTyingUtility is used for setting these tying constraints. Within the simulation script, a function, InitializeEmbeddedPointLagrangeTying, sets the tying condition between the end node of the beam \( X_i \) and the volume element containing that point. First, the local coordinates \( \xi(X_i) \) at the point location inside the volume element are determined. Then, the condition ties the displacements between the point and its projection inside the volume elements using the Lagrange multiplier as:

\[ (\mathbf{u}_{node} - \mathbf{u}_{vol}(\xi(X_i))) \cdot \lambda = 0, \] (5.2)

and the tying condition is added to the system of equation as:

\[
\begin{bmatrix}
\mathbf{K}_{node} & 0 & 1 \\
0 & \mathbf{K}_{node} & -1 \\
1 & -1 & 0
\end{bmatrix}
\begin{bmatrix}
\mathbf{u}_{node} \\
\mathbf{u}_{vol.} \\
\lambda
\end{bmatrix}
= 
\begin{bmatrix}
\mathbf{R}_{node} \\
\mathbf{R}_{vol.} \\
0
\end{bmatrix}
\] (5.3)

The full description of the numerical model requires the definition of other properties such as material, activation levels, etc., as well. A python script is created to automatize the model generation. First, it imports the geometry of the user defined segmental ring. Then, the segmental ring is
placed in its location and rotated according to the required staggered joint pattern. The properties and boundary conditions associated with the lining ring are assigned. The desired joint pattern can be generated independently of the finite element discretization of the ground. Figure 5.4 shows a staggered and aligned configurations of lining joints.

![Staggered joint pattern](image1)
![Aligned joint pattern](image2)

**Figure 5.4:** Different joint arrangements in segmental tunnel lining model

It should be noted that the staggered configurations of longitudinal joints are usually preferred in common tunneling practice. In (GERMAN TUNNELLING COMMITTEE (DAUB) 2013), it is explicitly stated that an offset, by half or third of the segment length, should be considered to prevent continuous longitudinal joints across multiple rings. This in return strengthens the lining stiffness and the sealing effect. Moreover, it is not suggested to have the hydraulic jacks pads at the location of longitudinal joints. Therefore, the proposed position of joints is adopted in such a manner that they do not match the position of hydraulic jacks.

### 5.1.1 Lining-Soil Interaction

The relation between the outer boundary of the segment and the surrounding grouting mortar requires a particular consideration in the case of explicit modeling of the segmental lining. As shown in Figure 5.5, the assumption of mesh compatibility with nodal connectivity between the lining and the grouting is only valid for a continuous lining model. To enable segment-wise ring installation, and the correct kinematics of the joints, the connection between the lining outer surface and the grouting material is modeled by means of a surface-to-surface tying procedure, which does not require mesh compatibility. The tying constraint preventing the relative displacements is enforced at the Gauss points using a penalty approach. The energy functional associated with the penalty term is defined as:

$$
\Pi^{Tying}(u) = \frac{1}{2} \epsilon \| (u^{lining} - u^{grout}) \|^2,
$$

(5.4)

where $\epsilon$ denotes the penalty parameter.
5.2 Computational Contact Mechanics

In this section, the basic equations, that are required to describe the contact mechanics, are introduced. First, the mathematical description of a two body contact is represented. Then, the governing constraints of the contact problem and its regularization are discussed.

5.2.1 Mathematical Description of Contact Problem

In order to define the contact problem between two adjacent deformable bodies ($\Omega_s$ and $\Omega_m$), a slave surface ($\Gamma_s$) and a master surface ($\Gamma_m$) must first be defined on each respective body, see figure 5.6. Therefore, the boundary of each body can be decomposed as:

$$
\Gamma_D^i \cup \Gamma_N^i \cup \Gamma_c^i = \partial \Omega^i
$$

$$
\Gamma_D^i \cap \Gamma_N^i = \Gamma_D^i \cap \Gamma_c^i = \Gamma_N^i \cap \Gamma_c^i = \emptyset \quad ; \quad (i = s, m),
$$

where $\Gamma_D^i$, $\Gamma_N^i$ and $\Gamma_c^i$ are the displacement, load and contact boundary conditions over a boundary $\partial \Omega^i$ of the body $\Omega^i$. For two possible opposite contact surfaces between any two contacting bodies, a point on the slave surface $x^s$, and its closest point projection onto the master surface $x^m(x^s)$ can be defined as shown in figure 5.6-right.

Figure 5.6: Illustration of contact problem: contact between two bodies $\Omega_s, \Omega_m$ via their contact surfaces $\Gamma_c^s, \Gamma_c^m$ (left) and contact surfaces in the deformed configuration in which the contact point $x^s$, its projection $x^m(x^s)$ and the coordinate system $[n, \tau_\alpha]$ are depicted (right)
Upon doing so, a gap, \( g \), can be determined between the contact point and its projection. This gap is defined as the scalar projection of the distance vector, \((x^s - x^m(x^s))\), along the outward unit normal vector, \( n \), to the master surface at the projection point as follows:

\[
g(x^s) = -(x^s - x^m(x^s)) \cdot n(x^s) \tag{5.6}
\]

According to the sign convention for \( g(x^s) \), penetration of \( x^s \) into \( \Omega_m \) occurs when \( g(x^s) \) is positive (i.e., interpenetration of the two surfaces occurs when the condition \( g(x^s) > 0 \) is fulfilled). Geometrical linearity is assumed and therefore the normal vector, \( n \), remains independent of the displacement. The resulting traction vector acting on the surfaces of the two contacting bodies, \( t_N \), corresponds to the contact pressure. With respect to the gap sign convention, the contact pressure is assumed to be positive only if interpenetration occurs. The enforcement of contact constraints concerning impenetrability upon compression are stated in terms of Kuhn-Tucker optimality conditions (Laursen 2002) as follows:

\[
\begin{align*}
t_N &\geq 0 \\
g(x^s) &\leq 0 \\
t_N g(x^s) &= 0 
\end{align*}
\tag{5.7}
\]

**Extension to Frictional Contact**

Frictional contact forces between the two bodies are described by the Coulomb’s friction law. The latter formulates the frictional forces with respect of the corresponding tangential deformation as:

\[
u_T = -\alpha t_T \quad \text{with} \quad \begin{cases} 
\alpha = 0 & \text{if } \|t_T\| < \mu t_N \\
\alpha \neq 0 & \text{if } \|t_T\| = \mu t_N
\end{cases}
\tag{5.8}
\]

where \( \mu \) is the frictional coefficient between the contacting surfaces and \( \alpha \) is a scalar dimensional quantity by which it is ensured that the tangential deformation will be co-linear with the frictional forces. Coulomb friction law states that no tangential deformations occurs if the frictional forces are lower than \( \mu t_N \) which is referred to as stick condition. Then, the increase of frictional forces is limited to \( \mu t_N \) that represents slip condition at which, tangential deformations take place. To describe the kinematics of frictional contact, tangent vectors, \( \tau_\alpha (\alpha = 1, 2) \), associated with \( x^s \in \Gamma^c \) are defined over the master contact point, \( x^m(x^s) \in \Gamma^m_c \), see figure 5.7. As a consequence of the assumption of geometric linearity, the basis vectors \( \tau_\alpha \) are as well independent of \( u \).

Within the coordinate system defined by \( [n, \tau_\alpha] \), the tangent vectors, \( \tau_\alpha \), have a proper orientation such that \( n \) is the outward normal. These vectors are given by:

\[
\begin{align*}
\tau_\alpha &:= \Psi_m(x^m(x^s)) : \Gamma^m_c = \Psi_m \text{ (contact area)} \\
n &= \frac{\tau_1 \times \tau_2}{\|\tau_1 \times \tau_2\|}
\end{align*}
\tag{5.9}
\]

where \( \Psi_m \) represents the mapping of the master contact surface from the isoparametric domain to the current configuration.
5.2. COMPUTATIONAL CONTACT MECHANICS

Figure 5.7: Representation of tangential vectors $\tau_\alpha$ at the projected contact point $x^m(x^s) \in \Gamma^m_c$

The tangent vectors, $\tau_\alpha$, are basis vectors and $\tau^\beta$ are dual basis vectors defined such that the relation $\tau_\alpha \cdot \tau^\beta = 1$ holds, with $1$ being the unit vector. The metric tensor $(m_{\alpha\beta})$ defines the relation between the two sets of basis vectors such that:

$$m_{\alpha\beta} = \tau_\alpha \cdot \tau^\beta$$

$$\tau_\alpha = m_{\alpha\beta} \tau^\beta$$ (5.10)

With such a definition of dual basis, the relative tangential displacement and the frictional traction vectors can thus be expressed in components form as:

$$u^T_T = u^\beta_T \tau_\beta$$

$$t^T_T = t^\beta_T \tau^\beta$$ (5.11)

where $(\bullet)_\alpha$ and $(\bullet)^\alpha$ are the covariant and contravariant components of a field variable. The Euclidean norm $\| (\bullet) \|$ of the vector $(\bullet)$ can be determined with the metric tensor as:

$$\| u^T_T \| = \sqrt{u^\alpha_T m_{\alpha\beta} u^\beta_T}$$

$$\| t^T_T \| = \sqrt{t^\alpha_T m^{\alpha\beta} t^\beta_T}$$; with $m^{\alpha\beta} = m^{-1}_{\alpha\beta}$ (5.12)

5.2.2 Constraint Enforcement by the Penalty Method

In the proposed formulation, the contact constraints are enforced using a penalty method as discussed in (Laursen 2002). This method provides a straightforward enforcement of the contact constraint without the necessity to regard any additional degrees of freedom. Penalty regularization is achieved by introducing a normal penalty ($\epsilon_N > 0$) to the overall energy functional $\Pi^{sys}$ of the system, which yields to:

$$\Pi^{sys} = \Pi^s + \Pi^m + \Pi^{contact}$$; $\Pi^{contact} = \frac{1}{2} \int_{\Gamma^c} \epsilon_N \langle g \rangle^2 d\Gamma$ (5.13)

The Macaulay bracket $\langle \bullet \rangle$ gives the positive part of its operand. When penetration occurs, as indicated in figure 5.8, the sign of the gap becomes positive which raises the system energy based on the chosen value of the penalty parameter $\epsilon_N$. 

The normal contact traction \( t_N \) can be calculated in terms of the penalty parameter as:

\[
t_N = \epsilon_N \langle g \rangle
\]

The plot in figure 5.9 shows the representation of Kuhn-Tucker condition, in which, the contact pressure, according to equation 5.14, appears only if positive gap occurs. It should be noted that exact representation can be achieved only when \( \epsilon_N \) tends to \( \infty \). Such a value is not feasible as very high penalty values lead to ill-conditioning of the stiffness matrix.

The regularization of Coulomb friction law can be introduced by providing a tangential penalty \( \epsilon_T \), see figure 5.10. The regularization is similar to the elasto-plastic law. The evolution form of Coulomb friction is given as:

\[
\Phi(t_T, t_N) := \|t_T\| - \mu t_N \leq 0
\]
\[
\dot{t}_T = \epsilon_T \left[ \dot{u}_T - \frac{\dot{\gamma} t_T}{\|t_T\|} \right]
\]
\[
\dot{\gamma} \geq 0
\]
\[
\dot{\gamma} \Phi = 0
\]

The implicit determination of frictional forces are realized by a return map algorithm as follows:

- Trial state: Assume a no slip state within the current increment and calculate trial tangential forces as:

\[
t_{T_{trial}}^{n+1} = t_{T_{n}} + \epsilon_T m_{\alpha \beta} \dot{u}_{T}^\beta
\]
5.3. IMPLEMENTATION OF CONTACT ALGORITHM IN KRATOS

- Check the stick/slip condition with the yield function 
  \[ \Phi_{\text{trial}}^{n+1} = \|t_{T_{n+1}}^{\text{trial}} \| - \mu t_{N_{n+1}} \]

- Calculate the frictional forces as:
  \[
  t_{T_{n+1}}^{n+1} = \begin{cases} 
  t_{T_{n+1}}^{\text{trial}} & \text{if } \Phi_{\text{trial}}^{n+1} \leq 0 \quad \text{stick} \\
  \mu t_{N_{n+1}} & \|t_{T_{n+1}}^{\text{trial}} \| \end{cases}
  \]

\[ (5.16) \]

**Figure 5.10:** Regularization of Coulomb friction law using frictional penalty

5.3 Implementation of Contact Algorithm in KRATOS

The surface-to-surface contact is used for the fulfillment of the contact constraints, that are formulated at each quadrature point on the slave surface. Penalty regularization is achieved by introducing normal and tangential penalty values, \( \epsilon_N \) and \( \epsilon_T \), respectively as discussed in the previous section. Herein, the implementation of the contact algorithm within the finite element code KRATOS is explained and a numerical example is introduced to verify the contact algorithm.

Within the object-oriented implementation of the finite element code, ContactUtility has been developed for setting the contact constraints. First, a SlaveCondition and a MasterCondition are defined over the discretized contact surfaces on slave and master volumes respectively. In order to define each set of possible mutual contact surfaces, a variable (named ContactIndex) is introduced. Each possible contacting slave and master surfaces are assigned with the same ContactIndex. Within the simulation script, a function (SetupContactLinks) sets the contact condition at each Gauss point on the slave surface. To do so, contact detection is required. Therefore, a search algorithm is performed in two steps; global and local levels. First, a global search is performed to specify the set of master surfaces that have the shortest distances with the Gauss point of the corresponding slave surface. Distances are measured between the Gauss points on the slave surface and all nodes of the master surface. Then, a
local search is performed to determine the exact closest point projection of the Gauss point on the corresponding master surface. This requires solving the following minimization problem:

\[ x^m(x^s) = \arg\min ||x^m - x^s|| \]  

(5.17)

The previous equation is minimized within an iteration scheme for a set of linear equations by evaluating \( \alpha \) that satisfies the following:

\[ x^s - x^m(x^s) + \alpha n(x^m(x^s)) = 0 \]  

(5.18)

Once the Gauss point is projected, a ContactLink is created. The latter includes a pointer to the Gauss point and its corresponding slave surface, as well the projected point and its corresponding master surface. The contact forces (\( t_N \) and \( t_T \)) are evaluated between the Gauss point, \( x^s \), at the slave surface and its closest point projection, \( x^m(x^s) \), at the master surface. ContactLink conditions are called during assembly to add their contributions to the overall virtual work of the system within the Newton-Raphson iterations for solving global equilibrium. Figure 5.11 briefly summarizes the ContactUtility scheme implemented within the finite element code.

\[ \int_{\Gamma \subset \Gamma_c} \left[ t_N \delta g + t_T \cdot \delta u_T \right] d\Gamma \]  

(5.19)

In the numerical analysis with finite element method, the current position \( x \) and the displacement field \( u \) are approximated using the element shape functions and the respective nodal values \( x^k_i \) and
5.3. IMPLEMENTATION OF CONTACT ALGORITHM IN KRATOS

\[ \mathbf{u}^e_k \approx \tilde{\mathbf{u}}_c = \sum_{e} \sum_{k} N_k(\xi) \mathbf{u}^e_k \]

where \( NE \) and \( NN \) are the total number of elements and the number of nodes for element \( e \), respectively. \( \xi \) denotes the local coordinate at the respective point. With such a definition, the variation of gap, as defined in equation 5.6, can be obtained as:

\[ \delta g(\mathbf{x}^s) = - (\delta \mathbf{x}^s - \delta \mathbf{x}^m(\mathbf{x}^s)) \cdot \mathbf{n}(\mathbf{x}^s) \]

\[ = - (\delta \mathbf{u}^s - \delta \mathbf{u}^m(\mathbf{x}^s)) \cdot \mathbf{n}(\mathbf{x}^s) \] (5.21)

With equations 5.20 and 5.21, the weak form of the contact contribution can be discretized and formulated as:

\[ \delta W^\text{contact} \approx \delta \tilde{\mathbf{u}} \cdot \mathbf{R}^\text{contact}, \] (5.22)

where the force vector for the contact interaction \( \mathbf{R}^\text{contact} \) is determined for the slave and master surfaces as:

\[ \mathbf{R}_k^{\text{slave}} = \int_{\Gamma_k} N^\text{slave}_k \mathbf{T} \left[ n_i t_N - \tau_i^\alpha t_{T^\alpha} \right] dA \]

\[ \mathbf{R}_k^{\text{master}} = \int_{\Gamma_k} N^\text{master}_k \mathbf{T} \left[ -n_i t_N + \tau_i^\alpha t_{T^\alpha} \right] dA \] (5.23)

The \textsc{Newton-Raphson} solution scheme is used to solve a non-linear problem, which requires linearization of the discretized weak form. This is achieved by \textsc{gâteaux} derivative, which can be expressed as:

\[ \Delta \delta W_{n+1} = \delta \mathbf{u}_{n+1} : \{ \Delta \mathbf{R}_{n+1} \}_{n+1} \]

\[ = \delta \mathbf{u}_{n+1} : \{ \mathbf{K}_{n+1} \} \Delta \mathbf{u}_{n+1} , \] (5.24)

where \( \mathbf{K}_{n+1} \) is the tangent stiffness matrix, i.e. the \textsc{gâteaux} derivative of the internal force vector \( \mathbf{R}_{n+1} \) with respect to field variable (i.e. displacement). Within the iterative solution, the incremental displacement at the iteration \( k \) is determined by:

\[ \Delta \mathbf{u}_{n+1} = [\mathbf{R}^k_{n+1}]^{-1} \{ \mathbf{R}_{ext} + \mathbf{R}^k_{int} \}_{n+1} \] (5.25)

In order to reduce the effort in the implementation of the contact condition, the consistent tangent is numerically evaluated according to Lee and Park (2002). The stiffness matrix at the \( j \)-th column is numerically obtained as:

\[ \mathbf{K}^\text{contact}_j \approx (\mathbf{K}^\text{contact})_j = \mathbf{R}^\text{contact}_j [\mathbf{d} + \mathbf{\varepsilon}_{num} \Delta \mathbf{d}_j] - \mathbf{R}^\text{contact}_j [\mathbf{d}] \]

\[ \mathbf{\varepsilon}_{num} \Delta \mathbf{d}_j \] represents a very small perturbation at the \( j \)-th degree of freedom, where \( \Delta \mathbf{d}_j \) defines the direction (i.e. zero components except for the \( j \)-th element) and \( \mathbf{\varepsilon}_{num} \) is a very small perturbation value.
5.3.2 Verification of Frictional Contact Behavior

The applicability of the presented frictional contact algorithm is demonstrated via a numerical example. Figure 5.12-a depicts the geometry and the displacement boundary condition of two elastic rectangular blocks. In this example, Young’s modulus and Poisson’s ratio are assumed as \( E = 1.0 \) GPa and \( \nu = 0.0 \) for both blocks. The surface-to-surface contact conditions are generated between the two blocks. In this problem, the upper block is forced downward into the lower block via prescribed displacement of the upper boundary. Then, the upper block is moved to the right. The penalty penalization enables the transfer of forces between the two blocks. The output vertical displacement is given in figure 5.12-b.

![Figure 5.12](image)

**Figure 5.12:** Verification of frictional contact model: (a) geometry of the benchmark example with boundary conditions and (b) vertical deformation for the contacting bodies

The effect of penalty parameters as well as the frictional response is further investigated. The integral of contact pressure at the contact interface defines the total contact force \( \mathbf{F} \) transferred between the two blocks as:

\[
\mathbf{F} = \int \left[ t_N \mathbf{n} + t_T \right] dA \tag{5.27}
\]

The normal component of \( \mathbf{F} \) is plotted in figure 5.13 with different penalty parameters. Normal penalty penalize the violations of kinematic constraint. Large enough penalty values provides admissible state of deformation that is relatively close to the exact solution. Noting that the impenetrability can only be achieved with finite penalty valve. Therefore, as can be seen, the values of the penalty parameter have to be chosen as large as possible, considering that ill-conditioning of the matrix is avoided.

Coulomb friction law adopts no tangential slip when the slip function \( \Phi \), Equation 5.15, is less than zero. Numerically, tangential penalty is utilized to regularize such stick zone; the latter is similar to the elastic zone in perfect plasticity problems. Similar to normal contact, exact representation of Coulomb friction is obtained with infinite tangential penalty value. Figure 5.14 depicts how the tangential penalty regularizes the Coulomb’s frictional law. The total tangential contact force is plotted against horizontal displacement. For different tangential penalties, it can be seen that the tangential force does not exceed the coefficient of friction times the normal contact. Using low penalty
5.3. IMPLEMENTATION OF CONTACT ALGORITHM IN KRATOS

**Figure 5.13:** Normal penalty constraint; normal contact force vs. vertical displacement for different normal penalties

value, large stick zone is promoted which represents less accurate enforcement of Coulomb’s law. On the contrary, imposing large enough tangential penalty value will force any significant tangential deformation to be in the slip zone.

**Figure 5.14:** Tangential penalty constraint; tangential contact force vs. horizontal displacement for different tangential penalties

Figure 5.15 shows force-displacement relation with different coefficient of friction. The absolute frictional forces are restricted to $\mu t_N$. After this limit, horizontal displacement increases without imposing additional frictional loads. For a higher values of friction coefficient, sticking state should be expected and the frictional forces will be linearly varying with lateral deformation.

**Figure 5.15:** Normal contact force vs. horizontal displacement for different friction coefficient
5.4 Model Validation

In this section, two validation tests have been performed in order to demonstrate that the proposed contact formulation is capable of reproducing the main characteristics of the physical joints behavior. For this purpose, the results from the numerical models are compared with the experimental results of a single joint test and a full-scale test of complete rings.

5.4.1 Concrete Joint Test

In Hordijk and Gijsbers (1996), segment joint rotation tests were carried out at different values of normal forces. Figure 5.16-a depicts the geometrical configuration of the performed test, consisting of two segments (width 350 mm), which are in contact with each other via a contact surface of 158 mm width. The material properties are as follows: Elastic modulus 32.4 MPa, Poisson’s ratio 0.2, compressive strength 69.9 N/mm² and tensile strength 4.9 N/mm². The joint height equals 158 mm, which corresponds to 45% of the actual segment thickness. As shown in figure 5.16-a, the total normal force applied on the joint is equal to \( F_N + F_M \). Here, the eccentric loading component, \( F_M \), is used to apply bending moments along the joints. It should be noted that the following experiments were performed with and without bolting and it was shown that the bolts have no significant effect on the observed moment-rotation relationship.

The 2D finite element mesh, with the adopted properties and boundary conditions, is presented in figure 5.16-b. The lower block is fixed at the bottom and the upper blocked is compressed with controlled eccentric loading. A non-uniformly distributed loading is used to describe the eccentric loading. Contact conditions are defined only along the joint area of 158 mm. A linear elastic material law is used to describe the concrete behavior. Vertical displacements at different time steps are plotted in figure 5.17. The comparison between the experimental data, Leonhardt’s analytical solution Leonhardt and Reimann (1966), and the finite element results is shown in figure 5.18. Both analytical and numerical solutions show good agreement with experimental data, although the analytical solution predicts slightly softer response than the observed data.
5.4. MODEL VALIDATION

\[
\begin{array}{cccc}
M &=& 0.0 & 48 & 96 & 108 \\
\varphi &=& 0.0 & 1.04 & 3.08 & 4.07
\end{array}
[kN.m/m] [mrad]
\]

**Figure 5.17:** Segment joint rotation test: vertical deformations of the model as obtained from the numerical simulation at a normal force level of 1600 kN/m

**Figure 5.18:** Segment joint rotation test: Moment-rotation relationship (a) comparison between experimental data and analytical solution and (b) comparison between experimental data and numerical solution

It is worth mentioning that no visible cracks (only hairline cracks) were observed on the test specimens. Therefore, results of numerical analysis match well with the experimental data which represent more or less the first two stages of the joint behavior as previously discussed. Since linear elastic material behavior is assumed in the analysis, it is expected that, for higher level of compressive stresses, numerical analysis may tend to over estimate the joint response as the stresses in the Gauss points is not bonded and the use of a suitable constitutive law will further improve the structural behavior. However, the verification of this is not within the scope of this thesis. The results show that the proposed contact algorithm captures the joint stiffness of segmental tunnel linings with sufficient accuracy and can replicate its physical behavior. This particularly holds for practical applications where the tunnel linings are designed to remain within the elastic range for any operational loads.
5.4.2 Full-Scale Test of Botlek Railway Tunnel (BRT) Segments

As was shown in the previous subsection, contact mechanics can replicate the joint behavior of the segmental tunnel lining. However, the structural behavior of the full-scale lining system including both longitudinal and ring joints still needs to be validated. For this purpose, the full-scale laboratory test of the BRT tunnel lining is considered (BLOM 2002, BLOM AND VAN OOSTERHOUT 2001), see figure 5.19. The latter has been performed with different loading scenarios to study the structural behavior of segmental tunnel lining during construction. Loading conditions can be summarized as follows:

- Load at once tests, which means that all segmental rings during testing are simultaneously loaded at serviceability limits
- Sequential loading tests, that represent loading during construction where subsequent segments have different loading conditions
- Loading exerted by the misalignment of the joints, such loading stands for geometrical tolerances and local deviation from perfect ring shape
- Loads during assembly and ring closure with the installation of a tight and loose keystone
- Jack forces from TBM steering

The test consists of three vertical rings, with an outer diameter of 9.05 m, that are placed with different orientation of the longitudinal joints, see figure 5.20 and table 5.1. The segmental rings in the test are assembled using the actual segments from the construction site of the BRT. The ring is constructed using 7+1 segments with a wedge-shaped keystone that is smaller than other segments. A standard loading procedure was applied on the tested rings. Loading consists of uniform and ovalisation radial loading as illustrated in figure 5.21. The level of loading is in the range of expected normal operation loading conditions. In addition, TBM jack forces are applied in the vertical direction.

An analytical solution, presented by BLOM (2002), was developed to simulate two consecutive segmental rings. Therefore, the analytical solution, for replicating this test, only considers two rings with half ring width. The test results have been used as a validation of the aforementioned
5.4. MODEL VALIDATION

Figure 5.20: Full-scale test of tunnel segments: joints arrangement for the top, middle and bottom rings

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lining outer diameter</td>
<td>9.05 m</td>
</tr>
<tr>
<td>Lining ring width</td>
<td>1.50 m</td>
</tr>
<tr>
<td>Lining thickness</td>
<td>400.0 mm</td>
</tr>
<tr>
<td>Longitudinal joint length</td>
<td>170.0 mm</td>
</tr>
<tr>
<td>Young’s modulus of concrete</td>
<td>40 GPa</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.2</td>
</tr>
</tbody>
</table>

Table 5.1: Geometrical and material parameter of the full-scale test

Figure 5.21: Full-scale test of tunnel segments: radially applied loads and its subdivision into a uniform compressive load and an ovalising load

analytical solution. It should be noted, however, that the analytical solution neglects the existence of the keystones and assumes seven equally divided segments. In addition, it does not explicitly apply the axial forces in the analysis, instead, coupling frictional forces are used.

To validate the behavior of the proposed numerical model, the test results of the loading scenario (load at once) are used and compared with the outputs of numerical analysis. Figure 5.22 shows the geometry and discretization of the numerical model. The model is loaded in radial and axial direc-
Radial loading is applied according to equation 3.6, in which the compression loading $\sigma_{\text{comp}}$ and ovalisation loading $\sigma_{\text{oval}}$ are equal to 492.5 kPa and 18.5 kPa, respectively, see figure 5.21. The contact interactions between the segments are defined according to figure 5.23, in which, a distinct contact index is defined for each mutual contact surface. Contact indices from 1-7 defines longitudinal joints and 8-9 defines ring joints. In the full-scale test, the following measurements were performed: radial deformations and concrete strains at the outer and inner fibers. The laboratory data shown in this subsection is only for ovalized loading, which is achieved by reducing the initial uniformly compressive load at the selected positions. Consequently, the loading in the numerical model is also applied in two steps. In the first step, only a uniform compressive load is applied. Then, the ovalisation loading is applied. And therefore, the change in deformations and stress states between the two loading steps represents the output resulting from the ovalisation load only, with the effect of the uniform radial compression load being deducted.

**Figure 5.22:** Description of the numerical model of the Full-scale test: segments volumes with the applied loading and displacement boundary conditions (left) and discretization of one segment (right)

**Figure 5.23:** Definition of contact surfaces in the numerical model of the Full-scale test
Figure 5.24 shows the measured and the predicted radial deformation of the top and the middle rings. Both top and bottom rings have longitudinal joints at the same location and therefore they experience the same radial deformation. The overall structural deformation resembles an ellipse following the applied ovalization loading. Compared with the numerical model, the numerical results agree well with the measurements data.

![Radial deformation comparison](image)

**Figure 5.24:** Re-analysis of full-scale test of tunnel segments: comparison between the measured radial deformations and the predicted ones from the numerical simulation; top ring (top) and middle ring (bottom). The locations of the joints are indicated by vertical dashed lines.

Strain measurements in the full-scale test have been used to predict the tangential bending stresses. Figure 5.25 compares the measured tangential bending stresses with the numerical results for the stresses in the finite elements at the outer fiber of the segmental ring model. It can be noted that the numerical results generally fits well with measurements, however, some deviations can be noticed near the joints location. The maximum negative tangential stresses is located at an angle of 0° for the middle ring and an angle of 180° for the top ring, these locations match the test measurements accurately. The maximum positive tangential stress at the sides of the top ring is slightly underestimated, while the tangential stresses at the joints (more specifically at angles of 77° and 180° for the middle ring and an angle of 102° for the top ring) are not well predicted. At the joint locations, the strain measurements were taken from strain gauges placed directly on the left and the right hand sides from the joints (at the top and bottom face), i.e. on locations, which are strongly affected by the 3D stress distribution in the vicinity of the joints, denoted as “boundary effects” in (BLOM 2002). To resolve this distribution more accurately would require a considerably more refined spatial resolution of this area.
Strain measurements in the full-scale test have been used to predict the tangential bending stresses. Again, the influence of axial compression has been subtracted and the shown results are due to ovalisation only (bending loading). Figure 5.25 compares the measured tangential bending stresses with the stresses in the elements at the outer fiber of the segmental ring model, which can be extrapolated by linear interpolation of the stress along the thickness. It can be noted that the numerical results generally fits well with measurements, however, some deviations can be noticed near the joints location. The maximum negative tangential stresses is located at the crown of the middle ring and at the invert of the top ring, these location match the test measurements accurately. The maximum positive tangential stress at the sides of the top ring is slightly underestimated. While, the tangential stresses at the joints are not exactly estimated. Since the determination of stresses at joints location is difficult either in the laboratory or numerically. In experiment, there exist boundary effects of concrete at the sides near the joints (BLOM 2002). On the other hand, numerical errors appears at the boundaries of the contact surfaces, such an error can be reduced with finer mesh discretization.

![Figure 5.25: Re-analysis of full-scale test of tunnel segments: comparison between the measured tangential bending stresses and the predicted bending stresses from the numerical simulation for the top ring (top) and the middle ring (bottom). The locations of the joints are indicated by vertical dashed lines.](image)

Since the design of tunnel lining segments requires a proper estimate of structural forces (i.e. normal forces and bending moments) in the linings, it is crucial to compare the developed bending moment in the numerical model. The resulted bending moments are compared with the analytical solution by BLOM (2002). It should be noted that the analytical solution assumes only two consecutive rings, therefore, the middle and top rings undergo the same extreme moments. For example, the maximum positive moment at the crown of the middle ring is 272 kNm and similarly, the maximum
positive moment in the top ring is 272 kNm and located at the invert. As well, the maximum negative moment is similar and equals to -228 kNm at the sides, located at the center of the segment. Such a behavior is justified, since the confinement of the middle ring is not achieved. On the contrary, the numerical model including the three rings account for such a confinement. The maximum positive and negative moments in the middle ring are 288 kNm and -275 kNm respectively. The top ring undergoes lower bending limits of 222 kNm and -203 kNm. This matches with the measured tangential stresses, where the extreme tangential stress in the middle ring are higher than the top ring. Quantitatively, the relative stiffness between top and middle ring can be evaluated by means of moments reduction in the numerical model and tangential stress reduction in the measurements data. A reduction of the maximum bending moments from 288 kNm to 228 kNm between the middle and top ring equals to 77%. Similarly, the corresponding maximum tangential stresses at these locations are approximately 6.5 MPa and 4.8 MPa, which equals to a reduction of 74%. Generally, the comparison of the measurements data and the numerical analysis results confirms the capability of the proposed numerical model.
Chapter 6

Numerical Assessment of Different Lining Models

In the design phase of tunnel lining, various approaches that differ in precision and complexity are employed to predict the lining structural forces. This chapter discusses the lining response with respect to different modeling approaches (i.e. bedded and continuum models). The proposed segmental lining model in the process oriented simulation ekate including lining joints by means of contact is used for the analyses of mechanized tunnel lining. The effect of joint is investigated by the comparison with different joints arrangements and with the continuous lining tube. The second model is the bedded beam model, a classical two rings model proposed by DAUB (German Tunnelling Committee (DAUB) 2013). Further investigation is presented by the comparison of the two models. This chapter aims to provide in-depth understanding into the extent to which the modeling level of detail plays a role in regards to tunnel lining design.

6.1 Geometrical Configuration and Properties

The numerical example represents a straight tunnel with an overburden depth of 1.7D assumed to be driven by a slurry tunneling machine in cohesionless soil. The soil is assumed to be fully saturated, with an elastic modulus $E = 50$ MPa, Poisson’s ratio $\nu = 0.3$, saturated density $\rho_{sat} = 2000$ kg/m$^3$, cohesion $c = 0.01$ kPa, effective friction angle $\phi = 30^\circ$, lateral earth pressure coefficient $K_0 = 0.42$ and water permeability $k_w = 10^{-5}$ m/s. The dimensions and the finite element discretization of the investigated tunnel section are depicted in figure 6.1. The finite element mesh, used in the model, approximately contains 31,500 various elements with 278,000 nodes.

The process parameters used in the 3D process oriented simulation are chosen according to the typical operational range. Tunneling is assumed to be performed by a shield machine, with a front diameter of 9.49m, a rear diameter of 9.47m and a cutting wheel diameter of 9.52m. The
weight of the machine (1,010 tons) is applied as a distributed pressure on the bottom of the shield skin. The shield is steered with twenty-eight hydraulic jacks that are equally distributed along the circumference of the machine. The individual elongations of each jack during machine advancement are obtained from the steering algorithm described in section 2.4.3. The lining shell is constructed using 7 segments configuration with elastic material properties of concrete (elastic modulus $E = 30$ GPa, Poisson’s ratio $\nu = 0.2$ and density $\rho = 2500 \text{ kg/m}^3$). Detailed lining dimensions and joints orientations are depicted in figure 6.2. With the adopted joints orientations of the two consecutive rings as shown in figure 6.2, longitudinal joints are installed in a staggered position.

**Figure 6.1:** Numerical analysis of the structural forces in the linings of a straight tunnel driven in soft soil: dimensions and finite element mesh used in the simulation model

**Figure 6.2:** Dimensions of the staggered segmental tunnel lining rings used in the investigated simulation model
6.2 Segmental Lining Model Embedded within the Process Oriented Simulation

In this section, the forces and the deformations of the segmental lining shell are evaluated according to the proposed segmental lining model incorporated in the process oriented model\textsuperscript{ekate}, in which the segment-wise installation procedure of the lining segments is implicitly considered. In this model, the soil-structure interactions and the time dependent loading of the lining from the surrounding tail void grout and the adjacent ground as well as the mutual interactions between the lining segments are accounted for. Hence, one may expect from this type of analysis to obtain a realistic insight into the structural behavior of segmental linings during mechanized tunneling. It should be noted, that in the model, due to the pressurization and hydration processes of the grouting material and due to consolidation processes in ground water saturated soft soils, the lining forces are time dependent. For the comparative assessment of models, the steady state of the lining is approximately considered after 15 m (i.e. after the installation of 10 rings).

Figure 6.4 shows the normal forces, bending moments and radial deformations of the tunnel lining at different construction steps. In agreement with the real construction process, the segmental ring is activated inside the shield. In this stage, it does not bear loading except its own weight, longitudinal jacking thrust and the coupling forces with the preceding ring. Once the ring leaves the shield and is completely embedded in the pressurized grout (after app. 1.5 hrs), it sustains radial loading from the ground (see Figure 6.4).

**Figure 6.3:** Development of mechanical properties and permeability for the annular gap grouting mortar with time. Circles indicate to experimental measurements of stiffness evolution according to (Schulte-Schrepping et al. 2018)

- Stiffness ratio - $E / E_{t28}$
- Grout permeability - $k_w$
- $E_{t28} = 700$ MPa
- $\nu = 0.3$
Figure 6.4: Segmental lining model embedded in process oriented advancement simulation: spatio-temporal response of the segmental lining for the investigated tunnel section (normal forces, bending moments and radial deformations) at four construction stages (1, 3, 7 and 15 rings after installation)

As shown in figure 6.4, the lowest normal force is generally recorded at the crown of the ring. On the other hand, the location of the maximum normal force changes with time. Once the ring is loaded, the maximum normal force is at the invert and eventually at steady state, it moves to the springline. This distribution agrees with the evolution of the ring configuration, i.e. an increasing ovalization with time (figure 6.5). With the hydration induced stiffening of the grouting mortar, the loading distribution acting on the lining, initially being equal to the applied pressure distribution of the fluid grouting material, changes gradually. The stiffening grouting allows for the transfer of
bending deformations to the lining, which increases the bending moments as well as the ovalization of the lining deformation, as was also discussed in Ninic and Meschke (2017). The temporal evolution of the lining ovalization is illustrated in figure 6.5. It shows, that a quasi steady state situation is reached after a time span of approximately 15 hours during which 10 rings are installed. In figure 6.4, the spatial distribution of the moments are plotted for one ring (the joint locations of this particular ring are illustrated on the right hand side of figure 6.4) for three different stages (1, 3, 7 and 15 rings after installation). The diagram shows the gradual increase of positive and negative moments during the initial phase, eventually reaching a steady state between 7 and 15 rings after installation. The maximum positive/negative moments are 44/-45 kNm/m and 104/-108 kNm/m after 1 ring and at steady state, respectively. The shape of the bending moment distributions at the different stages show the typical trend (i.e. the maximum positive moments are located at the crown and at the invert of the tunnel and the maximum negative moments are at the springlines). The existence of joints influences the moment distribution, as is reflected by the fact, that the moment distribution is not symmetric and the moment distribution at the springlines differ considerably. At 90°, where the middle of a segment is located, a higher moment is observed as compared to an angle of 270°, where a joint is located, leading to a significant reduction of the moment. Therefore, the effect of joint location is discussed in the next subsection. The increasing radial deformations shown in figure 6.4 are compatible with the increasing bending moments. Due to the presence of joints, non continuous radial deformations are obtained along the lining outer circumference. Except at 270°, a slight relative slip is observed at the joint locations with an average value of 0.6 mm and a maximum value of 0.9 mm at 218°. The final values of the horizontal and vertical convergence are predicted as approximately 8 mm and 9 mm, respectively (see figure 6.5).

**Figure 6.5:** Segmental lining model embedded in process oriented advancement simulation: computed horizontal and vertical convergence of the lining

### 6.2.1 Continuous and Segmental Lining Models

To assess the influence of joints in the context of 3D tunnel advancement simulations in mechanized tunneling, the previous results from the segmental lining installation procedure are compared to results from the modeling of segmental linings as a continuous tube, in which, the tunnel construction and advancement procedure is the same. In course of the simulation of shield supported tunnel, the simulation with continuous lining model adopts a ring-wise installation of the linings.
Figure 6.6 (top) shows the distribution of normal forces at steady state, i.e. 15 rings after the lining ring installation obtained from the two analyses. The red and green curves are related to analyses using a continuous lining model and a segmental lining model. The normal forces obtained from the analyses are comparable, which indicates that the segment joints in the present analysis are fully capable of transmitting the normal forces in the ring. It is noted, that the situation is different in cases, where excessive rotations connected with joint openings occur.

Figure 6.6 (bottom) shows the distribution of the bending moment from the two analyses. The consideration of the joints (green line) significantly reduces the bending moments. While the maximum positive and negative moments for the embedded installation scheme are recorded as 104 kNm/m and -108 kNm/m, the moments obtained from the continuous lining installation scheme with standard activation of the lining elements are obtained as 144 kNm/m and -149 kNm/m, respectively. The reduction of bending moments resulting from considering the segmentation of the lining shell is particularly large for the joints located at the crown, invert and the springlines, see, e.g. joint f in Figure 6.6 (bottom). In this case, the more realistic segmental lining installation scheme leads to a moment reduction of 54% at this location. According to this model, the maximum bending moments are not directly located at the joint locations, while the continuous lining model provides the maximum moments at the crown, invert and springlines.
In order to study the effect of considering lining segmentation in the simulation model on the computed ground deformations, the computed longitudinal surface settlement profiles are depicted in figure 6.7 for the two model variants. If, the lining elements are installed in a segment wise manner by means of contact conditions along longitudinal and radial joints, as it is proposed in this contribution, slightly larger ground deformations is obtained with the segmental lining model (green line) compared to the continuous lining model using the standard lining element activation scheme (red line). The difference of maximum settlements is approximately 1.2 mm. This reflects the reduction of the overall lining stiffness in this model due to the consideration of lining joints, in particular of the ring-to-ring coupling.

**Figure 6.7:** Influence of lining modeling approaches on surface settlement profile

### 6.2.2 Influence of Tunnel Overburden

Three different overburdens (1.3D, 1.7D and 2.0D) related to typical ranges of shallow tunneling projects are investigated in this subsection. In the comparative analyses, the support pressures, i.e. the face pressure and the grouting pressure are increased with the increasing overburden. In addition, the applied grouting pressure during construction is normally higher than the face pressure. This prevents, according to Nagel (2009), the inflow of the support medium from the excavation chamber. Herein, the face support pressure $P_{f,0}$ at the center of the tunnel is chosen equal to the water pressure at that depth, and the grouting pressure at the center of the tunnel is chosen as 1.3 $P_{f,0}$. In both cases, as described earlier, a linear increase of the supporting pressures with depth is considered.

Figure 6.8 shows the distribution of the normal forces and the bending moments at steady state (i.e. after 15 rings) for the three investigated levels of overburden. As expected, the normal forces increase proportional to the increasing depth. The computed average normal forces are 640 kN/m, 850 kN/m and 1025 kN/m for tunnel overburdens of 1.3D, 1.7D and 2.0D, respectively. Also, increasing maximum bending moments are observed. The maximum absolute moments are increasing from 91 kNm/m to 108 kNm/m and 120 kNm/m, respectively, with increasing depth of the tunnel. This corresponds to an increase of 60% of the average normal forces and 32% increase of the bending moments by increasing the overburden from 1.3D to 2.0D.

In order to investigate the influence of the overburden on the joint behavior, the moment-rotation relationship is plotted in figure 6.9 for two joints located at an angle of 12.85° (Joint-a) and 270° (Joint-f), respectively at steady state of the lining response. With increasing overburden, the joint stiffness increases since the normal forces induced in the lining ring increases with depth.
For comparison, the dashed lines represent the analytical solution, provided by Leonhardt and Reimann (1966), for different normal forces corresponding to the average normal forces recorded for the three variants. Generally, the joints in the segmental lining model exhibit a stiffer response as compared to the analytical solution. This can be explained by the fact, that the segments are confined by the pressurized grout and the soil which imposes an additional stiffness to the joints. While, this is accounted for in the 3D model considering the segmental lining installation the analytical solution is based on the assumption of joints subjected to compression without accounting the additional stiffness of the neighboring ground. The diagrams in figure 6.9 show, that this additional stiffness contribution from the confinement slightly increases with increasing depth.

Figure 6.9: Influence of different overburden on the computed moment-rotation relationships for two joints (dots: 3D model, dashed lines: analytical joint model)
6.2.3 Influence of Joint Arrangement

The influence of the joint pattern is investigated by analyzing the segmental lining response for three different locations of joints, keeping the number of joints constant. Figure 6.10 shows the distributions of the bending moments for Models a, b and c, characterized by a joint at an angle of 12.85°, 25.71° and 0.0° from the crown. In the three models, the joints of subsequent rings along the tunnel shell are assumed to be arranged in a staggered configuration as recommended in most design guidelines. To obtain an insight into the relative influence of a staggered vs. an aligned joint configuration, Model a has also been analyzed considering an aligned placement of joints along the complete tunnel length.

Figure 6.10: Influence of different joint patterns on the computed bending moments in segmental tunnel linings

The top diagram in figure 6.10 shows, that the staggered joint configuration increases the lining stiffness and, consequently, the maximum bending moments. Directly at the location of the joints,
no considerable change in moments is noticed. The maximum positive and negative moments are reduced from 104/-108 kNm/m for the staggered joint configuration to 77/-79 kNm/m for the aligned joint configuration. On the other hand, changing the joint location from Model a to Model b and Model c (middle and bottom diagrams in figure 6.10), the moment distribution and the location of maximum moments are affected, while, however, the maximum and minimum bending moments are only marginally influenced by the joint placement. Fixing the location, e.g. to the tunnel crown, the bending moments at the crown are obtained as 83 kN/m, 98 kN/m and 67 kN/m for Models a, b and c. The largest moment is obtained for Model b, as in this case, the crown is located at the symmetry of the top segment, while the smallest bending moment is obtained for Model c, where a joint is directly located at the crown. The maximum positive and negative bending moments recorded along the circumference of the tunnel ring are 104/-108 kN/m at 186° and 96° for Model a, 99/-103 kN/m at 6° and 96° for Model b and 105/-100 kN/m at 174° and 276° for Model c. For a rigid ring, the maximum moments are located at the crown, invert and sides of the tunnel. In a segmental ring, the maximum moments are usually located at sections that are in the middle of the segments and as well near the crown, invert and sides. The existence of a joint at a certain location reduces the moment according to the capacity of the joint which is more noticeable in the section where maximum moments are expected. For example, the moments at the crown for model a, b and c are 83 kN/m, 98 kN/m and 67 kN/m. The highest are for model b as the middle of the segment is located at the crown. In model a, the moment is somewhat reduced as the segment mid point is rotated from the crown. The lowest value is in model c where a joint is located at the crown. In general, normal force envelopes are similar for these models. This can be justified from figure 6.6-(top) as the existence of joints do not affect the lining capacity to sustain compression forces.

The computed bending moments are plotted versus the rotations at different joints (Joints a, b, e and f) in figure 6.11 for Models a, b and c using a staggered joint arrangement and for Model a using an aligned joint arrangement along the tunnel length. For comparison, the moment-rotation relationship as obtained from the analytical solution is included as dashed lines. In all cases, the joint characteristics obtained from the 3D lining segmental lining model always show a stiffer response compared with the analytical solution as discussed earlier. Moreover, all joints herein are experiencing relatively small rotations (< 1.0 mrad). In joints located at the crown, invert and springlines, a gap is opening. These joints are indicated in red color for joint a in Model c and joint f in Model a. Hence, it is expected, that with further loading, these joints would reach their ultimate capacity first. Therefore, it is not favorable to place the joints at these locations. This has been addressed also in BLOM (2002), where a possible failure mechanism (referred to as snap through mechanism) is investigated. It was shown that these joints may undergo large rotations and reach their ultimate capacity, leading to large deformations such that the lining ring does not resist compressive forces. In order to avoid having a joint at the crown, invert or springline in this scenario, the top joint a should be located at an angle of 19.28° from the crown. Such presumption matches with the findings presented in DO ET AL. (2013), where 2D simulations are used to determine the favorable locations of joints leading to higher flexural stiffness.
6.3 Bedded Beam Model

In this section, the classical analysis of the tunnel lining, used in the numerical example, is performed using the bedded beam model as presented in Figure 6.12-a. With respect to rotational spring stiffness and shear spring, Figure 6.13 shows both the moment-rotation relationship and the piecewise force-displacement relationship for describing the longitudinal and ring joints respectively. With regards to bedding, the concept of active/inactive springs is applied where spring stiffness is considered only in compression. In addition, different assumptions are adopted for the representation of the lining-soil interaction as shown in Figure 6.12-b, e.g., shallow tunnels are characterized by lack of support at the crown while deep tunnels are assumed to be fully bedded (DUDDECK 1980, PUTKE 2016). In the DAUB recommendations (GERMAN TUNNELLING COMMITTEE (DAUB) 2013), the bedding assumptions according to (DUDDECK 1980) are suggested; shallow soft ground tunnels have no bedding in the $90^\circ$ area around the crown. In the literature, both fully bedded and partially bedded beam models are proposed, however, it should be noted that such assumption should be selected along with the adopted loading assumption (i.e. if uplift forces are considered, the ground is expected to support the lining at the crown). In addition, it is suggested that grout pressurization
should provide certain degree of bedding all around the tunnel even for shallow tunnels.

Figure 6.12: Bedded beam model for tunnel linings analysis: (a) structural model with non-linear rotational springs and shear springs, and (b) model with different bedding assumptions

Moreover, the analysis of tunnel lining by bedded beam models is generally based on simplified loading assumptions and such simplification improperly reflects the actual loading condition. Therefore, the applied process oriented finite element model can, however, serve as a tool for capturing more accurate incorporation of the exact loading conditions (NINIC AND MESCHKE 2017). A comparison of results from the classical bedded beam models and the segmental lining installation procedure in the context of a 3D advancement simulation is presented in the next section.

In this study, the bedded beam model is analyzed with different loading assumptions that are generally based on the in-situ stress state. These assumptions are summarized in Figure 6.14. The first assumption is proposed in AHRENS ET AL. (1982). As shown in Figure 6.14-a, uniform pressures at the tunnel crown, springline and invert represent the total loading (i.e. vertical pressure is the total overburden at the crown and horizontal pressure is the total lateral pressure at the center of the tunnel). The linear variation of the in-situ loading across the height is neglected. Uplift of the tunnel is therefore also neglected (i.e. the vertical forces remain in equilibrium). Although lining
weight is not considered in the analytical solution in AHRENS ET AL. (1982), it will be accounted for in the analysis for comparison with other models. The JSCE model suggests loading assumptions as shown in Figure 6.14-b, which are used as well in a closed form solution for calculating member forces of circular tunnels JAPANESE SOCIETY OF CIVIL ENGINEERS (JSCE) (1996). The uniform vertical loads at the crown represent the total overburden (i.e. earth and water pressure). The loads are in equilibrium in both vertical and horizontal directions, the lateral pressure varies linearly with depth and an additional triangular lateral loading is considered. A detailed design example, including the calculation of this loading assumption, is presented in (ITA WORKING GROUP NO. 2 2000). ITA also provided definitions for the different loading that should be included in the design. Figure 6.14-c illustrates the earth pressure, water pressure and dead loads as proposed by ITA (ITA WORKING GROUP NO. 2 2000). Finally, the loading assumption in Figure 6.14-d is based on engineering practice; it is determined according to the design recommendation of a reference project. The vertical earth pressure on top is not uniform and assumed to increase with depth as represented
by the parabolic curve. The vertical pressure on bottom is reduced by the buoyancy forces acting on
the lining. Detailed calculations of loads are provided in appendix C.

In general, it is obvious that these loading assumptions do not account for the stress state during
construction (i.e. stresses redistribution in the tunneling vicinity). It neglects many conditions
related with shield tunneling. As a consequence, the loading conditions in bedded models are de-
termined via distributed loads that simplify the original in-situ stresses in the ground and the soil
structure interaction is reduced to springs.

Figure 6.15 shows the structural forces obtained by the bedded beam model for different load-
ing assumptions. The normal forces for all loading assumptions are in the same range except for
loading assumption (a); as it predicts lower normal force level. The maximum positive and negative
moments are 86/-75 kNm/m, 104/-77 kNm/m, 148/-135 kNm/m and 142/-139 kNm/m for loading
assumptions from (a) to (d), respectively, which corresponds to differences of 70% and 85% be-
tween the upper and lower limits of the maximum positive and negative moments. This shows that
the predicted maximum moments are highly dependent on the adopted loading condition.
In this section, the results from the segmental lining installation procedure incorporated in the 3D process oriented advancement model ekate are compared with the results from the different bedded models discussed in the previous section. In figures 6.16-6.18, the gray shaded area represents the range of the bedded beam model responses with the loading assumptions illustrated in figure 6.14. In figure 6.16, the computed radial pressure on lining from the segmental lining model incorporated in ekate and the used loading assumptions in the bedded beam model are compared. As pointed out earlier, bedded models adopt a simplified loading on the lining that represent the in-situ state of stresses without considering soil-structure interactions. In contrast, the 3D continuum advancement model captures the tunneling induced stresses redistribution. (e.g. successive ground excavation, heading support, shield overcut, shield conicity, annular gap grouting pressure and grouting hydration process). These processes have a large influence on the predicted lining structure forces, as discussed in chapter 4.

The top diagram in figure 6.17 contains the distribution of the normal forces. It is observed, that all embedded beam models generally tend to significantly overestimate the normal forces as compared to the results from the 3D simulation model. The average normal force obtained from the bedded beam model is \( \sim 1560 \text{ kN/m} \). For the segmental model in ekate, the average normal forces are \( \sim 1085 \text{ kN/m} \) and \( \sim 850 \text{ kN/m} \) for an overcut of 1 cm and 3 cm, respectively. Yet, according to the bottom diagram in figure 6.17, the bending moment distribution obtained from the full 3D simulation
Figure 6.17: Predicted lining responses as obtained from the 3D advancement simulation model in comparison with the predicted range of responses from the bedded beam models with various loading assumptions (gray shaded area)

model is within the range of the response obtained from the bedded beam model. Generally, loading assumptions (a,b) and (c,d) form the lower and upper bound respectively for the bedded model response in bending. The predictions from the 3D simulation model for the maximum moments are best reflected by loading assumption (a) in the vicinity of the crown and by loading assumptions (a,b) in the vicinity of the invert and at $\sim 270^\circ$. At $\sim 90^\circ$, the 3D model provides an average value in comparison with other loading assumptions. It is noted, however, that this cannot be generalized to different conditions. The effect of joints is more pronounced in the segmental installation model, in particular at the joints located at $167^\circ$ and $270^\circ$, respectively. In the bedded beam models, the moment distribution is more or less smooth since the large predicted normal forces increase the joint stiffness.

Moreover, due to the large compressive normal forces predicted by the bedded beam models, the eccentricities of the moment-normal force ratio are smaller as compared to those obtained by the 3D segment-wise installation simulation model. According to figure 6.18, the eccentricity from the 3D segment-wise installation model (with an overcut of 3 cm) is 24% larger at $\sim 0^\circ$, 56% larger at $\sim 90^\circ$, 12% larger at $\sim 180^\circ$ and 8% larger at $\sim 270^\circ$. Hence, as it can be assumed, that the segment-wise installation procedure incorporated directly in a 3D advancement simulation provides
the more realistic loadings and stresses in the segmental lining shell, it can be concluded, that the bedded beam model is not always a conservative design approach. Nevertheless, for the specific case analyzed in this chapter, the predicted eccentricities are located inside the cross section of the lining. It must be noted, that the bending moments and, consequently, the eccentricities strongly depend on project specifications and ground conditions.

The discrepancy of the results obtained from the two models reflect the fact, that in bedded beam models, the adopted loading acting on the lining represents the weight of the soil and its lateral pressure with no consideration of the stresses redistribution in the tunneling vicinity due to soil-structure interaction. In contrast, the process oriented simulation model including the segment-wise installation of the lining shell accounts for the stress redistribution in front, along and at the tail of the shield machine and is capable of considering the 3D arching action in the soil, connected with a re-distribution of the loading acting on the lining via the grouting material. Moreover, time dependent loading conditions are implicitly considered, as they emerge from the simulation of the construction process as was shown in section 6.2.

Figure 6.18: Predicted eccentricities in lining longitudinal cross section as obtained from the segment wise lining installation incorporated in a 3D advancement simulation model (blue curves) in comparison with the adopted loading assumptions in the bedded beam models (gray shaded area)

6.5 Assessment of the Load-carrying Capacity

The design process of tunnel lining requires a comprehensive identification of all loads acting on the lining during construction and throughout its lifetime. According to the required standards, different loading conditions should be considered such as segments weights, earth pressure, water pressure, surface loads, loads from existing or future buildings, changes in water level, temperature effects, traffic loads, grouting pressure, etc. Upon calculation of the stress resultants (i.e. normal force and bending moment), cross sections with maximum positive/negative bending moment and cross sections with maximum compressive axial force with respect to the various loading combinations,
are identified as the critical sections during the design process (ITA WORKING GROUP No. 2 2000). At these sections the structural stability should be checked. In addition, the factor of safety against partial area loading should also be checked at the jacks locations and at the joints, however, this is beyond the scope of this contribution.

Herein, the comparison is extended to investigate to which extent, different modeling schemes influence the lining design, particularly the required amount of reinforcement. It should be noted that this section does not aim to provide a complete lining design, instead, it provides a brief insight of the design process considering the range of the adopted loads for different modeling level of detail. Figure 6.19 (left) depicts the concrete dimensions of the longitudinal cross section of the tunnel segment with an equal amount of steel rebars at the extrados and intrados steel meshes. The basic assumptions for the cross sectional design in accordance with (DEUTSCHES INSTITUT FÜR NORMUNG 2011) is considered, in which, the stress and strain distribution is assumed as depicted in figure 6.19 (right) and with the consideration of the material parameters listed in table 6.1.

![Illustration of the lining’s longitudinal cross section: concrete dimensions and steel rebars reinforcement (left) and strain and stress distributions in concrete along the thickness (right)](image)

**Table 6.1:** List of concrete and steel properties used for defining the cross sectional capacity (i.e. interaction curve)

<table>
<thead>
<tr>
<th>Concrete C20/25</th>
<th>Steel rebars</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_{ck}$ N/mm²</td>
<td>20</td>
</tr>
<tr>
<td>$E_c$ N/mm²</td>
<td>30000</td>
</tr>
<tr>
<td>$\gamma_c$</td>
<td>1.5</td>
</tr>
<tr>
<td>$\alpha_c$</td>
<td>0.85</td>
</tr>
</tbody>
</table>

To this end, the comparison of lining responses from different modeling approaches investigates the influence of two design conditions; the lateral earth pressure coefficient and the height of water level. It was shown in section 4.2 that these two parameters have considerable effects on the bending behavior of the tunnel lining. Four different variations are considered for each parameter ($K_0 = 0.35, 0.42, 0.50$ and $0.67$) and ($\text{water level} = 0.0, -3.0, -7.0$ and $-11.0$). The comparison of results from the different models is considered at the critical cross sections with the maximum absolute moment. The load safety factors are taken as 1.0 and 1.50 for the normal forces and for the bending moments, respectively, since the consideration of a higher safety factor for compressive normal forces within
these specific cases is less conservative. For the evaluation of the reinforcement amount, the factored bending moments and normal forces are plotted in the interaction curves as shown in figure 6.20 for the different values of $K_0$ and in figure 6.21 for the different water levels. Each point indicates the cross section with the maximum absolute moment at the steady state. The circular and diamond points refer to the results from the continuum model using segmental and continuous lining, respectively. Whereas, triangular and square points refer to the results from the bedded beam model using loading assumption (a) and (c) respectively, which represent the upper and lower limits of response, see section 6.3.

Figure 6.20: Bending moment and normal force interaction curve for the evaluation of the reinforcement amount using results obtained from the 3D simulation model in comparison with the bedded beam model considering different $K_0$ values

According to figure 6.20 and figure 6.21, all of the investigated models show the same tendency with respect to the changes in the input parameters. The bending moments increase with the decrease of lateral earth pressure and the decrease of water level as discussed earlier in section 4.2. Considering the segmentation of the lining in the 3D continuum model, the segmental lining model leads to smaller bending moments and similar normal forces when compared with the continuous lining
Figure 6.21: Bending moment and normal force interaction curve for the evaluation of the reinforcement amount using results obtained from the 3D simulation model in comparison with the bedded beam model considering different level of water table.

The difference is more noticeable at $K_0 = 0.35$ and a water level of (-11.0). In figure 6.20, the points representing the segmental lining outputs lie in the gray zone for the minimum reinforcement. On the other hand, the continuous lining model over estimates the bending stiffness and therefore, requires additional reinforcement at $K_0 = 0.35$. In figure 6.21, the difference is more noticeable, in particular at ground water level of (-11.0). The results indicate that the segmental lining model in this scenario requires reinforcement of $5\phi 16/m$, while, the continuous model requires significantly heavier reinforcement of $9\phi 18/m$. With respect to the bedded beam model considering the loading assumptions (a) and (c), the obtained bending moments and normal forces fulfill the safety limits with minimum reinforcement. When compared to the results of the segmental model, the required amount of reinforcement is underestimated at low water level as shown in figure 6.21, since the bedded beam model overestimates the normal forces in the lining. Hence, it can be concluded that such approach does not always provide a conservative design.
In this chapter, the applicability and compatibility of ekate model are enhanced by the incorporation of Building Information Modeling (BIM) concepts. The objective is to provide a tool that reduces the user effort in the pre-and post-processing stages of the model. BIM concepts offer opportunities to streamline and simplify the simulation process by using geometrical BIM sub-models as a basis for performing structural calculations. The sub-models include the existing subsurface structures with different level of detail (LOD) (i.e. surrogate beam-, slab- or a full 3D-models). Hence, the efficiency of the proposed strategy is utilized for performing a multi-stage damage assessment concept adjustable to the necessary LOD. The ground movements are predicted using analytical or numerical approaches, while, damage is being assessed according to strain information or tilt. This enables efficient evaluation of potential damage to subsurface structures associated with various tunnel alignments during planning.

If the predicted damage levels exceeded the acceptable limits, ground improvement techniques should be accounted for in order to control ground deformation. Artificial Ground Freezing (AGF) is an environmentally friendly technique to provide temporary support and groundwater control during underground construction. Evidently, groundwater flow has a considerable influence on the freezing process. Large seepage flow may lead to large freezing times or even may prevent the formation of a closed frozen soil body. For safe and economic design of freezing operations, the simulation of AGF is employed and integrated within an optimization algorithm using the Ant Colony Optimization (ACO) technique to optimize ground freezing in tunneling by finding the optimal positions of the freeze pipes, considering the seepage flow. As demonstrated in the numerical applications of ground freezing in the presence of seepage flow, an optimized arrangement of the freeze pipes may lead to a substantial reduction of the freezing time and of energy costs.
7.1 Coupling Numerical Simulations with BIM Concepts

The use of finite element simulations is an essential part in the design phase of modern tunneling projects. However, these models are often time consuming to construct and require data from many different sources with different formats. Therefore, a Building Information Management based methodologies are proposed in literature to overcome these issues. These methods address the problems generated by decentralized data management by using standardized exchange formats such as the Industry Foundation Classes (IFCs) (ISO 2013) to ensure that a coherent data exchange exists between all models and information sources within a project. BIM models organize data on geometrical and spatial levels and, by modifying IFC’s, are able to easily augment a main model with the project specific elements. Such an element typically consists of a visual component that is linked to the main model geometry and an information component that is linked to the element properties. Information is always accessed through a geometrical model and is intuitively organized. Additionally, BIM concepts are able to address the entire lifecycle of a model, from planning to operation stages, which is critical for highly process oriented projects, such as Tunneling.

Figure 7.1: Coupling of BIM with numerical simulation: components of the BIM model including the ground model, the geological and monitoring data as well as the models for the TBM, the tunnel lining and the buildings (top), and the numerical simulation including the CAD model, the finite element mesh and the simulation results (bottom) (ALSAHY et al. 2018)
Although BIM methods have been originally applied to Buildings, they have also been applied to tunneling projects (Borrman et al. 2015, Hegemann et al. 2012, König et al. 2016, Schindler et al. 2014), which has been referred to as Tunnel Information Model (TIM). In (Schindler et al. 2014), the academic BIM model has been implemented to fit the tunneling projects using data taken from the Wehrhahn-line project in Düsseldorf, Germany. This model includes the tunneling related geometrical models (i.e. tunnel, tunnel boring machine, boreholes, ground and city models), properties, city data, and measurements (i.e. machine data and settlements). Machine data and settlements can be shown by both tabular and geometric representations. Not only does TIM provide a data management platform, but also it allows the user to visually interact with and analyze the data through animations or by sequentially time-stepping through processes.

BIM concepts offer opportunities to streamline and simplify this process by using geometrical BIM sub-models as a basis for performing structural calculations. From which, it is possible to automatically generate finite element simulation models with the required level of detail. Such concepts are capable of automatically incorporating the results of the numerical simulations into a coherent visualization scheme, see figure 7.1.

Figure 7.2: Schematic representation for the BIM-FEM technology for a reference project
TIM concepts are also demonstrated with a numerical simulation of another reference project. The latter is a twin-tube tunnel that passes under an urban area. The outer diameter of the tunnel is 10.97 m and it is driven by two hydroshield machines with an overburden between 4 m and 20 m. Both tunnel tubes have 50 cm thick concrete lining. For simplicity, one region of the project will be modeled under the construction of a single tunnel only. The topology data of the subsoil, the geotechnical properties of the soil layer, and existing structures by means of substitute models for buildings (as a simplified approximation), have been directly included, through the BIM, into the presented numerical simulation. As shown in figure 7.2, the tunnel passes under a steel frame warehouse. The presented case study at this section reveals the merits of the BIM-FEM coupling and shows that it is feasible to conveniently perform an automatic numerical simulation for a tunneling project with minimum user intervention. This section is equipped with a sensor field to monitor the settlements during the construction phase of the tunnel. A comparison between the measured settlements and the predicted ones, perpendicular to tunnel axis, is presented in figure 7.2.

7.2 Multi-stage Assessment of Tunneling-induced Building Damage

The determination of tunnel alignment during planning, a priori accounts for existing surface structures, in particular during tunneling in urban areas where minimizing tunneling induced damage of surface structures is of particular importance (Mark et al. 2012, Neugebauer et al. 2015, Schindler et al. 2016). The assessment of building-soil interaction is crucial in particular for historical and important buildings. If damage is predicted to occur, counter measures could be applied to control ground deformations (i.e. ground improvement or changing the tunnel alignment). The simplest approach of damage assessment uses the analytical equations for settlement prediction without the consideration of building-soil interaction (Peck 1969). The relative displacement with respect to the structure length determines the expected damage according to the structural system (Burland et al. 2001). Generally, such approach leads to a more conservative damage assessment. Therefore, an improved damage assessment is proposed by Franzius (2003) which represents the building as an elastic beam and accounts for the mutual interaction in the settlement prediction. A building’s response to tunnel-induced settlements is indeed a fully coupled 3D problem. Hence, numerical models of the tunneling process and its interaction with the buildings are addressed in recent publications (Bilotta et al. 2017, Burd et al. 2000, Fargnoli et al. 2015, Giardina 2013, Obel et al. 2018a). In which, the buildings are integrated as substitute models or via detailed representation of the main structural components.

In order to reduce work complexity, the models presented in this section are based on BIM concepts to allow for the automatic generation of a numerical model for a tunneling simulation. With the required level of detail of surface structures depending on the level of expected settlements and the structural response with regard to tunneling induced damage. In this context, process oriented simulation, based on BIM concepts, is used to properly evaluate the mutual interaction between surface structures and tunneling process. The staged analysis procedure is presented to provide an approach for the risk of damage assessment during tunneling in urban areas as well as a strategy for
the optimal use of numerical simulations for the damage assessment during tunneling. The analysis performed presents a step forward for a detailed evaluation of the tunnel-building interaction.

### 7.2.1 Concept of Damage Evaluation

The assessment of the tunneling-induced damage to a building requires the definition of three basic steps; surface settlement prediction, representation of building and the method of damage assessment. Each of these steps can be performed with a different level of detail which represents the varying accuracy, see figure 7.3.

![Schematic representation of different level of detail for settlement prediction, building idealization and damage assessment method for the definition of multi-stage damage assessment](taken from OBEL ET AL. (2018a))
This can be summarized according to Obel et al. (2018a) as:

a. Methods of settlement prediction:
   a.1. Analytical ground settlement prediction [green field] (Peck 1969)
   a.2. Numerical ground settlement prediction [green field] (Franzius 2003)
   a.3. Numerical ground settlement prediction with building substitute stiffness (Bilotta et al. 2017)
   a.4. Numerical ground settlement prediction with detailed building representation (Fargnoli et al. 2015)

b. Basics for structural idealization:
   b.2. Design and building data

c. Methods of damage assessment:
   c.1. Slope of relative displacement
   c.2. Beam model with damage related to the critical tensile strains (Burland and Wroth 1975)
   c.3. 2D finite element models of the building facade (Giardina 2013)
   c.4. 3D finite element models of the detailed building (Fargnoli et al. 2015)

For the aforementioned steps, damage assessment can be introduced with an increasing level of accuracy, which can be applied during the early design stage of tunnel alignments in urban environments with numerous buildings. According to the status of the building and the expected damage level, three possible approaches can be performed as:

I. Analytical settlement prediction (a.1) with damage assessment using the limits of relative displacement or beam model with the critical tensile strains (c.1/c.2)

II. Numerical settlement prediction for the green field (a.2) followed by the damage assessment using a separate 2D simulation of the building facade or 3D detailed simulation (c.3/c.4)

III. Numerical settlement prediction including buildings with substitute stiffness or with detailed discretization (a.3/a.4) followed by damage assessment as in (c.3/c.4) (For the detailed building model, damage can be evaluated by the maximum tensile principal strains or by using a particular damage model.)

It can be noted that simple approaches (i.e. I and II) are sufficient for the assessment of non-critical scenarios. While other situations, e.g. masonry structures, important or historical building w.r.t shallow tunnels, require more elaborated and reliable alternatives. The challenge is to balance between time and accuracy of damage assessment to maximize efficiency, e.g. keeping the costs for additional supporting measures as low as possible.
7.2. Multi-stage Assessment of Tunneling-Induced Building Damage

7.2.2 Idealization of Buildings for Damage Assessment

The response of a building is mainly governed by its structural system and the properties of the construction material. For low-rise structures, buildings with different materials can be encountered during tunneling in urban areas (e.g., masonry, wooden, concrete or steel). To realistically capture the overall building behavior, the main load-carrying components have to be identified as well as the foundation system. Generally, the interaction between a structure and the ground is mainly influenced by the flexural stiffness of the building (MAIR 2013). Masonry structures have less overall stiffness compared with concrete/steel structures. In addition, they adapt with surface settlements by introducing minor repairable cracks. However, more evolution of crack pattern leads to loss of structural integrity and failure. Damage occurs when the maximum strain limits are reached due to tunneling-induced deformation.

For the numerical idealization, buildings can be represented by beam, shell or volume elements, in which either simplified linear elastic or nonlinear materials are used. POTTS AND ADDENBROOKE (1997) performed various 2D analysis to investigate the interaction between surface structures and settlements due to tunneling. In that study, the buildings were modeled via beam elements resting on the ground and assumed to have linear elastic properties with no weight. A parametric study was carried out with different building stiffness, size and relative location to the tunnel axis. LIU (1997) and BLOODWORTH (2002) utilized 2D and 3D models to study tunneling interaction with masonry building. The latter is modeled via 2D shell elements which represent the main load carrying components (i.e. vertical brick walls) with no consideration of the flooring system. In addition, simple material behavior has been adopted in their simulation (linear elastic with no tension). Similar concepts are adopted in pertinent literature (BOONPICHETVONG ET AL. 2006, 2003, FRANZIUS 2003, NETZEL AND KAALBERG 2000).

With respect to building damage, BURLAND AND WROTH (1975) related damage to the critical tensile strains. The concept of critical tensile strain is applied to a simple structure (i.e. uniform, weightless, elastic beam). Using the assumption of a circular ground deformation, the beam undergoes two possible extreme modes (i.e. bending mode and shear mode), from which, the maximum tensile strain is determined, see figure 7.4. According to the strain limits, damage classes can be identified as shown in table 7.2. In damage level 2, buildings start to show slight damage. Damage level 3 affects the serviceability limits such that repair works are required and level 4 affects the structural integrity that requires major repair.

<table>
<thead>
<tr>
<th>Damage level</th>
<th>0</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Damage category</td>
<td>negligible</td>
<td>v. slight</td>
<td>slight</td>
<td>moderate</td>
<td>severe</td>
</tr>
<tr>
<td>Strain limits [%]</td>
<td>0-0.05</td>
<td>0.05-0.075</td>
<td>0.075-0.15</td>
<td>0.15-0.3</td>
<td>&gt;0.30</td>
</tr>
</tbody>
</table>

Table 7.1: Damage categories and limiting strains according to BOSCARDIN AND CORDING (1989)


7.2.3 Numerical Example

The multi-stage concept for the investigation of the building-soil interaction and the prediction of expected damage is demonstrated by a numerical example. The investigated tunnel section has the same geological and construction conditions as the one in figure 7.2 and the finite element discretization of this section is shown in figure 2.28. In this scenario, the tunnel passes under 11 residential buildings as illustrated in figure 7.5. The relevant dimensions, properties and the limit values of the substitute stiffnesses are given in (OBEL ET AL. 2018a). A lower bound solution assumes that only the walls perpendicular to the tunnel axis produce an effective bending stiffness (see equation 2.22), and an upper bound can be defined if all the main structural components are included in a shear stiff manner (i.e. adding the floors and foundation stiffness) as shown in equation 2.23.

The simplest approach for damage assessment is primarily utilized. It incorporates the analytical solutions for settlement prediction, and then uses these results as an input for the damage assessment with beam model by checking the limits of critical tensile strains. This resembles a conservative solution, in which, building 3 experience a damage class 3 as it is directly located above the tunnel axis, while other buildings are not in a critical situation. Moderate structural damage is predicted with the beam idealization and therefore further detailed investigations are necessary.
Only building 3 will be investigated with a higher level of detail. Table 7.2 shows the structural idealization of building 3 with the different levels of details. At this level, the use of finite elements models provides a more reliable tool for settlements prognosis. The numerical simulations are performed first for the green field scenario, and then, buildings are included as volume elements with substitute stiffness. Later, only building 3 that is predicted to experience damage is modeled with the highest level of detail. The settlement trough in green field condition, as shown in figure 7.6, is used as an input for a separate 2D model of the masonry facade of building 3 which leads to an improved damage prediction of class 1. The third combination of the prognosis is taken into account by the direct incorporation of the building-soil interaction. The main structural components of building 3 (i.e. walls, floors and footing) are discretized in the numerical model. The resulted maximum principal tensile strains are less than 0.03% and consequently the expected damage is expected to be negligible (damage class 0).

<table>
<thead>
<tr>
<th>Modeling approach</th>
<th>I</th>
<th>II</th>
<th>III</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building idealization</td>
<td>Beam</td>
<td>2D façade</td>
<td>3D detailed model</td>
</tr>
<tr>
<td>Analysis method for settlement prediction</td>
<td>Analytical</td>
<td>Numerical [green field]</td>
<td>Numerical [with building]</td>
</tr>
<tr>
<td>Damage classification for building number 3</td>
<td></td>
<td>Damage level</td>
<td></td>
</tr>
</tbody>
</table>

Table 7.2: Model complexity with different levels of damage assessment
7.3 Simulation of Artificial Ground Freezing

Artificial ground freezing (AGF) is an effective temporary ground improvement technique in geotechnical interventions in soft soils. It is a reversible process with no environmental impact to improve the hydro-mechanical properties (strength, stiffness and permeability) of the soil and to provide a local supporting structure. AGF has several applications in geotechnical engineering including slope stabilization, ground water control and excavation support during underground construction. In tunnel construction in difficult geological and hydrological ground conditions, e.g. in water-bearing soft ground, auxiliary ground improvement measures such as soil grouting or AGF are often applied to provide temporary excavation support and groundwater control. AGF has been commonly used in the last 20 to 30 years as a method to reliably mitigate risks of damage of existing structures during tunnel construction, in particular in tunnel excavation with low overburden in sensitive urban areas, and to efficiently control the groundwater during tunnel advancement (see e.g. HASS ET AL. (1994), JESSBERGER (1980), JOHANSSON (2009), KIRSCH AND RICHTER (2009), PAPAKONSTANTINOU ET AL. (2010), PIMENTEL ET AL. (2012), RUSSO ET AL. (2015), SCHULTZ ET AL. (2008)). When applying AGF in tunneling, a closed arch of frozen ground is formed after a period of time around the excavated area, which provides a protected area for the excavation of the tunnel cross section.

The ground freezing process converts pore water into ice by withdrawing heat from the soil. Depending on the coolant, there are mainly two types of AGF in use: brine freezing and liquid nitrogen (LN₂) freezing. In general, the time to establish a desired thickness of a frozen soil body with full temporary load carrying capacity depends on the type of coolant used in the freezing process and on the freeze pipes in terms of size, number and spacing. Moreover, the required freezing time is considerably influenced by the presence of seepage flow, since the flow provides a continuous source of heat. In case of large seepage flow, a state of thermal equilibrium can be reached, in which freezing stops and the closure of desired frost wall cannot be developed. Evidently, as an important indicator for energy consumption and hence operating cost, the required freezing time increases substantially with increasing seepage velocity.

For a safe and economic geotechnical design and construction, a reliable prediction of the coupled thermo-hydraulic behavior of the soil during freezing is required. By adopting the theory of thermo-poromechanics and the theory of premelting dynamics, ZHOU AND MESCHKE (2013) developed a three-phase finite element model for the description of coupled thermo-hydro-mechanical behavior of freezing soils. In this numerical model, solid particles, liquid water and crystal ice are considered as separate phases (see figure 7.7), and the mixture temperature, liquid pressure and solid displacement as primary field variables. Through three fundamental physical laws (overall entropy balance, mass balance of liquid water and crystal ice, and overall momentum balance) together with corresponding state relations, the model captures the most relevant couplings between the phase transition associated with latent heat effect, the liquid transport within the pores, and the accompanying mechanical deformation. Herein, since the ground freezing operation will only be investigated as a means for groundwater control during tunneling, with more focus on the influence of groundwater flow on the formation of the required frozen arch. The displacement field will be neglected for the sake of simplification (i.e. a coupled thermo-hydraulic freezing soil model is deduced from the original numerical model and will be used for the simulations presented herein).
7.3. SIMULATION OF ARTIFICIAL GROUND FREEZING

Considering the high energy costs connected with soil freezing, there is a strong economic interest to minimize the time needed to establish a fully frozen soil body with the desired dimensions, considering the influence of seepage flow. With this background, the main focus is on the optimization of the arrangement of freeze pipes during ground freezing in tunneling in the presence of seepage flow. Among the few available publications concerned with optimization of AGF operations in tunneling, ZIEGLER ET AL. (2009) have presented two optimized placements of freeze pipes, both of which have showcased a significant reduction in the freezing time with two extra freeze pipes. In contrast, the goal of this section is on the investigation of optimal pipe placements considering different levels of seepage flow, however, without increasing the number of pipes.

7.3.1 Numerical Simulation of Artificial Ground Freezing

The computational model for soil freezing described in the previous section is applied to the numerical simulation of AGF for the temporary ground support during tunneling. A case study of AGF performed by ZIEGLER ET AL. (2009) is re-analyzed numerically by the proposed freezing soil model. To obtain an arch of frozen soil with high load bearing capacity and impermeability, freeze pipes with a fixed temperature of $-35\,^\circ C$ and a diameter of 0.2 m are installed horizontally in a soil layer initially at an in-situ ground temperature of $13.45\,^\circ C$, which rests on an impervious base (depth: 10 m, width: 38 m) (see figure 7.8). The inner surfaces, where the freeze pipes are located, remain immobile and undrained. According to the geotechnical requirements, eventually a circular frozen arch with a thickness of $\approx 1.5$ m is desired.

Figure 7.8: Numerical simulation of AGF in tunneling: Geometry and dimensions of the problem (material parameters involved in the freezing soil simulation are provided in (MARWAN ET AL. 2016))
7.3.2 Influence of Seepage Flow on Frozen Arch Formation

The influence of horizontal seepage flow on the formation of a frozen arch wall is investigated numerically by means of the proposed computational model. Horizontal seepage flow is simulated by applying a constant positive water pressure on the left boundary and zero pressure on the right boundary. According to the model geometry in figure 7.8 and the material parameters employed in this case study, different levels of horizontal seepage flows are assigned on the horizontal boundary.

![Figure 7.9: Influence of seepage flow on the formation of a frozen arch for an equidistant distribution of freeze pipes](image)

Figure 7.9 compares the spatial distribution of the temperature obtained from numerical simulations for three different levels of the seepage flow ($v_L = 0$, 0.5 and 1.0 m/d) after 3, 6 and 9 days. Since groundwater flow provides a continuous source of heat, the freezing process is considerably affected and the formation of a closed frozen arch around the tunnel profile is delayed under a relatively high seepage flow. Initially, water flow is almost homogeneously passing through the entire cross section. As the frozen soil columns grow, the flow velocity increases considerably within the gaps between the freeze pipes, which inhibits the formation of a closed arch by delaying the connection between adjacent frozen pipes. Once the frozen arch is closed, there is no more water flow within the interior of the frozen arch and, consequently, the impact of seepage flow on the temperature evolution is significantly reduced. From then on, the frozen arch grows much faster towards the inwards direction than outwards and the desired thickness of $\sim 1.5$ m is reached. It is worth to mention here that in the presented tests, the desired frozen arch is said to be achieved if the computed values of temperature at all nodes of the desired arch boundary are lower than $-3^\circ$C. To generate an arch of frozen ground with a thickness of $\sim 1.5$ m required to support the excavated tunnel cross section, the required duration for three investigated scenarios for different seepage flows ($v_L = 0$, 0.5 and 1.0 m/d are computed as 10, 21 and 53 days, respectively. According to the analysis, at higher seepage flow (e.g. 1.5 m/d), the desired frozen arch cannot be achieved even after long time (see also (SCHULTZ ET AL. 2008)). This is attributed to the fact, that a state of thermal
equilibrium has been reached in this system and hence the soil stops freezing, see the temperature distribution in figure 7.10 after 3, 6, 9 and 16 days.

![Temperature Distribution](image)

**Figure 7.10**: Formation of the frozen arch with seepage flow of $v_L = 1.5 \text{ m/d}$ for an equidistant distribution of freeze pipes

### 7.3.3 Optimization of Freeze Pipe Arrangement

To this end, the multi-field finite element model for the numerical modeling of the freezing process is connected with a suitably designed optimization algorithm. For the highly nonlinear, multidimensional problem at hand, meta-heuristic methods have significant advantages as compared to gradient based methods (Hoos and Stuetzle 2004). Instead of computing the gradient or Hessian matrix of the objective function, stochastic approaches are used in meta-heuristic approaches. This significantly increases the ability to find optimal or near optimal solutions specially for complex problems with multiple local minima. Within the family of meta-heuristic optimization methods a number of specific algorithms such as Simulated Annealing, Tabu Search, Genetic Algorithms, Ant Colony Optimization and Particle Swarm Optimization have been developed (see e.g. Boussaid et al. 2013, Glover and Kochenberger 2003 for an overview). Ant Colony Optimization (ACO) is a probabilistic technique which aims to search the optimal path within a graph. Inspired by the foraging behavior of ants, this approach mimics the behavior of the ants seeking a path between their colony and a source of food (Dorigo and Blumb 2005). Ants use pheromones as a communication medium when searching for food. Similarly, ACO uses artificial pheromone trails as an indirect communication tool. The pheromone trails and its update scheme serves as numerical information that improves the search probability to select optimal solutions.

The first algorithm for ACO was proposed in the early 1990s as a novel technique for solving difficult combinatorial optimization problems (Dorigo 1992, Dorigo et al. 1996). Subsequently, different algorithms including ant colony system (Dorigo and Gambardella 1997) and Max-Min Ant System (Stuetzle and Hoos 2000) were introduced. Later, ACO was extended to solve multi-objective optimization problems. Multi-objective ACO is composed of an underlying ACO algorithm plus specific algorithmic components to tackle multi-objective optimization. This can be
achieved by different fashions such as using different pheromone matrices for each objective or using multi-colony approach with one colony for each objective (Iredi et al., 2001, Lopez-Ibanez and Stuetzle, 2012). ACO was also adopted to continuous optimization problems. Socha and Dorigo (2008) proposed ACOR; the most popular ACO algorithm for continuous domains. Extensions of ACOR to Diverse ACO (DACOR) and Incremental ACO (IACOR-LS) were proposed in (Leguizamon and Coello, 2010, Liao et al., 2011). DACOR uses the same basic principle of ACOR. However, it generates new solutions by considering an alternative approach to select probabilities for producing solutions. IACOR-LS is an ACOR with an extra search diversification mechanism and a local search procedure that enhance its search intensification abilities. Recently, a unified algorithm, which includes the previous algorithmic components for continuous optimization with ACO, was presented in (Liao et al., 2014).

A successful implementation of a meta-heuristic search shall provide a balance between the exploration and the exploitation. Exploration achieves diversification; it aims to efficiently explore the whole search space. Exploitation means intensification as it searches around current best solution to find better solutions. Less exploration with much exploitation could trap the algorithm in a local optimum. In contrast, more exploration with less exploitation could reduce the algorithm performance and efficiency. In ACO, such a balance can be typically maintained by the proper management of the pheromone trails. This is achieved by the appropriate choices for the pheromone trail update scheme to improve the diversity of the search (Stuetzle and Hoos, 2000). ACO was successfully applied to different problems and it is increasingly gaining interest for solving engineering and scientific problems such as design and optimization of laminated structures, analysis of water resources systems, optimization of computer systems and optimization of traffic signal timings as shown in (Ostfeld, 2011).

In this section, the Ant Colony Optimization algorithm is connected to the multi-field finite element model for ground freezing (Zhou and Meschke, 2013) in order to find optimal positions of freeze pipes in applications of artificial soil freezing in tunneling such that a minimum freezing time required to establish a fully frozen soil arch around the tunnel cross section is obtained. For a given seepage velocity, the optimized solution presents a cost effective pipe arrangement for ground freezing in tunneling with minimal energy consumption.

**Ant colony optimization**

The ant colony optimization (ACO) algorithm is a probabilistic technique which aims to search the optimal path in a graph by mimicking the behavior of ants seeking a path between their colony and a source of food. The ants deposit pheromone trails on the ground to mark food paths where these trails should be followed by other ants of the colony (Goss et al., 1989). Several algorithms were proposed in the literature (see Dorigo (1992), Dorigo and Gambardella (1997), Dorigo et al. (1996), Socha and Dorigo (2008), Stuetzle and Hoos (2000) for an overview).

The meta-heuristic search basically consists of an initialization step for the pheromone levels, and the iterative construction of solutions with the update of the pheromone table in order to represent the cumulative experience of the ant colony. The scheduled operations of selecting solutions,
The variables involved in the optimization problem are partitioned into a finite set of components, and the combinatorial optimization algorithm attempts to find their optimal combination or permutation. For a $N$-dimensional problem, the solution space can be represented as a graph (see figure 7.11), in which each node represents an individual partial solution. The user defines a number of artificial ants and each ant constructs its own solution from the available solution components $C_{ij}$ (with $i = 1, \ldots, N$ and $j = 1, \ldots, \text{size}(D_i)$), where $X_i$ and $D_i$ represent the variables and the set of discrete values attached to it. The algorithm is initiated by setting initial levels of pheromones to the pheromone table. The size of pheromone table is associated with the size of solution components. After setting the pheromone table, the algorithm starts the loop by constructing the solution.

Figure 7.11: Ant Colony Optimization: Illustration of discrete solution space, of note is that each column resembles one variable including the set of attached discrete values

ACO is an iterative algorithm, where the artificial ant is a simple computational agent. A walking ant on the graph, figure 7.11 simulates the solution selection process. At each iteration, each ant moves from a solution state to another solution state creating a partial solution until it constructs the complete solution. Constructing a solution is similar to searching for a food source, then the artificial ants return back to the nest, evaluate the results (food quality) and exchange information on the solution quality by means of the pheromone update. This selection process is achieved stochastically with a probability of:

$$P_{ij} = \frac{\tau_{ij}}{\sum_{k=1}^{\text{size}(D_i)} \tau_{ik}},$$

where $\tau_{ij}$ is the pheromone level of solution component $C_{ij}$. In the first iteration, the probabilities for all solution components are equal. Subsequently, the artificial ants evaluate the solutions and a predefined number of overall best solutions are used for updating the pheromone levels.

After evaluation of constructed solutions by the artificial ants, ants deposit pheromone along their paths on the graph. Pheromone update is the most important operation in the algorithm since pheromones are the communication tool between ants and adequate pheromone update leads to optimum solutions. The aim of pheromone update is to increase the pheromone levels for improved solutions and decrease it, if the solution is not improved. This can be achieved in two steps. First,
all pheromone trails for all edges are uniformly decreased by a factor called pheromone evaporation. Pheromone evaporation is needed to avoid rapid convergence and enhance the possibility of selecting new areas in the search space. In the second step, the pheromone trials for good solutions are reinforced in order to increase the probability of subsequent ants to select promising regions of the search space. The added amount of pheromone depends on the quality of the solution.

The pheromone update follows an algorithm denoted as Ant System. Variants of the original Ant System algorithm proposed by DORIGO ET AL. (1996), are the Max-Min Ant System and the Ant Colony System (DORIGO ET AL. 2006, STUETZLE AND HOOS 2000). In the Ant System, the pheromone values are updated at the end of the iteration by all ants in that iteration. In the Max-Min Ant System only the "best ants", representing the best solutions, will update the pheromone trails. The respective pheromone values are bounded. The definition of "best ants" is subjected to the designer decision; it can be considered as the best ant in the current iteration or the best ant since the start of the algorithm or a combination of both.

Ant colony algorithms were originally employed for discrete optimization. However, different attempts were developed to tackle continuous domains (SOCHA AND DORIGO 2008). In this work, a new approach is used to calculate the additional pheromones in a manner that suits the continuous domain. The amount of added pheromone depends on the quality of the solution which can be evaluated using a fitness function or a scaled fitness function. Such an evaluation, however, often has the problem that the presence of extreme fit values will dominate the added levels of pheromones, which may lead to a premature convergence. Therefore, in the proposed ACO algorithm, the solution is evaluated using ranking of the objective function to overcome the premature convergence problem associated with raw fitness and scaled fitness. A predefined number of best ants are stored according to their rank. The ranking is assessed with natural numbers starting with rank one for the worst solution to the highest rank for the best solution. The incremental rank-based pheromone added to the corresponding discrete solution component reduces the algorithm’s efficiency and the ability to converge. To overcome this problem, it is proposed that each ant deposits the additional pheromone not only on the selected edge but also to the adjacent edges according to a normal distribution. The standard deviation (σ) is an input variable assigned by the user and the mean is taken as the position of the solution component $C_{ij}$ selected by the best ants. The added amount of pheromones is updated by the predefined set of best ants according to

$$\Delta \tau_{ij}^{best} = \sum_{k=1}^{N} \left( \frac{R_k - 1}{N - 1} \right) \left( \frac{1}{\sigma \sqrt{2\pi}} \exp \left( \frac{-(j - \text{mean})^2}{2\sigma^2} \right) \right), \quad (7.2)$$

where $N$ is the total number of best ants, and $R_k$ is the solution rank of the $k^{th}$ best ant. The pheromone update scheme in iteration $t + 1$ is defined as:

$$\tau_{ij}^{t+1} = (1 - \rho) \tau_{ij}^t + \rho \Delta \tau_{ij}^{best}. \quad (7.3)$$

The first part in equation 7.3 represents evaporation with an evaporation ratio $\rho$ and the second part represents the additional amount of pheromone deposited by the best ants. The standard deviation $\sigma$ defines the shape of the probability distribution. When $\sigma$ is chosen small, pheromones are distributed
within a narrow area and convergence is achieved rapidly. Larger values lead to a more uniform probability distribution, providing more diversity to the search space, which is connected with a slower convergence. The variance $\sigma$ must be chosen larger than $1/\sqrt{2\pi}$. For $\sigma = 1/\sqrt{2\pi}$, each ant will update only a single solution component, which is considered as inappropriate for continuous domain.

As can be concluded from the previous subsection, a large seepage flow will considerably delay or even prevent the formation of a closed frozen arch around the tunnel profile during freezing process. In such a case, the frost zone around the freeze pipe does not form concentrically around the freeze pipes. An oval shaped area toward the downstream side is obtained (see figure 7.12) since groundwater heat is transported via advection by the flow to and around the freeze pipe. It is evident, that an equidistant distribution of the freeze pipes is not the optimal solution in case of presence of seepage flow. If groundwater flow is not adequately accounted for in the design of the freezing operation, the success of the freezing process may be endangered when a steady state is reached without forming a closed frozen arch. Therefore, the ACO optimization algorithm described herein has been employed to improve the freezing efficiency by searching for the optimal location of freeze pipes depending on the direction and magnitude of the groundwater flow.

**Figure 7.12:** Oval frozen zone developing during soil freezing in presence of ground water (JUMIKIS 1979)

---

**Parameterization of the optimization problem**

The positioning of $N$ freeze pipes could be parametrized by the radial coordinates $(R_i, \theta_i)$ with $i = 1, 2, \ldots N$, and a discrete optimization could be explicitly performed over these coordinates. However, considering that this requires $2 \times N$ optimization parameters, two distribution functions are defined for positioning the freeze pipe to reduce the dimension of the problem. The optimization algorithm searches for the optimum parameters defining the proposed functions.

The first distribution function is a transition function, $\delta R(\theta_i)$, which determines the radii of the freeze points. A comparison between the developed frozen zone in the absence and presence of the ground flow allows for a qualitative identification of this transition function (see figure 7.13).

The transition is assumed to have a linear variation with $\theta$ on the downstream side and to be constant on the upstream side (figure 7.14). The transition function is parameterized by the two
parameters \((d_1, d_2)\) defining the radial shift of the two freeze pipes located at the invert (see figure 7.14). These parameters are ranging from 0.0 to 0.5 m and divided into 21 solution components, allowing for a resolution of 0.025 m for finding optimal positions. The freeze pipe radius is be determined by

\[
R_i = R_0 + \delta R(\theta_i); \quad i = 1, 2, \ldots N, \tag{7.4}
\]

where \(R_0\) is the average radius of the frozen arch and the transition function \(\delta R(\theta_i)\) is defined as

\[
\delta R(\theta_i) = \begin{cases} 
(d_1 + d_2) \frac{2\theta_i}{\pi} - d_1, & \text{if } \theta_i < \frac{\pi}{2}; \\
 d_2, & \text{if } \theta_i \geq \frac{\pi}{2}.
\end{cases} \tag{7.5}
\]

During the formation of a frozen zone, it can be noted that in the presence of ground flow the frozen zone around pipes starts to merge at the crown position, see figure 7.9. Therefore, a spacing function \(\delta\theta\) is introduced to allow a change of the position of the freeze pipes also in tangential direction. The proposed spacing function is assumed as a normalized Gaussian distribution. It increases the angle between the pipes located at the top of the arch and decrease it at both sides. The spacing function is parameterized with the mean \((\mu)\) and standard deviation \((\sigma)\) of the Gaussian distribution. The angle of the \(i\)-th freeze pipe is determined by

\[
\theta_i = \begin{cases} 
0, & \text{if } i = 1; \\
\theta_{i-1} + \delta\theta_{i-1}, & \text{if } i > 1.
\end{cases} \tag{7.6}
\]

in which, \(\delta\theta_{i-1}\) is defined as

\[
\delta\theta_{i-1} = \frac{\pi G(i - 1, \mu, \sigma)}{\sum_{j=1}^{N-1} G(j, \mu, \sigma)}, \tag{7.7}
\]

**Figure 7.13**: Frozen arch profile: (a) in the absence and (b) in the presence of seepage flow \((u_L = 1.0 \text{ m/d})\)

**Figure 7.14**: Transition function \((\delta R(\theta))\) for the determination of the radii of the freeze points
with \( G(x, \mu, \sigma) \) representing the Gaussian distribution

\[
G(x, \mu, \sigma) = \frac{1}{\sigma \sqrt{2\pi}} \exp\left(-\frac{(x - \mu)^2}{2\sigma^2}\right). 
\]

(7.8)

In the optimization algorithm, the mean \( \mu \) ranges from 2 to 8 with an increment of 0.25, while the standard deviation \( \sigma \) ranges from 2.75 to 10 with increments of 0.25. For practical implementation, the values of the last two parameters are constrained in such a manner that the minimum angle \( \delta \theta \) between two adjacent pipes should exceed 10 degree.

**Optimization results**

The optimization problem is a four dimensional problem with \( 21 \times 21 \times 25 \times 30 = 330750 \) possible solutions in accordance with the input variables. The selected number of artificial ants is 6. The termination criterion for the algorithm is the maximum number of iterations, which is predefined according to the seepage velocity under investigation.

The optimization procedure was separately applied for different seepage velocities. The results for the freezing time obtained from the optimization of the locations of the freeze pipes for four different seepage velocities \( (v_L = 0.5, 0.75, 1.0 \text{ and } 1.25 \text{ m/d}) \) are summarized in table 7.3. For comparison, the respective freezing times obtained from the numerical simulation without optimization, i.e. with an equal distribution of the pipes are included. In figure 7.15, both results for the required freezing time are plotted in a diagram as a function of the seepage velocity.

<table>
<thead>
<tr>
<th>Seepage velocity ( v_L ) (m/d)</th>
<th>0.50</th>
<th>0.75</th>
<th>1.00</th>
<th>1.25</th>
</tr>
</thead>
<tbody>
<tr>
<td>Freezing time (days) with equal arrangement</td>
<td>21</td>
<td>27</td>
<td>53</td>
<td>( \infty )</td>
</tr>
<tr>
<td>with optimized arrangement</td>
<td>9.67</td>
<td>10.50</td>
<td>10.33</td>
<td>15</td>
</tr>
</tbody>
</table>

Table 7.3: Total freezing time required for obtaining a fully frozen arch at different seepage flows with equidistant and optimized arrangement of the freeze pipes.

The results show, that an arrangement of the freeze pipes determined by the optimization algorithm significantly reduces the freezing time. With increasing seepage velocity, the freezing time increases progressively in case of an even distribution of the freeze pipes, while only a moderate increase is observed if an optimal placement is chosen. The larger the flow velocity of the groundwater, the larger is the improvement from the optimization procedure. While for \( v_L = 0.25 \text{ m/d} \), the ratio of the freezing time with and without optimization is approximately 81\%, it decreases to 39\% for \( v_L = 0.75 \text{ m/d} \). While for an even distribution the formation of a frozen arch with the desired thickness is prevented for a seepage flow beyond a critical level of \( v_L \approx 1.5 \text{ m/d} \), the optimization procedure leads to relatively short freeze times in the range of \( \approx 20 \) days for this case.

It is interesting to note, that the plot obtained for the optimized locations of the freeze pipes shows a decrease of the freezing time for small levels of the seepage velocity \( v_L < 0.5 \text{ m/d} \) before the freezing time increases with increasing levels of \( v_L \) (see Figure 7.15). Obviously, the existence
of small seepage flow velocity slightly promotes the freezing process as compared to the case of equal arrangement with no seepage flow. This is attained due to a more optimal arrangement of the freeze pipes resulting in a (non-circular) freezing area around each pipe (see figure 7.12) which is better suited to form the desired freezing arch.

To evaluate the performance of the algorithm, the convergence history for $v_L = 1.0$ m/d is shown in figure 7.16. It also shows the gradual improvements of the average best results and it is expected that the average best results will further approach the best result as iterations increase. The results demonstrate the efficiency of the ant colony algorithm as it provides an optimum solution after approximately 40 iterations. Within each iteration step six numerical analyses are performed. Hence, in total, approximately 240 realizations were necessary. This is less than 1% of the total solution space.
The optimum arrangement of freeze pipes is presented in figure 7.17 for \( v_L = 1.0 \text{ m/d} \). It is noted, that the pipe locations are shifted against the flow direction and that the spacing is decreased at the upstream direction. For \( v_L = 1.0 \text{ m/d} \), the optimized solution requires a freezing time of 10 days to form a fully frozen arch. This must be compared to the original design with an equidistant placement of the freeze pipes, for which more than 50 days of freezing are required. Figure 7.18 shows the formation of the frozen arch at different stages of the freezing process for an optimized placement of the pipes for a seepage velocity of \( v_L = 0.5 \text{ m/d} \) and \( v_L = 1.0 \text{ m/d} \), respectively. When compared with figure 7.9, one observes, that the optimum arrangement provides a more symmetric and homogeneous growth of the frost body as compared to an equidistant arrangement of the pipes.

**Figure 7.17:** Optimum arrangement of freeze pipes for \( v_L = 1.0 \text{ m/d} \) in comparison with an equal distribution of the pipes

**Figure 7.18:** Influence of seepage flow on the formation of a frozen arch for an optimized arrangement of the freeze pipes.
Chapter 8

Summary, Conclusion and Outlook

8.1 Summary

In the first part of this thesis, selected aspects of mechanized tunneling process are introduced as a basis for simulating the shield supported tunnel construction process. The literature provides different computational models with various assumptions and modeling schemes. The effectiveness of these models depends on its capability to replicate the real conditions. In this context, the simulation model \textit{ekate} is used for the process oriented analyses, in which, the shield advancement is realized by means of an automatic steering algorithm. For an automatized process for the generation and execution of finite element simulations, the simulation model has been incorporated into the so-called Tunneling Information Model (TIM), which is a Building Information Modeling (BIM) based product model that has been developed in the context of the German Research Foundation funded Collaborative Research Center 837. The applicability and the effectiveness of the proposed framework to automatically generate a fully 3D finite element model have been demonstrated by means of two reference projects. The presented case studies reveals the merits of the proposed approach and shows that it is feasible to conveniently perform an automatic numerical simulation for a tunneling project with minimum user intervention.

With respect to segmental tunnel lining analysis, traditional analysis methods generally use simplified structural models that attain simplified loading assumptions. Consequently, such models can only serve to provide first-order estimates of the structural forces in tunnel lining. A review of the structural models that are used in design is provided, as well as the various loading assumptions that leads to a range of design structural forces. Throughout the discussion presented in this work, it becomes apparent that more detailed models need to be considered in order to accurately identify the underlying factors that control segmental lining response and to be able to determine which simplifications can be made without influencing the model accuracy. To this end, a detailed analysis of segmental tunnel lining is performed within the simulation model \textit{ekate} to accurately account for the actual loading on the lining. In addition, the interactions between the segments at the longitudinal and ring joints, are modeled by means of frictional contact to better capture the lining
kinematics. Furthermore, the mutual interactions between the segments along the joints, the interactions of the lining with the grouting material as well as the surrounding ground are accounted for realistically. The consideration of the initially fluid like grouting material and its hydration induced stiffening, as well as the consolidation processes of the saturated soft soils, enhances the capability of the model to provide realistic insights in the spatio-temporal loading on the tunnel linings during the simulation of the mechanized tunneling process.

Finally, extension to the use of the computational framework is demonstrated by advanced applications that are occasionally encountered during tunneling. The simulation model is used to evaluate the tunnel-building interaction. This is of great interest in urban tunneling, especially when historical or other important buildings are influenced. To this end, an approach for the damage assessment of the existing surface structures, considering different level of details of their structural components, is presented. This approach provides a flexible damage assessment concept that can be adopted during the planning phase. For the case of high risk of building damage, additional countermeasures are required. In order to address this issue in the framework of the numerical simulations, a coupled thermo-hydraulic formulation is introduced to model Artificial ground freezing. In addition, the influence of groundwater flow on the freezing arch formation is examined and an optimization solution for the freezing pipes’ position is provided based on the Ant Colony Optimization.

8.2 Conclusion

8.2.1 Segmental Lining Analysis

The simulation model has been used to predict the ground deformation and loading on lining, as well as the normal forces and bending moments induced by the construction process and the surrounding soil. Results from the parametric study in chapter 4 have shown that the model is able to provide detailed insights into the loading during construction with respect to various geotechnical and constructional aspects. It was demonstrated that the steering gap resulted from shield overcut and conicity have a significant influence on the redistribution of stresses in the tunneling vicinity and therefore, the predicted loads and the structural forces. It should be noted that such geometrical parameters can not be considered in the classical lining models. The ground water level and the lateral earth pressure coefficient affect the lining response, in particular the predicted bending moments. In general, based on the model assumptions, these lining forces and deformations are obtained from a simulation procedure that stays in line with the actual construction process.

With respect to joint response, a penalty based frictional contact algorithm was used to describe the contact between consecutive rings and between segments, this provides an explicit representation of lining joints. The proposed computational model was successfully validated by numerical simulations of a single joint and a full ring test. It was shown that the classical contact mechanics can be used successfully to characterize the lining joints.

By the developed segmental lining model in eKATE, it was shown that the application of grouting pressure at the tail of the machine provides a hydrostatic state of stresses, which provides sufficient compressive forces in the lining leading to a higher joint stiffness. With the advancement of the shield and the erection of new lining rings, the dissipation of the grouting pressure behind
the shield is compensated by the hydration of the grouting mortar. The hardened mortar not only increases the water tightness of the lining but also confines the segmental rings in a more stable state. Consequently, the rotational stiffness of the joint increases significantly as compared with the single joint behavior and the moment-rotation relationship for the longitudinal joints in segmental lining model does not follow the analytical solution, instead it provides a stiffer response. This is an outcome of the inclusion of the grouting and soil medium as a soil-structure interaction problem in the proposed computational model.

Results from a parametric study have shown, that the normal ring forces increasing with tunnel depth implicitly lead to higher joint stiffnesses and that the confinement provided by the surrounding grouting material leads to higher joint stiffnesses as compared to analytical models, in which this confinement effect is not considered. From a numerical study of the effect of different joint arrangements, characterized by 7 segments, it was demonstrated, that staggered arrangements of longitudinal lining joints provide an additional constraint along subsequent rings which increases the lining stiffness and increases the maximum bending moments. The joint orientation only slightly affects the maximum moments in the segmental ring in the considered scenario. However, it determines the distribution of the bending moments and the location of maximum moments and the moment reduction at the joints. Evidently, the effect of the joints is larger, when it is located near the zones of maximum moments, i.e. crown, springline and invert.

A comparison of the proposed segmental modeling strategy with the often used approach to model linings as a continuous, fully connected shell in the context of 3D tunnel advancement analyses has highlighted, that the maximum bending moments are considerably overestimated. Frictional contact between lining segments provides a straightforward method for the inherently discontinuous deformation. It is thus able to capture the shear-coupling mechanism in the ring joint between consecutive linings and allows for the prediction of possible joint openings. In comparison with the classical bedded beam models, a major advantage of the proposed model is that it does not require a priori loading assumptions. The obtained loading conditions are resulting directly from the interactions taken into account in the 3D tunnel advancement model. It was demonstrated, that due to the overestimation of normal forces in the bedded beam model, it is not always providing a conservative design approach.

8.2.2 Tunneling-induced Building Damage

A method for the numerical assessment of the risk of damage due to tunneling-induced settlements is introduced by means of a multi-staged procedure. In the first stage, all buildings, that are influenced by the tunneling activity, are checked with a simplified damage assessment. Here, all buildings can be characterized according to the expected damage level employing the proposed conservative approach. In the next stages, an improved numerical prediction of ground deformations and strains are adopted for a more reliable damage prediction. This reduces the computational costs by adopting the sufficient level of detail according to the predicted damage level and the status of the building. At the current state, the simulation model allows to perform a detailed building discretization including building’s weight and the stiffness of the main structural components. Such detailed modeling approach provides a powerful tool for the damage assessment, particularly in complex
scenarios where analytical methods are not sufficient, such as for complex geology or presence of strengthening measures.

8.2.3 Simulation of Artificial Ground Freezing

A three-phase finite element model for freezing soils has been applied to the artificial ground freezing process during tunneling. It has been shown, that the presence of ground water flow has a considerable influence on the formation of a closed and stable arch of frozen ground. The numerical model was integrated within an optimization algorithm using the Ant Colony Optimization (ACO) technique to optimize the freezing process by finding the optimal positions of the freeze pipes in the presence of seepage flow. The proposed ACO has proven to be very effective in obtaining an optimized arrangement of freeze pipes for different seepage velocities. The presented numerical application of the multiphase model together with the ACO method to a soil freezing operation for a tunnel has shown its efficiency in reducing the freezing time significantly and hence the energy consumption as compared to an equal spacing of freeze pipes. It was further demonstrated, that even for above-critical seepage flow, above which a thermal equilibrium state has been reached and hence the required frozen arch cannot be achieved with an equally spaced pipe arrangement, a stable closed arch can be formed by optimizing the location of the freeze pipes.

8.3 Future Work

With the modular design of the computational framework and the object-oriented implementation of the finite element code **KRATOS** (DADVAND ET AL. 2010), further developments are of interest to be integrated and investigated in the scope of the simulation model **ekate**. This can be summarized as:

- **Modeling the segment installation procedure and the sequential loading during shield advancement:**

  In the present work, the segment-wise modeling of mechanized tunnel lining is proposed. A possible extension of the model can be directed to the investigation of lining response during assembly stage. This can be achieved by the step-wise activation of each segment while retrieving the respective hydraulic jacks to realistically replicate the ring erection. The goal of this simulation is to gradually build the lining segments inside the shield and check, if the initial stresses developing during erection affects the ring response after it leaves the shield.

  The simulation model can also be utilized to examine the lining response during the shield movement. With respect to the ring leaving the shield, the main aspects which control the ring deformation, are the thrust forces, the sealing pressure and the gradually applied annular gap grout pressure. In this situation, the incremental development of hoop stresses within the longitudinal joints, as well as the ring coupling behavior with respect to the sequential compression can be assessed. Modeling such situation in a process oriented model poses extra difficulties, i.e. annular gap has to be finely discretized for sequential activation.
8.3. FUTURE WORK

- **Integration of AGF into the process oriented simulation:**
  Incorporating the three-phase formulation for describing the coupled thermo–hydro-mechanical behavior of freezing soils into the process-oriented simulation model will provide the basic framework for the investigation of the influence of artificial ground freezing in urban underground construction projects. Within such model, it would be possible to have a realistic simulation of the overall situation during the formation of the frozen arch and during melting. With the existence of seepage flow, the model can predict whether a complete frozen body will be formed or not, which turns to be of great interest to avoid project delays. Further investigation of the effect of such processes (i.e. freezing and melting) on the response of the tunnel lining is of interest as well.

- **Simulation of the infiltration process:**
  The infiltration process usually occurs during mechanized tunneling due to the use of pressurized fluids for supporting the soil. Nevertheless, the realization of the infiltration process in numerical simulation of mechanized tunneling is usually overlooked. To this end, the simulation model in (LAVASAN ET AL. 2017) accounts for the permeability change only due to the grouting mortar infiltration at the tail of the shield machine by using a meso-scale sub-model. While in (ZIZKA ET AL. 2016), finite element transient seepage analysis (no mechanical deformations are considered) is presented to assess the pressure transfer mechanism at the tunnel face by using a time-dependent permeability coefficient according to excavation parameters of the cutter head. Generally, these models require a high spatial resolution with sufficiently small element size at the excavation face along tunnel axis and at tunnel’s circumference along the radial direction in order to account for the abrupt reduction of permeability due to infiltration. Thus, high computational efforts are needed. The use of re-meshing techniques could be introduced to overcome this problem.
Appendices
Appendix A

Calculation of Cylindrical Stresses and Member Forces for Arbitrary Alignments

In order to provide design relevant parameters for lining design, structural member forces (i.e. normal forces and bending moments) must be extrapolated from the volume elements. These are calculated by first transforming the stresses to a cylindrical coordinate system defined along the width and the radius of each segment, and then by integrating stresses along the cross section of the segment. This process is performed at the gauss points of the volume elements used to represent the lining segments, where, it is assumed that the center of the lining thickness corresponds to its neutral axis.

A.1 Stress Transformation to Cylindrical Coordinate System

The output stresses at the Gauss points of lining elements are transformed from a Cartesian coordinate system, defined as $\mathbf{e} = \{e_x, e_y, e_z\}$, to a cylindrical coordinate system, defined as $\tilde{\mathbf{e}} = \{l, r, \theta\}$. In tunnels with straight alignments, a single cylindrical coordinate system can be used for each lining ring. The reference longitudinal axis is chosen to be the longitudinal axis of the tunnel along its center line, and the polar axes are defined by the radius and the angular position measured in clockwise direction from the tunnel crown. On the contrary, if the path of the tunnel is curved, a unique cylindrical coordinate system must be defined for each consecutive ring within the lining. In this context, each ring has its own cylindrical coordinate system in which the longitudinal axis is chosen to be perpendicular to the ring joint, parallel to the straight longitudinal joints, and passing through the center point of the ring. The polar axes are defined similar to that of the straight alignment ring. For this purpose, a special python utility, StructuralForcesUtility has been implemented in the framework of ekate. It automatically converts and integrates the stresses to extract the structural forces within the lining. This utility is initialized at the beginning of the simulation and the
inputs required for initialization are:

• A set of elements that forms a ring (ring (i) in figure A.1).
• A set of points that define tunnel lining path (points $C_i-2$ to $C_i+2$ in figure A.1).
• Lining central radius "$r_{avg}$" which defines the neutral axis.

Figure A.1: Lining rings in an arbitrary alignment and the definition of the axial direction for a specific ring

With the aforementioned inputs, the transformation matrix for stress transformation can be determined and stored within the utility initialization. The steps are summarized as follows:

1. Define the axial direction of the ring $n$ as the unit vector along ring width. For ring (i), shown in figure A.1, the axial direction can be determined from the central points as
   \[
   n = \frac{C_{i+1} - C_i}{\|C_{i+1} - C_i\|} \quad (A.1)
   \]
2. Loop over all the Gauss points in all the elements of the ring and the remaining steps are repeated for each Gauss point, see figure A.2.
3. Set the axial direction at the Gauss point $l$ as the axial direction of the ring, $l = n$.
4. Project the Gauss point on the axial direction. The projection is determined via the scalar parameter $\lambda$
   \[
   \lambda = (GP - C_i) \cdot n \quad (A.2)
   \]
5. Define the outward radial direction by the unit vector $r$ as
   \[
   r = \frac{R}{\|R\|}; \quad \text{where} \quad R = GP - C_i - \lambda n \quad (A.3)
   \]
6. Determine the tangential unit vector $\theta$ as the cross product of the axial and radial vectors, $\theta = l \times r$
7. With the definition of the coordinate system $\tilde{e} = \{l, r, \theta\}$ in hand, the transformation matrix can be introduced as:
   \[
   T = e \otimes \tilde{e} \quad (A.4)
   \]

At the end of a simulation step, the stored transformation matrix at each Gauss point can be used for the determination of the stresses in the cylindrical coordinates as follows:

\[
\tilde{\sigma} = T^T \cdot \sigma \cdot T \quad (A.5)
\]
A.2 DETERMINATION OF MEMBER FORCES

In practice, the structural member forces, i.e. normal forces, shear forces and bending moments, are used to check the safety of the lining. For any arbitrary cross section, the structural member forces in the ring direction, represented by the shaded cross section in figure A.3, are calculated from the cylindrical stresses by:

\[ N_{\theta\theta} = \sum_{elem} \int \sigma_{\theta\theta} \, dr \, dl \]

\[ M_{\theta l} = \sum_{elem} \int \sigma_{\theta\theta} (r - r_{avg}) \, dr \, dl \]

Figure A.2: Definition of cylindrical coordinate system at a Gauss point inside an element of a lining ring; \( l, \theta \) and \( r \) represent longitudinal, tangential and radial directions respectively.

Figure A.3: Representation of an element in the lining ring, in which the integration of the stress component \( \sigma_{\theta\theta} \) along the area \( dA = dr \, dl \) provides the element compression force in ring direction (noting that element natural coordinate is expressed as \( \xi_1 = \xi_l, \xi_2 = \xi_r \) and \( \xi_3 = \xi_\theta \) ).
The integrals of the previous equation are evaluated over the infinitesimal area vector \( dr \cdot dl \). In order to evaluate such integration numerically, the Jacobian has to be transformed from its regular form \( J \) into the cylindrical basis vectors form \( \tilde{J} \) where

\[
J = \frac{X}{\xi}; \quad X = \{X, Y, Z\} \text{ and } \xi = \{\xi_1, \xi_2, \xi_3\} \\
\tilde{J} = \frac{\tilde{X}}{\tilde{\xi}}; \quad \tilde{X} = \{l, \theta, r\} \text{ and } \tilde{\xi} = \{\xi_l, \xi_\theta, \xi_r\}
\]

Therefore, a permutation tensor \( \epsilon \) is introduced to reorder the natural coordinates \( \xi \) to the cylindrical coordinates \( \tilde{\xi} \) as follows:

\[
\epsilon = \epsilon_{ij} \xi \otimes \tilde{\xi}, \tag{A.8}
\]

and the Jacobean \( \tilde{J} \) is determined as:

\[
\tilde{J} = T^T \cdot J \cdot \epsilon \tag{A.9}
\]
Appendix B

Transformations of Loads between Vertical/Horizontal and Radial/Tangential Directions

Herein, the transformation of vertical and horizontal loads acting on tunnel lining into radial and tangential directions is discussed. This is useful for the comparison between loads for two different modeling schemes; first, the loads in the bedded lining model that are defined in vertical and horizontal directions. The second modeling scheme is the lining model in the 3D process-oriented simulation, in which the loads on the lining are obtained in radial and tangential directions.

Figure B.1: Illustration of the load components acting in vertical/horizontal and radial/tangential directions, as well as the geometrical relation in between

Figure B.1 shows the geometrical relation between the load components, i.e. $\sigma_v$, $\sigma_h$, $\sigma_r$ and $\sigma_t$, representing the vertical, horizontal, radial and tangential loads distributed per meter run. Therefore,
the vertical and horizontal loads distributed over an arc length of one meter are equal to \( \sigma_v \cos \varphi \) and \( \sigma_t \sin \varphi \) and the relation between the different components can be described as:

\[
\begin{bmatrix}
\sigma_r \\
\sigma_I
\end{bmatrix} =
\begin{bmatrix}
\cos \varphi & \sin \varphi \\
-\sin \varphi & \cos \varphi
\end{bmatrix} 
\cdot
\begin{bmatrix}
\sigma_v \cos \varphi \\
\sigma_h \sin \varphi
\end{bmatrix}
\]

or

\[
\begin{bmatrix}
\sigma_v \cos \varphi \\
\sigma_h \sin \varphi
\end{bmatrix} =
\begin{bmatrix}
\cos \varphi & -\sin \varphi \\
\sin \varphi & \cos \varphi
\end{bmatrix} 
\cdot
\begin{bmatrix}
\sigma_r \\
\sigma_I
\end{bmatrix}
\]
Appendix C

Calculation of Loads used for the Investigation of the Bedded Beam Models

The bedded beam model requires an a priori basic assumption for the acting loads that are usually based on the in-situ stress state. In section 6.3, different loading assumptions are adopted for the investigation of the bedded beam response for a certain scenario. In what follows, the calculations of the adopted loads are presented in detail. To facilitate the follow up, figure C.1 is provided, which summarizes all the needed parameters for the respective calculation.

Ground surface and ground water level

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$D$</td>
<td>9.20 m</td>
</tr>
<tr>
<td>$h$</td>
<td>15.64 m</td>
</tr>
<tr>
<td>$E_s$</td>
<td>50 MPa</td>
</tr>
<tr>
<td>$\gamma_s$</td>
<td>20 kN/m$^3$</td>
</tr>
<tr>
<td>$\gamma'$</td>
<td>10 kN/m$^3$</td>
</tr>
<tr>
<td>$\gamma_w$</td>
<td>10 kN/m$^3$</td>
</tr>
<tr>
<td>$k_o$</td>
<td>0.42</td>
</tr>
<tr>
<td>$E_c$</td>
<td>30 GPa</td>
</tr>
<tr>
<td>$\gamma_c$</td>
<td>25 kN/m$^3$</td>
</tr>
</tbody>
</table>

Figure C.1: Geometry and properties used for the calculations of loads adopted in the bedded beam models
C.1 Loading Assumption A

According to Ahrens et al. (1982), loads are applied as uniform pressures \( p \) and \( q \) as shown in figure C.2. They represent the vertical and horizontal total pressure loading at the tunnel crown and the tunnel center line respectively (total refers to soil and water pressures). Segment’s weight "\( g \)" is not considered in the analytical solution by Ahrens et al. (1982), however, it has been added herein for the sake of comparison with other load assumptions. Loads are defined as:

\[
p, \text{ total vertical pressure at the crown} = h \gamma' + h_w \gamma_w \approx 313 kN/m^2
\]

\[
q, \text{ total horizontal pressure at the tunnel centerline} = k_0 (h + D/2) \gamma' + (h_w + D/2) \gamma_w \approx 289 kN/m^2 \quad \text{(C.1)}
\]

\[
g, \text{ segments weight per unit length for a width of 1m} = \gamma_c \times \text{thickness} \times \text{width} = 25 \times 0.45 \times 1.0 = 11.25 kN/m
\]

C.2 Loading Assumption B

Figure C.3 shows the different load components according to the JSCE, in which, earth and water pressures are separately considered. The vertical earth and water pressures are determined at the tunnel crown, while horizontal loads linearly vary with the depth, with a minimum and maximum value at the crown and the invert respectively. The ring weight over the diameter is added as an upward distributed reaction force. Moreover, an additional lateral reaction that represents lateral soil confinement is adopted (noting that this component is not considered in the other loading assumptions investigated herein).
C.2. LOADING ASSUMPTION B

**Figure C.3:** Assumptions for the loading on tunnel lining according to JSCE JAPANESE SOCIETY OF CIVIL ENGINEERS (JSCE) (1996), Koyama (2003)

---

**earth pressure**

\[ p_{1e}^c, \text{vertical soil pressure at the crown} \]
\[ = h \gamma' \]
\[ = 15.64 \times 10 \approx 156 \text{ kN/m}^2 \]  
\[ q_{1e}^c, \text{lateral soil pressure at the crown} \]
\[ = k_0 h \gamma' \]
\[ = 0.429 \times 15.64 \times 10 \approx 67 \text{ kN/m}^2 \]  
\[ q_{2e}^c, \text{lateral soil pressure at the invert} \]
\[ = k_0 (h + D) \gamma' \]
\[ = 0.429 \times (15.64 + 9.2) \times 10 \approx 106 \text{ kN/m}^2 \]  

**water pressure**

\[ p_{1w}^w, \text{water pressure at the crown} \]
\[ = h_w \gamma_w \]
\[ = 15.64 \times 10 \approx 156 \text{ kN/m}^2 \]  
\[ p_{2w}^w, \text{water pressure at the invert} \]
\[ = (h_w + D) \gamma_w \]
\[ = (15.64 + 9.2) \times 10 \approx 248 \text{ kN/m}^2 \]  

**dead load reaction**

\[ R_g = \frac{\text{ring weight/m}}{D} = \frac{\pi/4 (D^2 - D_{int}^2) \gamma_{conc.}}{D} \]
\[ = \frac{\pi/4 (9.2^2 - 8.3^2) \times 25}{9.2} \approx 34 \text{ kN/m}^2 \]
soil reaction

\[ R_s = K \delta \]
\[ = 10,870 \times (1.954 \times 10^{-3}) \approx 21 \text{kN/m}^2 \]

\( K \), spring stiffness

\[ = \frac{E_s}{D/2} \]
\[ = \frac{50,000}{4.6} \approx 10,870 \text{kN/m}^3 \]

\( \delta \), lateral deformation at the tunnel center line

\[ = \frac{(2p_1 - q_1 - q_2) R^4}{24(E_c I + 0.0454 KR^4)} \]
\[ = \frac{(2 \times 312 - 223 - 354) \times 4.6^4}{24(227,812.5 + 0.0454 \times 10,870 \times 4.6^4)} \approx 1.954 \times 10^{-3} \text{m} \quad \text{(C.5)} \]

\( p_1 \), vertical pressure at the crown

\[ = p_1^c + p_1^w = 156 + 156 = 312 \text{kN/m}^2 \]

\( q_1 \), lateral pressure at the crown

\[ = q_1^c + p_1^w = 67 + 156 = 223 \text{kN/m}^2 \]

\( q_2 \), lateral pressure at the invert

\[ = q_2^c + p_2^w = 106 + 248 = 354 \text{kN/m}^2 \]

\( E_c I \), flexural stiffness of the lining

\[ = 30,000,000 \times \frac{1 \times 0.45^3}{12} \approx 227,812.5 \text{kNm}^2/\text{m} \]

### C.3 Loading Assumption C

This loading assumption, as shown in figure C.4, is quite similar to the loading assumption B, except that the soil reaction is not considered. As well, the distribution of water pressure is depicted
as radial pressure that linearly increase with depth. The determination of the loads can be found in section C.2, where same notations are used.

### C.4 Loading Assumption D

In comparison with loading assumption C, only the vertical pressure is different; on top, the pressure is not uniform and increases with depth. On bottom, the pressure is reduced by buoyancy forces. The distributions of loads are shown in figure C.5, while, the determination of vertical pressure is obtained as follow:

\[
\begin{align*}
    p_1^e &= h \gamma' \\
    &= 15.64 \times 10 \approx 156kN/m^2 \\
    p_2^e &= (h + D/2) \gamma' \\
    &= (15.64 + 4.6) \times 10 \approx 202kN/m^2 \\
    p_3^e &= p_1^e - \frac{\text{uplift force}}{D} \\
    &= 156 - 356/9.2 \approx 118kN/m^2 \\
\end{align*}
\]

\[\text{uplift force} = \text{buoyancy} - \text{ring weight}\]
\[= \pi/4 D^2 \gamma_w - \pi/4 (D^2 - D_{int}^2) \gamma_{\text{conc.}}\]
\[= \pi/4 \times 9.2^2 \times 10 - \pi/4 (9.2^2 - 8.3^2) \times 25 = 665 - 309 = 356kN/m\]

**Figure C.5:** Assumptions for the loading on tunnel lining according to design recommendation of a reference project


ACI COMMITTEE 318 (2008). Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary. Technical report, American Concrete Institute, 38800 Country Club Drive, Farmington Hills, MI 48331 U.S.A. [cited at p. 53]


DRUCKER, D. AND W. PRAGER (1952). Soil mechanics and plastic analysis or limit design. Quarterly of Applied Mathematics 10(2), 157–162. [cited at p. 34]


BIBLIOGRAPHY


List of Figures

1.1 Changes in urbanization across several regions or subregions of the world from 1950 to 2015; black line is the urbanization in developing countries between 1800 to 2015 as a base line for comparison [taken from (DESA 2018)] .................................................. 1

1.2 Representative figure of Mixshield components: (1) cutting wheel, (2) submerged wall, (3) air cushion, (4) jaw crusher, (5) bulkhead, (6) air lock, (7) slurry circuit, (8) thrust cylinders, (9) shield skin, (10) erector, (11) wire brushes and (12) Backfilling (TBM ©Herrenknecht AG) ............................................................... 2

1.3 Statistics of the individual construction methods of tunneling activities in Germany during the period 2016/2017 (SCHÄFER 2017) .................................................... 3

1.4 Metro lines I-VI in Greater Cairo. Some metro lines are currently in operation and the rest are under construction and planning phases. Feasibility studies are performed by SYSTRA 1998/2000 and JICA 2000/2002 (©Egyptian National Authority of Tunneling) ......................................................... 4

2.1 Longitudinal section of a hydro-shield machine showing its main structural components in black and shield equipment, cutting wheel and erector in light gray .............. 13

2.2 TBM induced ground support (i.e. (1) face support pressure at the tunnel face, (2) radial contact pressure along TBM length and (3) annular gap grouting pressure at the tail) .................................................. 14

2.3 Optimum application range of EPB shields and hydro shields (ZUMSTEG AND LANG-MAACK 2017) ................................................................. 16

2.4 Schematic illustration of the annular gap (WITTE 2006) ..................................... 16

2.5 Illustration of the infiltration process during mechanized tunneling and the formation of a filter cake (THIENERT 2011) ......................................................... 17

2.6 Mechanisms of slurry support pressure transfer: (a) membrane model, (b) suspension penetration model (HAUGWITZ AND PULSFORT 2009) and (c) hybrid model with partial filter cake and reduced penetration (THIENERT 2011) .................................................. 18

2.7 Stagnation of bentonite slurry in the test specimens (MIN ET AL. 2013): (a) complete filter cake, (b) slurry penetration and (c) partial filter cake and reduced penetration 18
2.8 Laboratory test for slurry penetration (XU ET AL. 2017): (a) sketch of test apparatus and (b) test results of water discharge and excess pressure for a bentonite content of 40 g/l .................................................. 19

2.9 Discretization at the tunnel face for simulating the pressure transfer mechanism (ZIZKA ET AL. 2016) ............................................................. 20

2.10 Grout infiltration model (SCHAUFLER 2015): (a) soil and grout at the micro-scale, b) the components of the four phase model ($\phi^s$ solid phase, $\phi^f$ fluid phase, $\phi^{sa}$ fines behave solid-like and $\phi^{af}$ fines behave fluid-like), (c) illustration of the 1-dim simulation model with boundary conditions and (d) numerical model for a cross section of a tunnel lining .................................................................................. 21

2.11 Components of the simulation model by MANSOUR (1996) .................. 22

2.12 Components of the simulation model by MROUEH AND SHAHROUR (2003) .... 22

2.13 Components of the simulation model by SCHMITT ET AL. (2005) ........ 23

2.14 Components of the simulation model by MÖLLER (2006) .................. 24

2.15 Components of the simulation model by KASPER (2005), NAGEL (2009) .... 25

2.16 Components of the simulation model by ZHAO ET AL. (2012) ............ 26

2.17 Components of the simulation model by DO ET AL. (2014a) .............. 27

2.18 Components of the simulation model by LAVASAN ET AL. (2017) ........ 28

2.19 Components of the simulation model by KAVVADAS ET AL. (2017) ....... 29

2.20 Computational model for mechanized tunneling ekate. left: main components involved in the simulation of the mechanized tunneling process and, right: finite element discretization of the model components; (1) Geological and ground Model, (2) Shield Machine, (3) Tunnel Lining, (4) Tail void grouting and (5) Thrust Jacks ............ 31

2.21 Fully saturated soil modeled according to TPM .................................. 32

2.22 Yield function in principal stress space and in the $p' - q$ plane: DRUCKER-PRAGER-model (left) and CLAY AND SAND-model (right) ......................... 34

2.23 Illustration of the main aspects related to the numerical representation of the shield machine: main structural components represented by the thick black lines (left) and radial distribution of hydraulic jacks (right) ............................................................... 35

2.24 Finite element mesh of the shield machine, the hydraulic jacks and the lining, and the geometrical parameters involved in the definition of the shield model .......................... 35

2.25 Annular gap grouting: (a) sketch of annular gap grouting through a nozzle in shield skin and (b) the process of grouting mortar hydration with stiffness and permeability evolution .................................................... 37

2.26 Development of grouting mortar properties with time; (a) permeability evolution for two different analysis parameters and (b) description of the parametric function $\beta_E(t)$ where the grout is fully hardened after 28 days .............................................. 37

2.27 Integration of buildings in numerical simulations; (a) 3D city model, (b) simplified volume geometries, (c) lower level discretization by shell or volume elements with substitute stiffness and (d) detailed discretization of the main structural components . . 40
2.28 Incorporation of buildings in the simulation model for tunnel advance by means of substitute stiffness and connected with the ground by Lagrange tying algorithm . . . . 

2.29 Prescribed boundary conditions of face support pressure: (a) stresses within the two phase element (total stresses $\sigma$, effective stresses $\sigma''$, partial solid stresses $\sigma'$ and water pressure $p''$), (b) formation of an impermeable filter cake (equation 2.24) and (c) penetration model with no filter cake sealing the tunnel face (equation 2.25) . . . . 

2.30 Shield positioning system according to FESTA ET AL. (2015) with the reference points (Reference Point at Front "RPF", Reference Point at Rear "RPR" and Reference Point at Cutting Wheel "RPCW") and the measuring device (Active Laser Target Unit "ALTP") . . . . 

2.31 Process liquid pressure around shield machine: (a) shield skin subdivisions at different location for the computation of fluid pressure and (b) FDM at a certain location along the shield axis with the final pressure combination at the steering gap . . . . . 

2.32 FE mesh of the ground with boundary conditions for the displacements components $u_x$, $u_y$, $u_z$ and pore pressure $P_w$ . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 

2.33 Preliminary steps at the beginning of the simulation of mechanized tunneling: (a) initial position of the shield at the model boundary with the initialization of contact analysis, (b) shield with free deformation supported by the soil pressure and the hydraulic jacks, situation before the start of step-wise simulation . . . . . . . 

2.34 Repetitive scheme for the step-wise simulation of mechanized tunneling process: (a) stand still position, (b) shield advancement and soil excavation achieved by means of the steering algorithm and the de/re-activation of the respective elements, (c) ring construction and resetting of the hydraulic jacks . . . . . . . 

3.1 Illustration of lining segments layout with longitudinal joints and ring joints . . . 

3.2 Detailed drawing of a typical rectangular precast concrete segment in a 7+1 ring layout; concrete dimensions in mm (top) and traditional reinforcement details using rebars (bottom) . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 

3.3 Types of segmental concrete lining rings with respect to ring tapering for the construction of straight and curved alignments: (a) straight ring, (b) tapered ring (left and right rings) and (c) universal ring (GUGLIELMETTI ET AL. 2007) . . . . . . . 

3.4 Depiction of the different types of longitudinal joints with flat, convex and convex/concave contact surfaces . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 

3.5 Depiction of the different types of ring joints: flat surface, tongue-and-groove connection and cam-and-pocket connection . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 

3.6 Illustration of the different connecting systems in concrete segments: shear dowels in ring joints and curved bolts in longitudinal joints (top) and inclined bolts in both ring and longitudinal joints (bottom) . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 

3.7 Lining bedding assumption used in analytical continuum and bedded models (PUTKE 2016); partial bedding (left) and full bedding (right) . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 

3.8 The in-situ stress loading assumption with partial bedding for the analytical solution by SCHULZE AND Duddeck (1964) . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 

3.9 Illustration of the different cross-sections of segmental concrete lining rings and ends (left) and typical cross-section for prefabrication (right) . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 

3.10 Depiction of the different types of segmental concrete lining rings with respect to the method of assembly: (a) cast-in-place, (b) precast concrete and (c) prefabricated rings (PUTKE 2016) . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 

3.11 Depiction of the different methods of assembly used in segmental concrete lining: (a) cast-in-place, (b) precast concrete and (c) prefabricated rings (PUTKE 2016) . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
3.9 Simplified loading assumptions used in the analytical continuum model by Ahrens et al. (1982) in which the vertical stresses are in equilibrium and the horizontal pressures are uniform .................................................. 61
3.10 Loading components used for lining design in analytical solution by the JSCE including earth pressure, soil reaction, water pressure, surcharge loading and lining weight . . 62
3.11 Moment distribution at the joint and the middle of the segment within a uniform rigidity model according to ITA Working Group No. 2 (2000) .......................... 63
3.12 Analytical model developed by Blom (2002); the geometry of the two lining rings including joint location (left) and schematic overview and structural model of the ring-to-ring coupling by means of shear springs and the longitudinal joints by means of rotational springs (right) ............................................................... 64
3.13 Numerical lining model presented by Koyama (2003); structural beam model with full bedding in radial and tangential direction (left) and main loading assumption considered including vertical earth pressure, horizontal earth pressure and water pressure in radial direction (right) ................................................................. 66
3.14 Bedded beam model of two consecutive coupled rings (different diameters expanded for illustration purposes only) ................................................................. 67
3.15 Relation between joint eccentricity \( m \) and joint rotation \( \varphi \) according to Leonhardt and Reimann (1966) ................................................................. 68
3.16 Moment-rotation relationship for segment joints under various normal pressures and the respective idealization for the joint behavior indicated by the dashed line (taken from Thiernert and Pulsfort (2011)) ................................. 68
3.17 Description of the cam-and-pot ring coupling according to Daub; left, geometrical configuration of the connection with different states of relative motion and left, simplification of the relation between coupling force and relative deformation (here, the possible slip is identified according to the clearance between the cam and pot) . . 68
3.18 Typical rectangular concrete lining segment of L9 subway tunnel used in the analysis of tunnel lining in (Arnaud and Molins 2011); segment dimension, configuration of longitudinal joint and packer locations (left) and FE discretization of the segment using shell elements and interface elements for the representation of longitudinal and ring joints (right) ................................................................. 69
3.19 Loading assumption used in the analysis of Barcelona-L9 tunnel lining for the bedded shell model by Arnaud and Molins (2012) including soil and water pressure . . 69
3.20 Three dimensional finite element model for the "Green Heart" tunnel developed by Blom et al. (1999) using interface elements for the representation of joints . . 70
3.21 Ring joint stiffness in the axial, radial and rotational directions as developed by Do et al. (2014a) ................................................................. 71
3.22 The numerical representation of longitudinal and ring joints via rotational springs and shear springs in the simulation model by Kavvadas et al. (2017) .................. 72
3.23 3D volume representation of the segmental lining model including segments joints via contact and embedded bolts (Chenghua et al. 2016) .................. 72
4.1 Numerical investigation of the effect of design related parameters; dimensions used in the simulation models (left) and finite element mesh with the detailed model components and the shield geometrical parameters (i.e. overcut, conicity and length) (right) 76

4.2 (a) Computed surface settlements at the monitoring point during shield advance, (b) radial loading on lining, (c) normal forces and (d) bending moments at the monitoring section at the steady state for different soil material response 78

4.3 Volumetric plastic deformations in the soil at the crown, springline and invert of the tunnel at the steady state 79

4.4 (a) Computed surface settlements at the monitoring point during shield advance, (b) radial loading on lining, (c) normal forces and (d) bending moments at the monitoring section at the steady state for different coefficients of lateral earth pressure 81

4.5 (a) Computed surface settlements at the monitoring point during shield advance, (b) radial loading on lining, (c) normal forces and (d) bending moments at the monitoring section at the steady state for different levels of ground water table 83

4.6 (a) Computed surface settlements at the monitoring point during shield advance, (b) radial loading on lining, (c) normal forces and (d) bending moments at the monitoring section at the steady state for different shield geometries 84

4.7 (a) Computed surface settlements at the monitoring point during shield advance, (b) radial loading on lining, (c) normal forces and (d) bending moments at the monitoring section at a steady state value, for different friction coefficient between the shield skin and the excavated ground 86

4.8 Longitudinal deformation along tunnel axis (in meters with 50x magnification): (a) frictionless contact between the shield and the soil and (b) case of frictional contact with a friction coefficient of 0.50 in which red arrows indicates the direction of frictional forces 87

4.9 (a) Computed surface settlements at the monitoring point during shield advance, (b) radial loading on lining, (c) normal forces and (d) bending moments at the monitoring section at the steady state for different levels of annular gap grouting pressure 88

4.10 (a) Computed surface settlements at the monitoring point during shield advance, (b) radial loading on the lining, (c) normal forces and (d) bending moments at the monitoring section at the steady state for two different grout material models 89

4.11 Numerical investigation of the effect of the driven tunnel path; finite element mesh for a curved alignment with half of the ground domain and a detailed representation of the tapered lining rings and the shield machine 90

4.12 (a) Computed radial loading on the lining, (b) normal forces and (c) bending moments at the monitoring section at the steady state during shield advance along straight and curved alignments 91

4.13 Schematic illustration of the shield orientation upon driving along curved alignment (top view) 92
4.14 Computed range of loading on lining from the simulation model with various model parameters (gray shaded area): loading in radial direction (left) and loading in tangential direction (right), noting that the variations in the coefficient of lateral earth pressure and the ground water level are excluded ................................................................. 93

4.15 Computed range of loading on the lining in vertical and horizontal directions from the simulation model (gray shaded area) in comparison with the loading assumption according to ITA WORKING GROUP NO. 2 (2000) (solid line) ......................... 94

5.1  ekate representative model for segmental lining geometry including bolts and dowels ................................................................. 96

5.2  Definition of contact surfaces between the segments as defined in the numerical model 96

5.3  Representation of bolts and dowels in segmental lining joints ................................. 97

5.4  Different joint arrangements in segmental tunnel lining model ............................... 98

5.5  Modeling of lining-soil interactions for the continuous (left) and the segmental lining (right) ................................................................. 99

5.6  Illustration of contact problem: contact between two bodies $\Omega_{s,m}$ via their contact surfaces $\Gamma_{s,m}$ (left) and contact surfaces in the deformed configuration in which the contact point $x^s$, its projection $x^m(x^s)$ and the coordinate system $[n, \tau_{ab}]$ are depicted (right) 99

5.7  Representation of tangential vectors $\tau_{ab}$ at the projected contact point $x^m(x^s) \in \Gamma_{c}m$ 101

5.8  Penalization of energy functional due to contact penetration, shaded area indicates the impermissible penetration zone ................................. 102

5.9  Relation between normal contact traction and gap according to penalty approximation of Kuhn-Tucker condition for normal contact ................................. 102

5.10 Regularization of Coulomb friction law using frictional penalty ................................. 103

5.11 Basic description for the implementation of contact algorithm in Kratos ................................. 104

5.12 Verification of frictional contact model: (a) geometry of the benchmark example with boundary conditions and (b) vertical deformation for the contacting bodies ................................. 106

5.13 Normal penalty constraint; normal contact force vs. vertical displacement for different normal penalties ................................................................. 107

5.14 Tangential penalty constraint; tangential contact force vs. horizontal displacement for different tangential penalties ................................................................. 107

5.15 Normal contact force vs. horizontal displacement for different friction penalties ................................. 107

5.16 Segment joint rotation test: (a) Experimental setup and (b) FE mesh and boundary conditions ................................................................. 108

5.17 Segment joint rotation test: vertical deformations of the model as obtained from the numerical simulation at a normal force level of 1600 kN/m ................................. 109

5.18 Segment joint rotation test: Moment-rotation relationship (a) comparison between experimental data and analytical solution and (b) comparison between experimental data and numerical solution ................................................................. 109

5.19 Full-scale test of BRT Segments at TU-Delf (BLOM AND VAN OOSTERHOUT 2001) 110

5.20 Full-scale test of tunnel segments: joints arrangement for the top, middle and bottom rings ................................................................. 111
5.21 Full-scale test of tunnel segments: radially applied loads and its subdivision into a uniform compressive load and an ovalising load ........................................... 111
5.22 Description of the numerical model of the Full-scale test: segments volumes with the applied loading and displacement boundary conditions (left) and discretization of one segment (right) ......................................................... 112
5.23 Definition of contact surfaces in the numerical model of the Full-scale test ............... 112
5.24 Re-analysis of full-scale test of tunnel segments: comparison between the measured radial deformations and the predicted ones from the numerical simulation; top ring (top) and middle ring (bottom). The locations of the joints are indicated by vertical dashed lines ......................................................... 113
5.25 Re-analysis of full-scale test of tunnel segments: comparison between the measured tangential bending stresses and the predicted bending stresses from the numerical simulation for the top ring (top) and the middle ring (bottom). The locations of the joints are indicated by vertical dashed lines ......................................................... 114
5.26 Re-analysis of full-scale test of tunnel segments: Comparison between the measured tangential bending stresses and the predicted bending stresses from the numerical simulation for the top ring (top) and the middle ring (bottom). The locations of the joints are indicated by vertical dashed lines ......................................................... 115
6.1 Numerical analysis of the structural forces in the linings of a straight tunnel driven in soft soil: dimensions and finite element mesh used in the simulation model ........ 118
6.2 Dimensions of the staggered segmental tunnel lining rings used in the investigated simulation model ................................................................. 118
6.3 Development of mechanical properties and permeability for the annular gap grouting mortar with time. Circles indicate to experimental measurements of stiffness evolution according to (SCHULTE-SCHREPPING ET AL. 2018) ............................................. 119
6.4 Segmental lining model embedded in process oriented advancement simulation: spatio-temporal response of the segmental lining for the investigated tunnel section (normal forces, bending moments and radial deformations) at four construction stages (1, 3, 7 and 15 rings after installation) ......................................................... 120
6.5 Segmental lining model embedded in process oriented advancement simulation: computed horizontal and vertical convergence of the lining ......................................................... 121
6.6 Comparison of segment-wise segment installation model and continuous lining model embedded in the process oriented advancement model ekate: Distribution of normal forces (top) and bending moments (bottom) at steady state ......................................................... 122
6.7 Influence of lining modeling approaches on surface settlement profile .................. 123
6.8 Influence of different overburden on the computed structural forces in segmental tunnel linings ................................................................. 124
6.9 Influence of different overburden on the computed moment-rotation relationships for two joints (dots: 3D model, dashed lines: analytical joint model) .................. 124
6.10 Influence of different joint patterns on the computed bending moments in segmental tunnel linings ................................................................. 125
6.11 Influence of different joint patterns on the computed moment-rotation relationship . 127
6.12 Bedded beam model for tunnel linings analysis: (a) structural model with non-linear rotational springs and shear springs, and (b) model with different bedding assumptions 128
6.13 Bedded beam model for tunnel linings analysis: (a) the non-linear rotational springs stiffness for the description of longitudinal joints, and (b) coupling force vs. relative deformation for the description of ring joints . 128
6.14 Adopted loading assumptions for the investigation of the bedded beam model responses according to (a) Ahrens AHRENS ET AL. (1982), (b) JSCE JAPANESE SOCIETY OF CIVIL ENGINEERS (JSCE) (1996), KOYAMA (2003), (c) ITA (ITA WORKING GROUP NO. 2 2000) and (d) design recommendation of a reference project . 129
6.15 Predicted lining structural forces for different bedded beam models with loading assumptions adopted according to Figure 6.14 . 130
6.16 Predicted loading on lining as obtained from the segment wise lining installation incorporated in a 3D advancement simulation model (blue curves) in comparison with the adopted loading assumptions in the bedded beam models (gray shaded area) . 131
6.17 Predicted lining responses as obtained from the 3D advancement simulation model in comparison with the predicted range of responses from the bedded beam models with various loading assumptions (gray shaded area) . 132
6.18 Predicted eccentricities in lining longitudinal cross section as obtained from the segment wise lining installation incorporated in a 3D advancement simulation model (blue curves) in comparison with the adopted loading assumptions in the bedded beam models (gray shaded area) . 133
6.19 Illustration of the lining’s longitudinal cross section: concrete dimensions and steel re-bars reinforcement (left) and strain and stress distributions in concrete along the thickness (right) 134
6.20 Bending moment and normal force interaction curve for the evaluation of the reinforcement amount using results obtained from the 3D simulation model in comparison with the bedded beam model considering different K_0 values . 135
6.21 Bending moment and normal force interaction curve for the evaluation of the reinforcement amount using results obtained from the 3D simulation model in comparison with the bedded beam model considering different level of water table . 136

7.1 Coupling of BIM with numerical simulation: components of the BIM model including the ground model, the geological and monitoring data as well as the models for the TBM, the tunnel lining and the buildings (top), and the numerical simulation including the CAD model, the finite element mesh and the simulation results (bottom) (ALS AHLY ET AL. 2018) 138
7.2 Schematic representation for the BIM-FEM technology for a reference project . 139
7.3 Schematic representation of different level of detail for the settlement prediction, building idealization and damage assessment method for the definition of multi-stage damage assessment [taken from ŒBEL ET AL. (2018a)] 141
7.4 Damage assessment of a building according to Burland and Wroth (1975): (a) actual masonry building, (b) idealization as a beam, (c) damage in bending mode and (d) damage in shear mode ......................................................... 144
7.5 Investigation of tunneling induced building damage: plan view of buildings with respect to tunnel alignment, building 3 with thick black borders is under detailed investigation 144
7.6 Predicted surface settlements in green field condition and w.r.t various building discretization .................................................................................................................................................. 145
7.7 Illustration of the three phases (liquid water, ice and solid) involved in freezing soil (left) and averaging principle on the macroscopic level (right) (Zhou and Meschke 2013) ........................................................................................................................................... 147
7.8 Numerical simulation of AGF in tunneling: Geometry and dimensions of the problem (material parameters involved in the freezing soil simulation are provided in (Marwan et al. 2016)) .................................................................................................................................................. 147
7.9 Influence of seepage flow on the formation of a frozen arch for an equidistant distribution of freeze pipes .................................................................................................................................................. 148
7.10 Formation of the frozen arch with seepage flow of $v_L = 1.5$ m/d for an equidistant distribution of freeze pipes .................................................................................................................................................. 149
7.11 Ant Colony Optimization: Illustration of discrete solution space, of note is that each column resembles one variable including the set of attached discrete values ........................................................................................................................ 151
7.12 Oval frozen zone developing during soil freezing in presence of ground water (Jumikis 1979) .................................................................................................................................................................. 153
7.13 Frozen arch profile: (a) in the absence and (b) in the presence of seepage flow ($v_L = 1.0$ m/d) ............................................................................................................................................................... 154
7.14 Transition function ($\delta R (\theta_i)$) for the determination of the radii of the freeze points ........................................................................................................................................................................ 154
7.15 Total time required to obtain a fully frozen arch for different seepage velocities with equidistant and optimized arrangement of the freeze pipes, dotted lines refer to the function fitted to data points .................................................................................................................................. 156
7.16 Evolution of time required to obtain a fully frozen arch during the iterative optimization procedure for $v_L = 1.0$ m/d ...................................................................................................................................................... 156
7.17 Optimum arrangement of freeze pipes for $v_L = 1.0$ m/d in comparison with an equal distribution of the pipes ...................................................................................................................................................... 157
7.18 Influence of seepage flow on the formation of a frozen arch for an optimized arrangement of the freeze pipes ...................................................................................................................................................... 157
A.1 Lining rings in an arbitrary alignment and the definition of the axial direction for a specific ring .................................................................................................................................................................. 168
A.2 Definition of cylindrical coordinate system at a Gauss point inside an element of a lining ring; $l, \theta$ and $r$ represent longitudinal, tangential and radial directions respectively ........................................................................................................................................... 169
A.3 Representation of an element in the lining ring, in which the integration of the stress component $\sigma_{\theta\theta}$ along the area $dA = dr dl$ provides the element compression force in ring direction (noting that element natural coordinate is expressed as $\xi_1 = \xi_l, \xi_2 = \xi_r$ and $\xi_3 = \xi_\theta$) ............................................................................................................................................................ 169
B.1 Illustration of the load components acting in vertical/horizontal and radial/tangential directions, as well as the geometrical relation in between ........................................ 171

C.1 Geometry and properties used for the calculations of loads adopted in the bedded beam models ....................................................................................... 173

C.2 Assumptions for the loading on tunnel lining according to AHRENS ET AL. (1982) 174

C.3 Assumptions for the loading on tunnel lining according to JSCE JAPANESE SOCIETY OF CIVIL ENGINEERS (JSCE) (1996), KOYAMA (2003) ......................... 175

C.4 Assumptions for the loading on tunnel lining according to ITA WORKING GROUP NO. 2 (2000) ................................................................. 176

C.5 Assumptions for the loading on tunnel lining according to design recommendation of a reference project ............................................................... 177
## List of Tables

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1</td>
<td>Development of computational models for mechanized tunneling simulation</td>
<td>30</td>
</tr>
<tr>
<td>4.1</td>
<td>Design related parameters and their variations for the parametric investigation</td>
<td>77</td>
</tr>
<tr>
<td>4.2</td>
<td>Maximum predicted surface settlements and the radial loading on the lining at different locations with respect to different material behavior</td>
<td>79</td>
</tr>
<tr>
<td>4.3</td>
<td>Maximum predicted structural forces and their deviation for different material behavior</td>
<td>79</td>
</tr>
<tr>
<td>4.4</td>
<td>Maximum predicted surface settlements and the radial loading on the lining at different locations with respect to different coefficients of lateral earth pressure</td>
<td>80</td>
</tr>
<tr>
<td>4.5</td>
<td>Maximum predicted structural forces and their deviation for different coefficients of lateral earth pressure</td>
<td>80</td>
</tr>
<tr>
<td>4.6</td>
<td>Maximum predicted surface settlements and the radial loading on the lining at different locations with respect to different levels of ground water table</td>
<td>82</td>
</tr>
<tr>
<td>4.7</td>
<td>Maximum predicted structural forces and their deviation for different levels of ground water table</td>
<td>82</td>
</tr>
<tr>
<td>4.8</td>
<td>Maximum predicted surface settlements and the radial loading on the lining at different locations with respect to different shield geometries</td>
<td>85</td>
</tr>
<tr>
<td>4.9</td>
<td>Maximum predicted structural forces and their deviation for different shield geometries</td>
<td>85</td>
</tr>
<tr>
<td>4.10</td>
<td>Maximum predicted surface settlements and the radial loading on the lining at different locations with respect to different levels of annular gap grouting pressure</td>
<td>87</td>
</tr>
<tr>
<td>4.11</td>
<td>Maximum predicted structural forces and their deviation for different levels of annular gap grouting pressure</td>
<td>88</td>
</tr>
<tr>
<td>4.12</td>
<td>Maximum predicted surface settlements and the radial loading on the lining at different locations with respect to two different grout material models</td>
<td>90</td>
</tr>
<tr>
<td>4.13</td>
<td>Maximum predicted structural forces and their deviation for two different grout material models</td>
<td>90</td>
</tr>
<tr>
<td>5.1</td>
<td>Geometrical and material parameter of the full-scale test</td>
<td>111</td>
</tr>
</tbody>
</table>
6.1 List of concrete and steel properties used for defining the cross sectional capacity (i.e. interaction curve) ................................. 134

7.1 Damage categories and limiting strains according to BOSCARDIN AND CORDING (1989) 143
7.2 Model complexity with different levels of damage assessment ................................. 145
7.3 Total freezing time required for obtaining a fully frozen arch at different seepage flows with equidistant and optimized arrangement of the freeze pipes. ................................. 155
About the author

Personal information

Name
Ahmed Marwan

Date of birth
26.03.1985

Place of birth
Minia, Egypt

Nationality
Egyptian

Address
IC 6/173, Universitätsstraße 150,
44801 Bochum, Germany

Telephone
0234 32 29062

E-mail
ahmed.marwan@rub.de
ahmed.marwan@mu.edu.eg

Educational background

06.2013 – 04.2019
Doctoral study at Faculty of Civil and Environmental Engineering, Ruhr University Bochum, Germany

09.2006 – 07.2010
Master’s degree in Structural Engineering at Faculty of Civil Engineering, Minia University, Egypt. Thesis title: "Numerical Modelling of NATM for Egyptian Tunnels"

09.2001 – 06.2006
Bachelor’s degree in Civil Engineering at Faculty of Civil Engineering, Minia University, Egypt

09.1998 – 07.2001
High School degree, Minia, Egypt

Professional experience

06.2013 – 04.2019
Research associate at Institute for Structural Mechanics, Ruhr University Bochum, Germany

04.2007 – 06.2013
Assistant lecturer at Faculty of Civil Engineering, Minia University, Minia, Egypt.

09.2008 – 06.2013
Design Engineer (part-time) at Cairo Consulting Group, Cairo, Egypt.
Publication

Journal articles


Conference papers


**Co-Auther of Handbooks**
